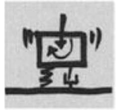
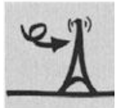
VIBRATION PROBLEMS IN STRUCTURES





PRACTICAL GUIDELINES

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Preface

Modern structures such as buildings, factories, gymnasia, concert halls, bridges, towers, masts and chimneys can be severely affected by vibrations. Vibrations can cause either serviceability problems reducing people's comfort to an unacceptable level or safety problems with danger of failure.

The aim of this book is to give guidelines for the practical treatment of vibration problems in structures. The guidelines are mainly aimed at practising structural and civil engineers who are working in construction and environmental engineering but are not specialists in dynamics.

In four chapters with totally twenty sub-chapters, tools are given to aid in decisionmaking and to find simple solutions for cases of frequently occuring "normal" vibration problems. For more complicated problems and for more advanced solutions further hints are given. In such cases these guidelines should enable the user to proceed in the right direction for finding the appropriate solutions - for example, in the literature - and possibly assist him to communicate authoritatively with a dynamic specialist.

Dynamic actions are considered from the following sources of vibration:
- human body motions
- rotating, oscillating and impacting machines
- wind flow
- road traffic, railway traffic and construction work.

Earthquake-induced vibrations, impact problems and fatigue effects are not treated in these guidelines. Such problems have to be solved using relevant sources from literature.

For an easier use of the guidelines each sub-chapter has a similar format and structure of content:

1 Problem description
2 Dynamic actions
3 Structural criteria
4 Effects
5 Tolerable values
6 Simple design rules
7 More advanced design rules
8 Remedial measures

In ten appendices important theoretical and practical fundamentals are summarised. The basic vibration theory and other significant definitions are treated, and often used numerical values are given. These fundamentals may serve for a better understanding and use of the main chapters.

It is not intended that these guidelines should replace relevant national codes. The guidelines have been compiled so as to give more general rules and more general hints than are detailed in national codes. Whenever appropriate, however, codes and standards have been referenced for illustrative purposes.

The present guidelines were elaborated by an international Task Group "Vibrations" of the "Comité Euro-International du Béton (CEB)". They were originally published as "Bulletin d'Information No. 209". After using and testing the Bulletin over the past three years leading to some modifications, the guidelines are now to be published as a book enabling a broader use in practice.

The authors would like to thank the Comité Euro-International du Béton for allowing the publication of the Bulletin as a book. Sincere thanks are addressed to Mrs. Tilly Grob, Mr. Marco Galli, Mr. Guido Göseli and Mr. Lucien Sieger from the Institute of Structural Engineering (IBK) of the Swiss Federal Institute of Technology (ETH), Zürich, Switzerland, for their untiring and careful work in processing the text and drawing the figures. And last but not least, as chairman of the former CEB Task Group "Vibrations", the first author would like to express his thanks to all members of the group for their sustained support during this challenging work.

Zürich, September 1994 Hugo Bachmann

Preface to the second edition

The authors are pleased about the interest shown by the profession in this book, necessitating the printing of a second edition less than two years after appearance of the first edition. In this second edition, apart from correcting a few printing errors, no substantial changes have been made.

Zürich, November 1996 Hugo Bachmann

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1 Vibrations induced by people

H. Bachmann, A.J. Pretlove, J.H. Rainer

This chapter deals with structural vibrations caused by human body motions. Of great importance are vibrations induced by rhythmical body motions such as

- walking
- running
- jumping
- dancing
- handclapping with body bouncing while standing
- handclapping while being seated
- lateral body swaying.

Of minor importance are vibrations induced by single body motions such as

- heel impact
- jumping off impact
- landing impact after jumping from an elevated position.

Vibrations induced by people may strongly affect the serviceability and, in rare cases, the fatigue behaviour and safety of structures.

In this chapter man-induced vibrations of the following structure types are treated in subchapters:

1.1 Pedestrian bridges
1.2 Floors with walking people
1.3 Floors for sport or dance activities
1.4 Floors with fixed seating and spectator galleries
1.5 High-diving platforms.

The dynamic forces from rhythmical human body motions are given in Appendix G. Other fundamentals are given in the other appendices.

1.1 Pedestrian bridges

A.J. Pretlove, J.H. Rainer, H. Bachmann

1.1.1 Problem description

Structures affected by pedestrians are predominantly footbridges, but there are similar problems associated with stairways and ship gangways. Stairways are usually much stiffer structures than bridges and on ship gangways there is rather more expectation of vibration by the user and it is therefore more tolerable. The vibration of floors in buildings caused by people, and their psychological response to it is very similar to that of pedestrian bridges and this is discussed in Sub-Chapter 1.2. This section will confine itself entirely to pedestrian footbridges.

Footbridges are usually constructed of continuous concrete or steel, some of them being composite. They may have a large number of spans but it is usually three or less. Timber, cast iron and aluminium alloy are much less common. The economics of modern design and construction dictates that the structural design be efficient in terms of material volume. This has increasingly led to slender and flexible structures with attendant liveliness in vibration.

1.1.2 Dynamic actions

In most cases the vibration problem is one of forced motion caused by the stepping rate of pedestrians (see Table G.1). The average walking rate is 2 Hz with a standard deviation of 0.175 Hz. This means that 50% of pedestrians walk at rates between 1.9 Hz and 2.1 Hz or, alternatively, 95% of pedestrians walk at rates between 1.65 and 2.35 Hz. Depending on the span of the bridge only a finite number of steps is taken to cross the bridge. As a result the motion is often one of a transient nature, no steady-state being reached. Some bridges have to accommodate running pedestrians and this can be at a rate of up to 3.5 Hz, but usually not beyond. The frequency of the second and third harmonic of the normal walking rate at 4 Hz and 6 Hz can be important, particularly for structures with coincident natural frequencies. The forcing spectrum is somewhat different for men and women. Two (or more) persons walking together often walk in step, naturally, and this can increase the forces. The dynamic forces from walking and running can be modelled as shown in Appendix G.

Vandal loading has been considered by some authorities, see [1.1]. Except in unusual circumstances the worst case to be considered is two or three pedestrians walking or running in step at the fundamental natural frequency of the bridge. Footbridges may often be modelled as equivalent single-degree-of-freedom (SDOF) systems (see Appendix A).

1.1.3 Structural criteria

a) Natural frequencies

The condition most to be avoided is a coincidence of average walking rate with a natural frequency. Figure 1.1 shows an assembly of data from different parts of the world for 67 footbridges. Also shown is the band of walking rates expected from 95% of the pedestrian population.

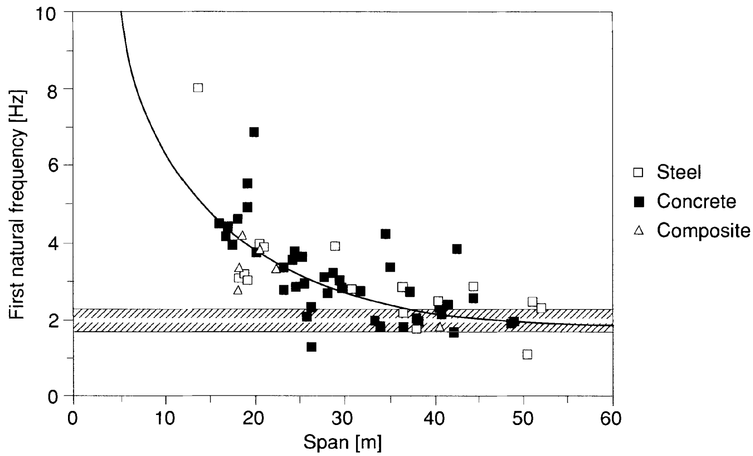


Figure 1.1: Footbridge fundamental frequency as a function of span

Taking all bridges together a least squares fit shows this data to follow the relationship

$$f_1 = 33.6 \cdot L^{-0.73} \tag{1.1}$$

where L = span [m]
f_1 = fundamental natural frequency [Hz]

It can be seen that there is a good deal of spread in the data. Similar relationships can be deduced for the various construction types (materials), as follows:

Concrete	$f_1 = 39 \cdot L^{-0.77}$
Steel	$f_1 = 35 \cdot L^{-0.73}$
Composite (caution: only 6 data points)	$f_1 = 42 \cdot L^{-0.84}$

It follows from this data that there is an increased likelihood of problems arising for spans in the following ranges:

Concrete	$L \geq 25\text{m}$
Steel	$L \geq 35\text{m}$

It must also be remembered that short span bridges with fundamental natural frequencies at a multiple of the walking rate can also have significant problems. The formulae above give a useful guide to fundamental natural frequency but such values can not replace a proper design prediction.

b) Damping

Modern, structurally-efficient footbridges, particularly in steel and prestressed concrete have very little vibration damping. As a result, vibrations can build up very steeply during the passage of a pedestrian. At higher levels of vibration, damping increases and this may serve to limit the vibration, though not before it has exceeded acceptable levels. Data from 43 UK footbridges show the values for the equivalent viscous damping ratio ζ (measured at the vibration level caused by one pedestrian walking at the bridge natural frequency f_1) given in Table 1.1.

	damping ratio ζ		
Construction type	min.	mean	max.
Reinforced concrete	0.008	0.013	0.020
Prestressed concrete	0.005	0.010	0.017
Composite	0.003	0.006	--
Steel	0.002	0.004	--

Table 1.1: Common values of damping ratio ζ for footbridges

This data shows that more problems might be expected from steel footbridges than from concrete ones. This is borne out by Figure 1.2. Note that it is not possible at present to predict the damping value for a bridge with any accuracy. The use of past experience, as given for example in Table 1.1, is the best present guide to design. For an acceleration limit of 0.7 m/s^2 (see

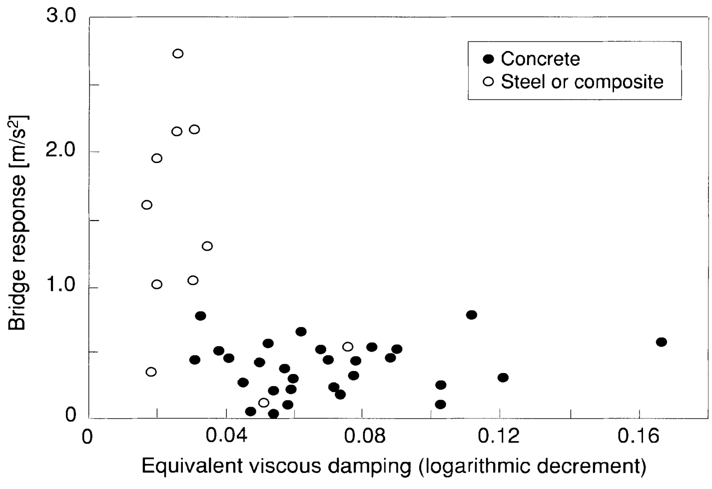


Figure 1.2: Response of footbridges to a pedestrian walking at f_1 for different values of damping [1.14]

Section 1.1.5) Figure 1.2 shows that a problem with vibration is not very likely to occur if the damping ratio is greater than 0.006 (logarithmic decrement of ~ 0.04). Further information on damping may be found in [1.1] and [1.14] and in Appendix C.

c) Stiffness

The stiffness of a footbridge (point force divided by point deflection at centre span) is a factor which can be predicted with some accuracy provided that the constraints offered by supports and abutments can be defined. Measured stiffnesses are generally less for steel structures than for concrete. Overall they are typically in the range of 2 to 30 kN/mm. Figure 1.3 shows how maximum bridge response varies with bridge stiffness for a pedestrian walking at the bridge natural frequency f_1. If an acceleration limit of 0.7 m/s^2 is accepted (see Section 1.1.5) then it may be concluded that no vibration problem is likely to arise if the stiffness is greater than 8 kN/mm.

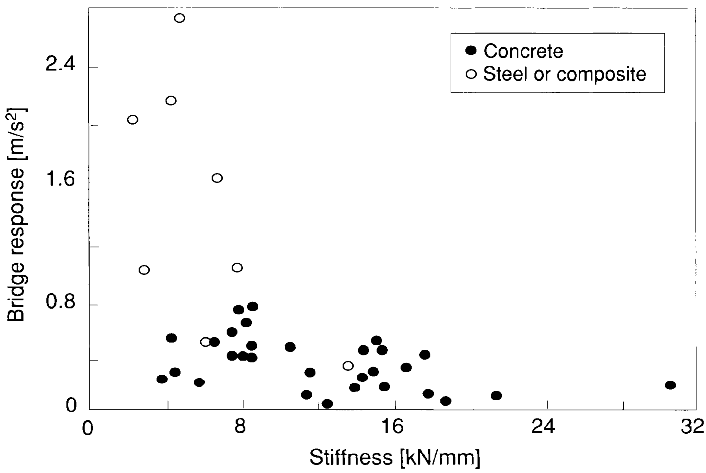


Figure 1.3: Bridge response to a pedestrian walking at f_1 in relation to stiffness [1.1]

1.1.4 Effects

A general account of the effect of vibration on people is given in Appendix I. For the purpose of footbridge vibrations specific design targets are given in bridge design codes (see Section 1.1.5). To give some idea of relevant levels of vibration acceleration for vertical vibration of pedestrians, reference can be made to [1.2] where a "severe" response can be expected at 2 Hz at an acceleration level of 0.7 m/s^2. A common human problem is that motion causes the pedestrian to become anxious about the safety of the structure even to the extent of refusing to use it. In such cases the actual danger of structural collapse is most unlikely, the strains involved often being 10 to 100 times less than those which might initiate damage. Nevertheless it is a serious matter for the designer and account must be taken of the human response to vibration in terms of disquiet, anxiety or even fear.

1.1.5 Tolerable values

The approximate limits of acceptability for vibration acceleration have already been indicated. There are only two national bridge design codes which take pedestrian response to vibration into account (see Section 1.1.6). [BS 5400] gives a vibrational acceleration serviceability limit of

$$0.5 \cdot f_1^{\,0.5} \ [\mathrm{m/s^2}] \tag{1.2}$$

for fundamental natural frequencies f_1 (in Hz) less than 5 Hz. At the vulnerable bridge frequency of 2 Hz this gives a limit of 0.7 $\mathrm{m/s^2}$.

The Ontario bridge code [ONT 83] is rather more conservative. A criterion has been selected by consideration of a large number of experimental results on human tolerance. A mean line is given in graphical form which corresponds to a serviceability acceleration limit of

$$0.25 \cdot f_1^{\,0.78} \ [\mathrm{m/s^2}] \tag{1.3}$$

At 2 Hz this gives a limit of 0.43 $\mathrm{m/s^2}$.These limits are stated for a bridge excitation by one pedestrian. No allowance is made for multiple random arrivals of pedestrians.

In [ISO/DIS 10137] the suggested tolerable value for vibration of footbridges is 60 times the base curves [ISO 2631/2]. At 2 Hz and in the vertical direction this gives an r.m.s. acceleration of about 0.42 $\mathrm{m/s^2}$ or a peak value of 0.59 $\mathrm{m/s^2}$: from 4 to 8 Hz this suggested tolerable peak value is 0.42 $\mathrm{m/s^2}$.

If the more advanced design methods of Section 1.1.7 are used, there are no agreed tolerable values. However, it is clear that an acceleration limit of about 0.5 $\mathrm{m/s^2}$ is appropriate.

1.1.6 Simple design rules

In Section 1.1.3 some hints are given for avoiding difficulty with vibration including control of natural frequencies, damping and stiffness. In addition other simple design rules may be considered.

a) Tuning method

First, all means possible should be taken to avoid a fundamental frequency in the range 1.6 to 2.4 Hz and, to a lesser extent, the higher range 3.5 to 4.5 Hz [SIA 160]. However, this may not easily be possible because, as we have seen, span is a major determinant of the fundamental natural frequency. Two other simple methods can be used as follows.

b) Code method

A simple and standard design procedure is that recommended in the British [BS 5400] and the Ontario [ONT 83] codes. The method determines the maximum vertical acceleration resulting from the passage of one pedestrian walking with a pace rate equal to the fundamental natural frequency of the bridge.

For footbridges up to 3 spans the value is:

$$a = 4\pi^2 \cdot f_1^2 \cdot y \cdot K \cdot \psi \text{ [m/s}^2\text{]} \tag{1.4}$$

where f_1 = fundamental natural frequency of the bridge [Hz]
y = static deflection at mid-span for a force of 700 N [m]
K = configuration factor
ψ = dynamic response factor

The configuration factor K is unity for a single span, 0.7 for a double span, and between 0.6 and 0.9 for a triple span. More details of K-values are given in the Ontario code than in the British code. The dynamic response factor ψ, which is not the same thing as the Dynamic Magnification Factor (DMF) of $1/(2\zeta)$ as described in Appendix A, is given in the graphical form reproduced in Figure 1.4.

The resulting value of a has to be compared with the tolerable values given in Section 1.1.5 for [BS 5400] and [ONT 83] respectively.

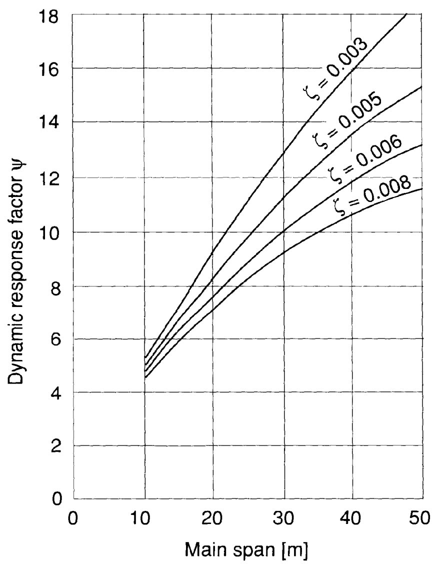


Figure 1.4: Dynamic response factor ψ as a function of span length and damping ratio ζ

c) Calculation of upper bound response for one pedestrian

A simple way of calculating an upper bound deflection is to use the information given in Appendix A for forced vibration. The static weight of the pedestrian and the central stiffness of the bridge are used to calculate the static deflection. This is then factored by α, the Fourier coefficient of the relevant harmonic of the walking rate (to be found in Appendix G). It is then further multiplied by the maximum Dynamic Magnification Factor (DMF) of $1/(2\zeta)$ as described in Appendix A. The value for ζ may be chosen from Table 1.1.

This procedure will give an overestimation of the response because it does not take into account the following two factors:

a) the limited effectiveness of the pedestrian when he is not at the span centre
b) the limited number of steps (limited time) taken in crossing the span.

d) Effects of several pedestrians

Some consideration is needed for the case of random arrival of pedestrians with a range of walking rates. If a Poisson distribution of arrivals is assumed, a magnification factor m can be derived equivalent to the square root of the number of persons on the bridge at any one time. This factor m is then applied to the response caused by a single pedestrian. There is no experimental confirmation of this result although some computer simulation studies have been made which support the theory.

1.1.7 More advanced design rules

A more detailed and rational calculation method for the response of footbridges is to be found in [1.3]. The formula to be used to calculate the peak acceleration resulting from the passage of one pedestrian is essentially the same as Equation 1.4 but with slight modification:

$$a = 4\pi^2 \cdot f_1^2 \cdot y \cdot \alpha \cdot \Phi \; [\mathrm{m/s^2}] \tag{1.5}$$

where y = static deflection at mid-span for a force of 700 N [m]
α = Fourier coefficient of the relevant harmonic of the walking or running rate (see Appendix G)
Φ = dynamic amplification factor for one pedestrian (Figure 1.5); takes account of the two factors a) and b) mentioned in Sub-Section 1.1.6c.

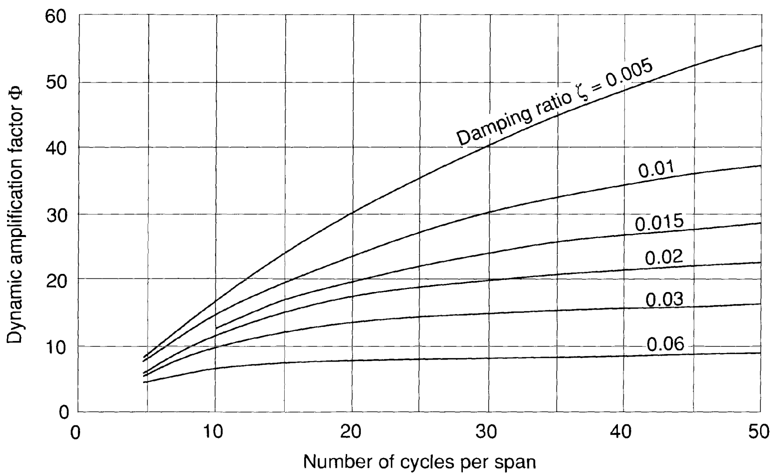


Figure 1.5: Dynamic amplification factor for resonant response due to sinusoidal force moving across simple span [1.3]

The calculated peak acceleration responses thus obtained for the case of walking ($\alpha = 0.4$ for the 1st harmonic) may then be compared with the value given in the last lines of Section 1.1.5.

This method has the advantage of permitting the introduction of actual measured Fourier coefficients of the forcing functions for walking as well as for running. In addition, bridge response to the second or even higher harmonics can be determined. In this case, the "number of cycles per span" in Figure 1.5 is the number of steps times the number of the harmonic considered. A typical length of footstep of 0.7 m may be assumed for walking (2 Hz), 1.2 m for running (2.5 Hz). As an example, for a 23 m span with $f_1 = 4.2$ Hz the number of cycles per span for the second harmonic of walking ($\alpha = 0.2$) is $2 \cdot 23 / 0.7 = 66$. Thus for $\zeta = 0.02$ becomes $\Phi = 24$.

1.1.8 Remedial measures

a) Stiffening

It has been shown that footbridges with a stiffness of greater than 8 kN/mm should not be at risk. Stiffening to this value could therefore be added as a retrofit to bridges which have a vibration problem. However, in most cases this will be a prohibitively expensive procedure. At the design stage it would be useful to aim at a minimum stiffness of 8 kN/mm.

b) Increased damping

Damping is the most economic and appropriate means of vibration control for footbridges which are found to have vibration problems. The Ontario code recommends the inclusion of devices to achieve this end. Damping or energy absorption can be added in a variety of ways. For example it can be incorporated in bearings and supports or a thick layer of high viscosity asphalt can be used for surfacing. Neither of these options is often very practicable.

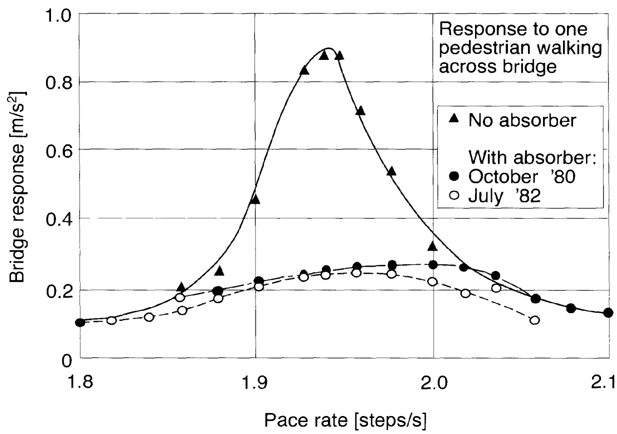


Figure 1.6: The effect of fitting a tuned absorber to a real footbridge

c) Vibration absorbers

Tuned vibration absorbers have been successfully used in several bridges. Briefly, the principle of operation is the addition of a subsidiary mass-spring-damper system whose natural frequency is nearly the same as that of the bridge (see also Appendix D). However, the subsidiary system is much smaller than the bridge itself, having a mass from 0.05% to 1% of that of the bridge. The approach is to use a small tuned vibration absorber with an optimised damper. This has the advantage of minimum cost and, because of small size, relative ease of fitting unobtrusively into an existing structure. However, it suffers from the disadvantage of relatively high amplitude motion when it is working most effectively. Not only must the design of the absorber allow for large motion but space must be accommodated in the bridge structure. Details of how to optimise the design of a damped vibration absorber are given in [1.4]. Figure 1.6 shows how effective a vibration absorber can be. In this case the absorber mass was 0.6% of the bridge mass.

1.2 Floors with walking people

J.H. Rainer, H. Bachmann, A.J. Pretlove

1.2.1 Problem description

Floors in office or apartment buildings are subject to the dynamic forces induced by people when they walk, and occasionally, run, jump or dance. The latter three apply especially when an office building contains facilities such as running tracks on roofs, exercise rooms, dance floors, or gymnasia, even if small in size. In corridors or on long floors, running could be contemplated, but this will likely occur only in isolated instances. Relevant dynamic loadings and frequency ranges for these activities are described in Sub-Chapter 1.3 on floors for sport and dance activities. This sub-chapter deals only with floor vibrations due to walking.

The nature of floor vibrations in office and residential buildings is influenced by many factors, among them the configurations of partitions, furnishings, ceiling structures, load concentrations and geometric shapes of floor area. These factors not only affect the mode shapes and natural frequencies of the floors, but also the damping. Rational calculations of vibration amplitudes induced by dynamic forces become rather complicated and uncertain. Consequently, empirical and semi-empirical methods have been developed to deal with this situation. Such a method is presented, beside the tuning method, as a practical design tool for certain cases until more reliable rational procedures become available.

1.2.2 Dynamic actions

For walking, dynamic forces are exerted with every footstep that a person takes. Besides describing the walking forces by their time variation, one can conveniently express the forcing function in terms of Fourier components having frequencies that are multiples of the walking rate. Most walking occurs at around 2 steps per second, but can range from about 1.6 to 2.4 Hz. (See Appendix G for further details). Most complaints have arisen from one person causing the vibrations. However, larger groups of people causing annoying vibrations cannot be excluded, especially when coordinated walking takes place.

1.2.3 Structural criteria

a) Natural frequencies

Floors in buildings can have large spans, especially in some "open offices", resulting in natural frequencies as low as 4 to 6 Hz. Corridors and pedestrian connections between stairways, escalators or elevators can also assume relatively low frequencies. Experience has shown that mainly composite structures (i.e. concrete slab on steel joists) with a natural frequency of less than about 7 to 8 Hz may be prone to annoying walking vibrations, more so than cast-in-place slab-and-beam concrete floors of the same frequency range (due to greater mass and higher damping). As a general rule, cast-in-place continuous concrete slabs are not prone to annoying walking vibrations since the natural frequencies of the slabs are usually well above 7.5 Hz (see Sub-Section 1.2.6a).

b) Damping

Equivalent viscous damping ratios of floors in buildings vary in a wide range, depending on ceiling, flooring, furniture, non-structural elements, partition walls, etc., and cannot be generally stated. For purposes of calculations using the heel impact method, damping ratios are suggested in Sub-Section 1.2.6b.

1.2.4 Effects

The effects on people are those of annoyance, apprehension as to the structural safety of the building, loss of mental concentration, and occasionally an unwell feeling resembling seasickness. None of these effects are considered "harmful" to people, but because of the annoyance factor, buildings with vibration problems may be subject to more complaints than others.

In most cases, the effects on the structure are not critical since the vibration amplitudes are well below the critical stress levels as far as strength and fatigue are concerned.

1.2.5 Tolerable values

The tolerable values are those that apply to the occupancy in the building. Experience has shown that an intermittent sustained (10-30 cycles) peak vibration amplitude of 0.5% g is acceptable for regular office occupancy. For special quiet locations, such as boardrooms, or areas where the occupants require a high degree of mental concentration, smaller limits (as little as 0.2% g) may have to be attained. The human perception limits in Appendix I can be applied, although some allowance should be made for the type of activity and the environment in which the activity takes place.

1.2.6 Simple design rules

Two simple design methods are suggested. Either method should produce satisfactory performance of floors to walking. Particular characteristics and limitations will be further explained for each method.

a) High tuning method

Since the annoying vibration amplitudes are caused by a coincidence of the natural frequency of the floor with the frequency of one of the harmonics of the walking forces, the problem can be avoided by keeping these frequencies away from each other, i.e. avoiding a near-resonance condition. For highly damped floors ($\zeta \geq \sim 5\%$) the lowest natural frequency of the floor should thus be above the frequency range of the second harmonic, i.e. above $2 \cdot 2.4$ Hz = 4.8 Hz. Thus, a natural frequency of 5 Hz or higher should be targetted. For floors with low damping ($\zeta \leq 5\%$) the lowest resonance frequency should be above the frequency of the third harmonic, i.e. $3 \cdot 2.4$ Hz = 7.2 Hz. To allow for some scatter in the accuracy of estimating parameters, a natural frequency of 7.5 Hz or higher should be targetted. Thus a remedial measure consistent with this procedure is to shift the natural frequency of the floor outside the offending frequency range, i.e. increase the natural frequency to a value of $\geq$ 7.5 Hz.

This high tuning method (see Appendix A) is simple and effective, but may be unnecessarily conservative since it does not take account of damping explicitly, nor of the effect of a large participating mass. Consequently, some floors that fall below the 7.5 Hz criterion can perform quite satisfactorily to walking excitation.

b) Heel impact method

One possible empirical method for avoiding annoying floor vibrations from walking in residential and office buildings is based on the "heel impact" and has been found to provide satisfactory performance for floors below about 10 Hz natural frequency and spans greater than about 8 m [CSA 1984]. The method is particularly suited to concrete slab steel joist systems, but has also been used for concrete slab and beam construction [1.5].

The heel impact method can also be used for evaluating shorter spans, up to about 5 m, but for short spans and light floor construction an allowance should be made for the mass of the impacting person as well as the person's contribution to the damping of the floor. This has, however, not been quantified yet and therefore for the straight-forward application of the heel impact method to concrete slab steel joist system, the recommended lower limit of applicability is a span of 8 m.

The heel impact consists of a person weighing about 700 N raising his heels high and suddenly dropping them to the floor. The procedure can be used both as a design tool and for evaluation of existing floors for acceptability against walking vibrations. It needs to be emphasized that the heel impact is a way of testing for adequacy for walking vibrations, and is not to be viewed as a simplified replacement to the continuous walking phenomenon.

The design or evaluation procedure consists of the following steps:

a) Calculate or measure the lowest natural frequency of the floor.
b) Calculate or measure initial peak acceleration from an impulse representing the heel impact.
c) Determine the aggregate damping value of the floor.
d) Compare heel impact criterion with values applicable to the floor properties.

Simply supported spans

For simply supported spans these steps will now be explained in more detail:

The lowest natural frequency of floors can be evaluated by any rational method. For concrete slabs supported by simply supported steel joists (see Figure 1.7), the natural frequency f can be calculated closely from:

$$f = 156 \cdot \left(\frac{EI_T}{WL^4}\right)^{0.5} \text{ [Hz]} \tag{1.6}$$

where E = modulus of elasticity [N/mm^2] (200,000 for steel)
I_T = moment of inertia of the transformed section [mm^4], where the concrete slab width is equal to the joist spacing s and the total thickness t_c is transformed to the area of steel.
W = total dead load per joist spacing [N/mm]
L = span [mm]

For floor systems where the beams or joists are supported on flexible girders (see Figure 1.7), f can be calculated from Dunkerley's approximation

$$f = (f_1^{-2} + f_2^{-2})^{-1/2} \quad [\mathrm{Hz}] \tag{1.7}$$

where f_1 is the natural frequency of the floor system on rigid supports, and f_2 is the natural frequency of the girder with a mass corresponding to the tributary floor area supported by the girder. For floor systems where the natural frequency f_2 of the girder is less than f_1 of the joist or beam system, for continuous systems, or for more complex configurations see Section 1.2.7.

An initial peak acceleration a_o can be derived from the impulse-response relationship which gives:

$$a_o = \frac{2\pi \cdot f \cdot I}{M} \cdot 0.9 \quad [\mathrm{m/s^2}] \tag{1.8}$$

where f = lowest natural frequency of floor [Hz]
I = impulse representing heel impact = 67 Ns
M = effective vibrating mass [kg]

The factor 0.9 is an adjustment factor to account for damped peak response. The mass M pertaining to the initial dynamic response is determined by assuming an isolated vibrating floor panel of width 40 t_c, where t_c is the total thickness of the concrete slab. For a vibrating shape of a sine wave the effective mass is 0.67 times the total mass of the floor panels having width B and length L. This then gives:

$$a_o = \frac{60 \cdot f}{w \cdot B \cdot L} \quad [\%\mathrm{g}] \tag{1.9}$$

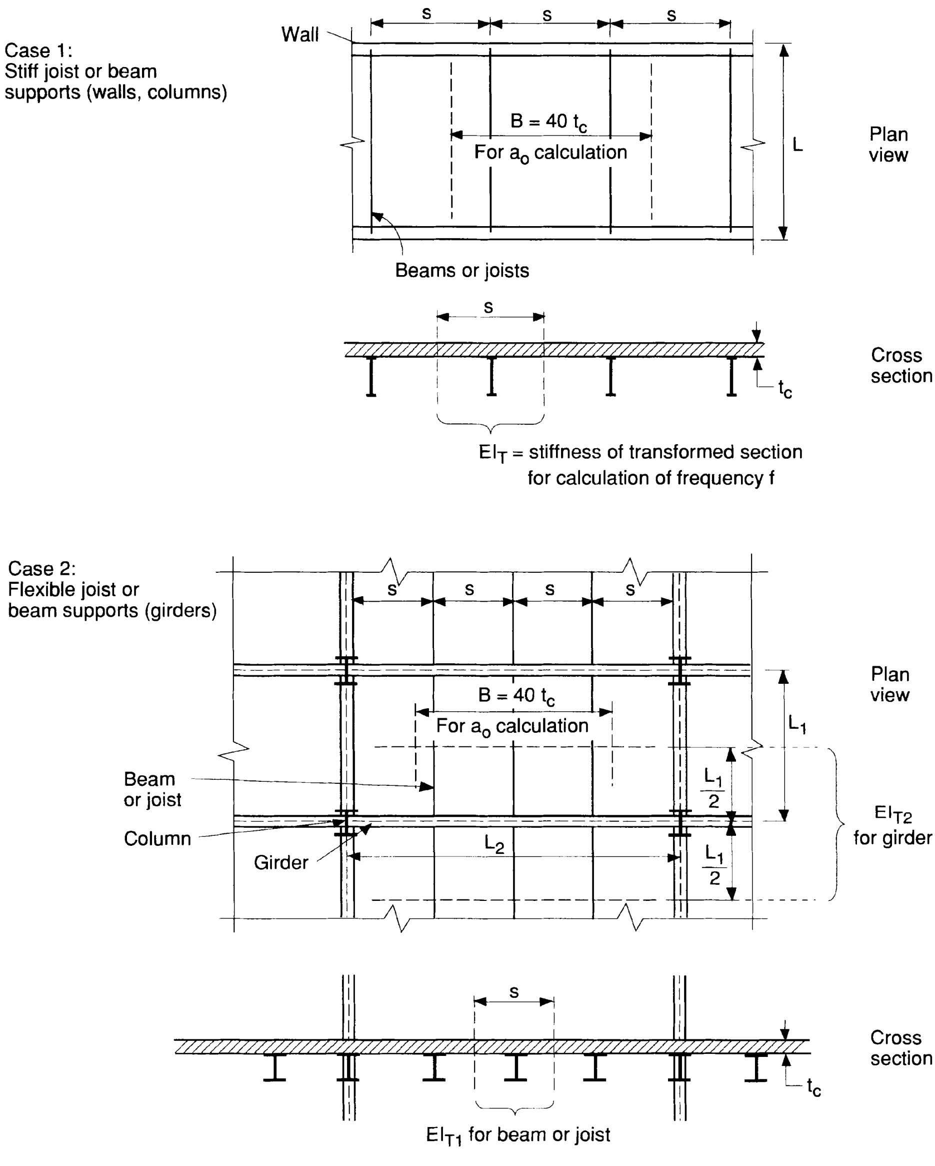
where w = weight of floor per unit area plus actual contents [kN/m^2]
B = $40 \cdot t_c$ = width of floor panel responding to heel impact [m]
L = length of floor panel [m]

Suggested damping ratios ζ, as a fraction of critical, associated with the heel impact criteria are as follows:

bare floor (fully composite)	0.03
finished floor (with ceiling, ducts, flooring, furniture)	0.06
finished floor with partitions	0.12

It should be noted that these damping ratios are obtained from the decay rate of the heel impact applied to the centre of the finished and occupied floor and are generally larger than those measured at barely perceptible amplitudes.

The criteria are given in Figure 1.8 for which the value of a_o should be less than the curves pertaining to the applicable damping ratio. These criteria are specifically applicable only to the heel impact calculation and the heel impact test method and were determined from experience [1.6]. The floor is deemed satisfactory for walking vibrations if the initial peak accel-



EI_{T1} = Stiffness of transformed beam or joist section
EI_{T2} = Stiffness of transformed section if girder and slab act integrally
f_1 = frequency of beam or joist system using EI_{T1}, L_1
f_2 = frequency of girder system using EI_{T2}, L_2
$f = (f_1^{-2} + f_2^{-2})^{-1/2}$

Figure 1.7: Geometric values for calculating the lowest natural frequency of floors with simply supported spans

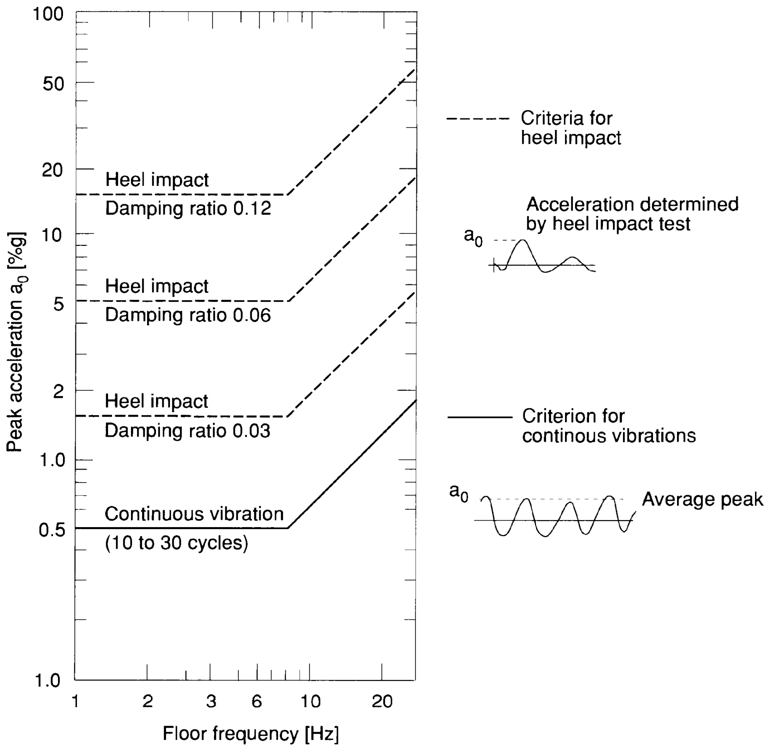


Figure 1.8: Suggested acceptability limits of floor vibrations due to walking in normal office building and residential occupancy, as determined by the heel impact criterion [CSA 84]

eration a_o plots below the respective criteria lines. If it falls above, the mass or the damping (or both) should be increased.

The results achieved by this empirical method will exhibit the characteristics that are inherent in this technique. The method will not produce a convergent answer for progressive increases in stiffness, but rather responds favourably to increases in mass and damping in order to satisfy the criterion. This is consistent with the principle that increases in natural frequency are not likely to be very helpful until the 7.5 Hz high tuning criterion is reached, where resonance is no longer of practical concern. Thus, remedial measures consistent with this procedure are increases in floor mass and/or damping until the heel impact criteria in Figure 1.8 are satisfied.

Other design procedures against walking vibrations have been developed, among others, by [1.7], [1.8], [1.9] and [1.10].

Continuous spans

For continuous spans, i.e. for floor systems that are structurally continuous across one or more supports such as walls, the effective vibrating mass M needs to be adjusted in the calculation of a_o. For two equal continuous spans, the mass M is double that of each one of the simple spans. Thus for calculating a_o with Equation (1.9) one should use $L = 2$ times the single span length.

Special care should be taken in evaluating the satisfactory vibration level of the span that is adjacent to the one on which the walking activity occurs. Because of the structural continuity, the vibrations will be transmitted to the adjacent span with nominally the same amplitudes as on the excited span. If the adjacent span supports a quiet occupancy or if the two floors are visually separated, then more stringent acceptability criteria should be considered.

For unequal spans or when the support is a flexible beam, more advanced design rules should be used (see Section 1.2.7).

1.2.7 More advanced design rules

More sophisticated design rules can be devised that are based on calculations of the dynamic response of floors subjected to the force variations produced by walkers or from other human activities. Due to a number of uncertainties such calculations are at present not carried out routinely. Nevertheless, an upper bound solution can be obtained by applying the applicable harmonic force component from walking to the equivalent single-degree-of-freedom system of the applicable mode of the floor and calculating its steady state response. This can then be compared with the perception or annoyance criteria, suitably chosen to reflect the type of receiver and the environment in which the vibrations are perceived.

Special attention needs to be paid to the transmission of vibrations from one bay to another in floor systems that are structurally continuous over supports.

When the critical point of vibration perception is not at the centre of a simply supported span, as is assumed in the above calculation, then the limiting acceleration can be adjusted by the ratio of the modal amplitudes at the point of observation to that at the centre of the span.

1.2.8 Remedial measures

a) Shift of the natural frequency

Shift the natural frequency away from the offending harmonic of walking forces, by changing the stiffness or the mass of the floor, and/or increase damping, especially if the offending frequency corresponds to that of the third harmonic of walking forces.

b) Non-structural elements

For floors being assessed by the heel impact criterion, increase the mass and/or increase the damping by adding various non-structural elements such as full-height partitions, or by adding damping devices.

Measurements of the vibrations on existing floors is recommended, both for understanding the nature of the problem and for the design of an appropriate remedial measure.

1.3 Floors for sport or dance activities

H. Bachmann, J.H. Rainer, A.J. Pretlove

1.3.1 Problem description

Floors treated in this sub-chapter are situated in buildings that contain

- Sport halls, gymnasia or other rooms for gymnastic training ("sport floors")
- Dance halls, concert halls without fixed seating or other community halls without fixed seating ("dance floors").

The causes of vibrations are

- in the case of sport floors: mostly rhythmical exercises such as jumping, skipping and running. These exercises are often carried out by numerous people to the beat of music and with a high degree of synchronisation (e.g. fitness training, jazz dance training, aerobics, etc.) and last up to 20 seconds or more. Such exercises may provide very high dynamic forces.
- in the case of dance floors: normal dancing such as march, waltz, tango, polka, schottisch, rock and roll, etc., or rhythmical handclapping of a standing audience often combined with simultaneous vertical body bouncing (in concerts).

1.3.2 Dynamic actions

The dynamic forces caused by sport activities or by dance activities may be very wide ranging, but there are some important common features. In most sport activities the ground contact of the feet is temporarily interrupted, resulting in high rhythmical impact forces (see Figure G.2). The observed maximum activity rate is about 3.5 Hz, and the density of people is relatively small. On the other hand, most dance activities are characterised by the fact that there is a continuous ground contact resulting in smaller forces which are comparable with those of brisk walking. "Wild" dances could, however, generate forces comparable to sport activities. The observed maximum pace (dance) rate is about 3.1 Hz and thus somewhat smaller than the maximum rate for sport activities. The density of dancers, however, can be much higher than the density of people performing sport activities.

For the dynamic design of a *sport floor,* the type of activity "jumping" defined in Tables G.1 and G.2 can be taken as representative. The relevant range of activity rate is 1.8 - 3.4 Hz. The Fourier coefficients shown in Table G.2 correspond to an impact factor of $k_p \cong 4.5$ (see Figure G.2), which is a good assumption for a fairly large group [1.11]. Table G.2 also gives the presumed density of people on sport floors for dynamic calculations.

For the dynamic design of a *dance floor,* the type of activity "dancing" defined in Tables G.1 and G.2 can be taken as representative. The relevant range of activity rate is 1.5 - 3.0 Hz. Table G.2 gives the relevant Fourier coefficients to be used, together with the density of people for dynamic calculations of dance floors. The case of rhythmical handclapping combined with vertical body bouncing of a standing audience is not relevant because of the small Fourier coefficients combined with the same or lower design density of people (see Table G.2).

1.3.3 Structural criteria

a) Natural frequencies

Long-span *sport floors* which have been designed only for static load often have a fundamental frequency of 4.5 to 5.5 Hz. Such floors can be excited to strong resonance vibrations especially by the frequency of the second harmonic of the forcing function ($2 \cdot 1.8$ Hz to $2 \cdot 3.4$ Hz gives 3.6 Hz to 6.8 Hz). Smaller-span sport floors with a fundamental frequency of 6 to 9 Hz can also be excited to resonance vibrations by the frequency of the third harmonic (e.g. $3 \cdot 2.0$ Hz or $3 \cdot 3.0$ Hz gives 6 Hz or 9 Hz respectively) and in extreme cases the fourth harmonic of the forcing function.

Dance floors can also be excited to resonance vibrations by the frequency of the second harmonic ($2 \cdot 2.0$ Hz to $2 \cdot 3.0$ Hz gives 4.0 Hz to 6.0 Hz) or by the frequency of the third harmonic (e.g. $3 \cdot 2.5$ Hz gives 7.5 Hz) and in extreme cases by the fourth harmonic of the forcing function .

b) Damping

In the case of sport floors or dance floors in gymnasia or relevant buildings, damping of the bare structure is augmented by considerable contact damping produced by nonstructural elements (see Appendix C). Table 1.2 shows equivalent viscous damping ratios ζ for different construction types.

	damping ratio ζ		
Construction type	min.	mean	max.
Reinforced concrete	0.014	0.025	0.035
Prestressed concrete	0.010	0.020	0.030
Composite	0.008	0.016	0.025
Steel	0.006	0.012	0.020

Table 1.2: Common values of damping ratio ζ for sport and dance floors

1.3.4 Effects

In the cases of sport floors and dance floors, people in adjoining rooms or in rooms directly under the affected floor can be annoyed by vibrations while people causing the vibrations do not feel them or are not disturbed by them. In the case of dance floors, people sitting at tables on the affected floor may also be annoyed. A similar case exists for a stage dance performance when there is a seated or standing audience in front of the stage. In extreme cases the vibrations may be severe enough to cause the affected persons to leave the room quickly; it may even lead to panic. Of great importance may be the fact that in addition to mechanical vibrations, acoustic effects çreated through reverberations and rattling of equipment as well as the impression of a moving ceiling and of moving walls can be very disturbing.

Vibrations from sport or dance activities can affect the serviceability of the relevant building by damaging non-structural elements such as a ceiling, cladding, windows, etc. Safety problems caused through fatigue or overstress in the load-bearing structure are rare. However, a safety problem for people may arise as a result of ceiling elements becoming detached due to vibrations and falling down.

1.3.5 Tolerable values

The maximum tolerable acceleration of sport floors (tolerance threshold) is about 5% g (at most 10% g) [1.11], [NBCC 90]. Stress in the load-bearing structure should be verified and compared with limit values from relevant codes or Appendix J.

The maximum acceleration of dance floors should also be limited to about 5% g. However, in cases where there is an atmosphere of euphoria, e.g. rock concerts, much higher accelerations up to 30% g have been observed without complaints or anyone leaving the floor. The tolerance level of 5% g should be reduced to 2% g when people are seated around the dancing surface [NBCC 90]. When seated persons are not in visual and auditory contact with the dancers or when adjacent rooms are used by a different clientele, then even more stringent criteria may be required by the receiver, down to vibration amplitudes near the threshold of perception (see Appendix I). However, such criteria may be difficult to achieve in practice.

1.3.6 Simple design rules

As a first criterion the fundamental natural frequency of sport floors and of dance floors should be higher than the upper bound frequency of the second harmonic of the representative normalised dynamic force. For *sport floors* the relevant activity is "jumping", for *dance floors* "dancing" (see Appendix G). This leads to a fundamental frequency of *sport floors* higher than $2 \cdot 3.4$ Hz = 6.8 Hz and of *dance floors* being higher than $2 \cdot 3.0$ Hz = 6.0 Hz. In addition, as a second criterion, taking into account the tremendous energy input created by large groups of athletes or dancers, different values of effective stiffness, mass and damping for various construction types should be approximately taken into account (e.g. steel structures are much more lively than concrete structures under the same dynamic action).

These criteria lead to the following recommendations for lower bounds of the fundamental frequency of the floors:

	Sport floors	Dance floors
Reinforced concrete structures	$f_1 > 7.5$ Hz	$f_1 > 6.5$ Hz
Prestressed concrete structures	$f_1 > 8.0$ Hz	$f_1 > 7.0$ Hz
Composite structures	$f_1 > 8.5$ Hz	$f_1 > 7.5$ Hz
Steel structures	$f_1 > 9.0$ Hz	$f_1 > 8.0$ Hz

Calculations of natural frequencies should always be carried out with careful thought being given to the structural contribution of floor finish, the dynamic modulus of elasticity, and - in reinforced concrete structures - the progressive cracking, including the tension stiffening effect of the concrete between the cracks. It is advisable to carry out sensitivity calculations by varying these parameters.

1.3.7 More advanced design rules

If more sophisticated considerations are desirable it is recommended that a calculation of a forced vibration for the representative normalized dynamic force be carried out and the results compared with tolerable values. This may be the case if the above mentioned recommendations for lower bounds of the fundamental frequencies of a floor cannot be adhered to, or if a claim for higher comfort is made.

The required data for the representative normalised dynamic force for "jumping" or "dancing" can be taken from Table G.2 (Fourier coefficients and phase angles, design density of people). The calculation is normally carried out for the steady state according to the rules of Appendix A of linear-elastic dynamic theory using bracketing assumptions for stiffness, mass and damping. The critical design case for vibration will usually occur when the frequency of the second or third harmonic of the forcing function is equal to the fundamental frequency of the floor resonance. The tolerable values can be taken from Section 1.3.5 or Appendix I.

1.3.8 Remedial measures

a) Raising the natural frequency by means of added stiffness

In many cases the most appropriate remedial measure for existing sport or dance floors is to increase the fundamental frequency by increasing the stiffness (but beware of the effect of added mass).

b) Increasing structural damping

An increase of structural damping is usually difficult to achieve. Some possibilities are described in [1.1].

c) Use of vibration absorbers

In exceptional cases the installation of a vibration absorber (mass-spring-damper, see Appendix D) tuned with respect to the critical frequency of the vibrating floor may be possible. However, to date, no successful applications are known to the authors.

1.4 Floors with fixed seating and spectator galleries

H. Bachmann, J.H. Rainer, A.J. Pretlove

1.4.1 Problem description

Structures treated in this sub-chapter are

- floors with fixed seating in concert halls and theatres
- spectator galleries in stadia, grandstands and theatres.

The sources of vibration may be

- rhythmical hand clapping of a seated audience to the beat of the music or demanding encores. This is quite common in "soft" pop-concerts, but it may also occur in classical concerts (e.g. when clapping to a piece like the Radetzky March by Strauss).
- rhythmical hand clapping with simultaneous vertical body bouncing of an audience standing between the fixed seat rows. Hand clapping with body bouncing may occur in "hard" pop-concerts while in classical concerts generally only hand clapping need be taken into account.
- rhythmical lateral body swaying of a seated or standing singing audience. This may occur when people are seated on a bench without armrests or when they are standing close to one another linked with the next person's arm.

1.4.2 Dynamic actions

The dynamic forces caused by the above mentioned activities can be manifold. The vertical dynamic force of a hand-clapping seated person depends mainly on the clapping intensity and whether or not a simultaneous shoulder movement occurs (see [G.5]). The dynamic force of a standing person is about the same as for a seated person, if the same kind of clapping and no simultaneous body bouncing is performed. Vertical body bouncing by bending and straightening the knees produces a much higher dynamic force than hand clapping alone. The horizontal dynamic force from rhythmical lateral body swaying depends mainly on the swaying frequency, the displacement amplitude and the participating mass of the human body.

For the dynamic design of a relevant structure for *vertical dynamic forces*, the types of activity "hand clapping while being seated" and "hand clapping with body bouncing while standing" defined in Table G.1 can be taken as representative. The relevant range of activity rate is 1.5 to 3.0 Hz. Fourier coefficients for some load frequencies are given in Table G.2 (note, however, the great scatter due to the large variety of possible rhythmical body motions).

For the dynamic design of a relevant structure for *horizontal dynamic forces*, the type of activity "lateral body swaying" (of a seated or standing singing audience) can be taken as representative. It must be recognised that slow rhythms with a frequency of the main beat of about 0.8 to 1.4 Hz may be relevant and that the rate of lateral body swaying is one half of the beat frequency, i.e. about 0.4 to 0.7 Hz. The Fourier coefficient for the relevant harmonic has not yet been determined by experiments, but it can be assumed to be 0.3 for an activity rate of 0.6 Hz (beat frequency of 1.2 Hz) for a standing person (remark: this value has been determined for a sinusoidal displacement amplitude of ± 200 mm at a sway frequency of 0.6 Hz as follows: $a = 200 \text{ mm} \cdot 4 \cdot \pi^2 \cdot 0.6^2/\text{s}^2 \cong 3 \text{ m/s}^2$) and two-thirds of that for a seated person.

1.4.3 Structural criteria

a) Natural frequencies

Long-span floors of concert halls or theatres or spectator gallery structures which have been designed only for static loads can show a *vertical vibration fundamental frequency* down to about 2 Hz. The damping ratio may lie between 0.015 and 0.03. Such floors can be excited to strong vertical resonance vibrations by the frequency of the first harmonic of the vertical forcing function of a seated handclapping audience (i.e. 1.5 Hz to 3.0 Hz). For floors with a higher fundamental frequency but a standing bouncing audience, the frequency of the second harmonic of the vertical forcing function (2 · 1.5 Hz to 2 · 3.0 Hz gives 3 to 6 Hz) can also cause disturbing vibrations.

Depending on horizontal stiffness, spectator gallery structures may have a *horizontal fundamental frequency* down to about 1 Hz. They can be excited to strong horizontal resonance vibrations by the frequency of the third harmonic of the horizontal swaying forcing function (3 · 0.4 Hz to 3 · 0.7 Hz gives 1.2 Hz to 2.1 Hz).

b) Damping

For floors in buildings with fixed seating the same damping ratios ζ as for sport and dance floors given in Table 1.2 are applicable. For spectator galleries with fewer non-structural elements about two-thirds of these values may be appropriate.

1.4.4 Effects

The effects on people are generally similar to those described in Sub-Chapter 1.3. In the case of a horizontal sway of a soft structure with a fundamental frequency equal to the beat frequency, the possibility of panic cannot be excluded.

1.4.5 Tolerable values

In general, for concert halls and theatres with classical concerts and a hand clapping audience, a tolerable acceleration of ~ 1% g maximum sustained vertical peak acceleration may be taken as acceptable. For pop-concerts this threshold may be increased to 5% g (see also Sub-Chapter 1.3).

For horizontally swaying spectator galleries a tolerable horizontal acceleration of about half of the tolerable vertical maximum sustained peak acceleration may be taken as acceptable. In addition there may be displacement limits for specific structures.

1.4.6 Simple design rules

The *vertical fundamental frequency* of the structure should be established with respect to the following criteria (present day knowledge):

- floors of concert halls and of theatres with fixed seating with classical concerts or "soft" pop-concerts only: higher than the upper bound frequency of the first harmonic of the representative normalised dynamic force for the activity "hand clapping" (see Appendix G). This leads to fundamental frequencies of such floors being higher than $1 \cdot 3.0$ Hz = 3.0 Hz.
- floors of concert halls and theatres with fixed seating and spectator gallery structures with "hard" pop-concerts: higher than the upper bound frequency of the second harmonic of the representative normalised dynamic force for the activity "hand-clapping with body bouncing while standing" (see Appendix G). This leads to fundamental frequencies of such structures higher than $2 \cdot 3.0$ Hz = 6.0 Hz.

The *horizontal fundamental frequency* of a spectator gallery structure should be established with respect to the upper bound frequency of the third harmonic of the representative normalised dynamic force for the activity "lateral body swaying of a seated or standing audience" (see Appendix G). This leads to horizontal fundamental frequencies of such structures higher than $3 \cdot 0.7$ Hz = 2.1 Hz.

These criteria lead to the following recommendations for lower bounds of the fundamental frequency:

floors of concert halls and of theatres with fixed seating with classical concerts or "soft" pop-concerts only	$f_1 > 3.4$ Hz
floors of concert halls and of theatres with fixed seating and spectator gallery structures with "hard" pop-concerts	$f_1 > 6.5$ Hz
spectator gallery structures with fixed seating and lateral swaying and singing audience	$f_{1horiz} > 2.5$ Hz

Calculations of natural frequencies should always be carried out with careful thought being given to the structural contribution of floor finish, the dynamic modulus of elasticity, and - in reinforced concrete structures - the progressive cracking including the tension stiffening effect of the concrete between the cracks. It is advisable to carry out sensitivity calculations by varying these parameters.

1.4.7 More advanced design rules

Section 1.3.7 is applicable.

1.4.8 Remedial measures

Section 1.3.8 is applicable.

1.5 High-diving platforms

H. Bachmann, A.J. Pretlove, J.H. Rainer

1.5.1 Problem description

High-diving platforms in open air or indoor swimming pools can be affected by vibrations if the platforms have not been designed for dynamic forces [1.12]. The vibrations are mainly caused by the athlete through impulsive action immediately before or at take-off.

1.5.2 Dynamic actions

A major distinction has to be made as to whether or not a springboard for figure diving is mounted on the platform.

Normal high diving is done from a rigid concrete platform slab with or without a running start. For jumping off without running, the dynamic force consists of a single impulse. Jumping off after running activates additional impulses.

For figure diving a relatively soft springboard is mounted on the platform. The springboard flexibility results in larger amplitudes attained by the centre of mass of the athlete's body when he or she gains momentum by jumping on the spot or by running. Compared with a rigid platform slab, impulses on the springboard are significantly larger.

For design purposes the forcing function need not be known. A simple distinction between a platform with a rigid slab and a platform with mounted springboard is sufficient.

1.5.3 Structural criteria

a) Natural frequencies

High-diving platforms designed only for static loads often have natural frequencies between 2 and 3 Hz [1.12], [1.11]. Athletes can excite such platforms to excessive vibrations.

The following vibration patterns are possible:

- swaying of the support column (in a direction not necessarily coincident with the take-off direction)
- rigid-body motion of the platform (similar effects)
- vibration of the platform slab (particularly unpleasant as the athlete may be given an unwanted spin).

b) Damping

In the case of high-diving platform structures, material damping of the structure itself can be augmented by considerable energy radiation to the soil (see Appendix C). This may be true when the foundation of the platform structure stands on rather soft soil and can rotate, (i.e. it is not connected to a basin or its foundation) or when the shaft above the foundation is embedded over a certain length of soil. Then the vibration deformation of the structure can lead to considerable energy radiation into the soil. However, most high-diving platform structures have a relatively low equivalent viscous damping ratio ζ as shown in Table 1.3.

	damping ratio ζ		
Construction type	min.	mean	max.
Reinforced concrete (~ uncracked or only few cracks)	0.008	0.012	0.016

Table 1.3: Common values of damping ratio ζ for high-diving platform structures

1.5.4 Effects

Strong support column vibrations irritate the athlete and hamper his or her performance. In extreme cases, the high-diving platform has to be strengthened or totally rebuilt [1.11].

Vibrations mainly affect the serviceability of the platform. Problems of fatigue or impending failure are rarely relevant.

1.5.5 Tolerable values

The definition of upper limits of velocities or accelerations is impractical as the vibrations are transient and their direction is also of significance. It has been found more useful to comply with certain frequency and stiffness criteria (see paragraph below).

1.5.6 Simple design rules

A high-diving platform must fulfill the following two types of criteria [1.12]:

- stiffness criteria to be checked with relatively simple static calculations
- frequency criteria corresponding to high tuning of the structure and therefore requiring a frequency computation.

a) Stiffness criteria

The spatial vector displacement of the front edge of the platform caused by spatial static force with $2F_x = F_y = F_z = 1$ kN according to Figure 1.9 must remain

$$\delta(F_x, F_y, F_z) \leq 1 \text{ mm} \tag{1.10}$$

and the lateral front displacement alone must be

$$\delta_x = 0.5 \cdot \delta \leq 0.5 \text{ mm}. \tag{1.11}$$

The stiffness criteria are particularly stringent. Practical experience shows that reinforced-concrete platforms can be assumed to maintain their uncracked stiffness in bending as well as torsion. The listed bounds were derived from approved high-performance platforms and represent competition standards for normal high diving and for figure diving. For less professional demands in recreational indoor or outdoor swimming facilities, these bounds could well be relaxed.

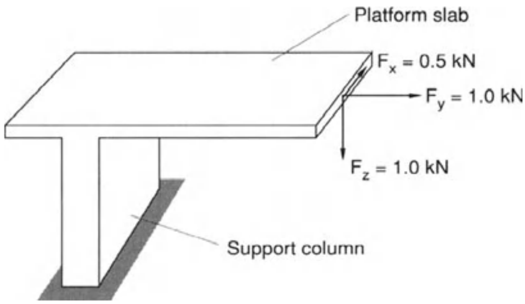


Figure 1.9: High-diving platform with spatial static load [1.12]

b) Frequency criteria

The frequency bounds to be observed are listed in Table 1.4. They concern support column sway, rigid-body motion and platform slab vibration. A major distinction is to be made when a springboard for figure diving is mounted on the platform. As described before, rhythmic jumping on the springboard contributes much to the excitation of the platform, so that stricter frequency bounds apply.

Frequency bounds	without spring board	with spring board
Support column vibrations (all fundamental modes in longitudinal and lateral sway and in twist)	$f_1 \geq 3.5$ Hz	$f_1 \geq 5.0$ Hz
Rigid-body vibration (flexibility of foundation)	$f_1 \geq 7.0$ Hz	$f_1 \geq 10.0$ Hz
Slab vibration	$f_1 \geq 10.0$ Hz	$f_1 \geq 10.0$ Hz

Table 1.4: Recommended minimum frequencies for high-diving platforms in swimming pools

1.5.7 More advanced design rules

No design rules can be recommended other than those given above.

1.5.8 Remedial measures

Inadequate high-diving platforms can be strengthened with the objective of attaining the frequency and stiffness criteria given in Section 1.5.6. In some cases a tuned vibration absorber can be installed [1.13]. An increase of inherent damping or other measures for improvement are generally difficult to put into practice.

References to Chapter 1

[1.1] Tilly G.P., Cullington D.W., Eyre R.: "Dynamic Behaviour of Footbridges". Surveys S-26/84 of the International Association of Bridge and Structural Engineering (IABSE), 1984.

[1.2] Wiss J.F., Parmelee R.A.: "Human Perception of Transient Vibrations". Proceedings of the American Society of Civil Engineers (A.S.C.E.), 100, ST4, 773, 1974.

[1.3] Rainer J.H., Pernica G., Allen D.E.: "Dynamic Loading and Response of Footbridges". Canadian Journal of Civil Engineering, 15(1), 66, 1988.

[1.4] Jones R.T., Pretlove A.J., Eyre R.: "Two Case Studies in the Use of Tuned Vibration Absorbers on Footbridges". The Structural Engineer, 59B, 27, 1981.

[1.5] Allen D.E., Rainer J.H. and Pernica G.: "Vibration Criteria for Long-Span Concrete Floors". In "Vibrations of Concrete Structures", Special Publication SP-60, p. 67-78, American Concrete Institute, Detroit, Michigan, 1979.

[1.6] Allen D.E. and Rainer J.H.: "Vibration Criteria for Long-Span Floors", Canadian Journal of Civil Engineering, 3, (2) June 1976, p.165-173, 1976.

[1.7] Murray T.M.: "Acceptability Criterion for Occupancy-Induced Floor Vibrations". Engineering Journal, American Institute for Steel Construction, 18(2), 1981, p. 62-70, 1981.

[1.8] Ohlsson S.: "Springiness and Human Induced Floor Vibration - A Design Guide". Document D12:1988, Swedish Council for Building Research, Stockholm, 1988.

[1.9] Wyatt T.A.: "Design Guide on the Vibration of Floors". Publication 076, The Steel Construction Institute (SCI), Sunningdale, Berkshire U.K., 1989.

[1.10] Allen D.E., Murray T.M.: "Design Criterion for Vibrations Due to Walking". Engineering Journal, American Institute of Steel Construction (AISC), Vol. 30, No. 4, 1993, pp. 117 - 129.

[1.11] Bachmann H., Ammann W.: "Vibrations in structures - Induced by Man and Machines". Structural Engineering Documents No.3e, International Association for Bridge and Structural Engineering (IABSE), Zürich, 1987.

[1.12] Mayer H: "Schwingungsverhalten von Sprungtürmen in Frei- und Hallenbädern" ("Vibrational Behaviour of High-Diving Platforms in Outdoor and Indoor Swimming Pools"). Jahresbericht der Höheren Technischen Lehranstalt (HTL), Brugg-Windisch, Schweiz, 1970/1971.

[1.13] Bachmann H.: "Beruhigung eines Sprungturms durch einen Horizontaltilger" ("Vibration upgrading of a High-Diving Platform by means of a Horizontally Acting Tuned Vibration Absorber"). Schweizer Ingenieur und Architekt Nr. 21/1994, Zürich , 1994.

[1.14] Tilly G.P. (ed.): "Dynamic Behaviour of Concrete Structures". RILEM 65 MDB Committee, Elsevier, 1986.

2 Machinery-induced vibrations

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This chapter deals with structural vibrations induced by machinery equipment permanently fixed in place.

In this context permanently fixed equipment means all machinery, components or installations working continuously and causing vibrations. This applies equally to bells, especially when mounted in bell towers, and as such they are treated in this chapter. On the other hand, construction equipment is not dealt with in this chapter even though the causes of vibrations are similar. This type of equipment is dealt with in Chapter 4.

Direct dynamic effects, in situ, are of great importance. These consist not only of effects on equipment and people in the immediate vicinity but also of effects on the structure to which the machinery is attached as well as the foundation it stands on.

Besides the direct dynamic effects, permanently fixed equipment can have indirect dynamic effects. Such effects can stem from the transmission of vibrations by propagating waves leading to structure-borne acoustic waves. This kind of indirect sound is often particularly unpleasant. The sound is caused by vibrations travelling long distances via various transmitting media connected to the structure and in turn is then radiated from structural elements as airborne sound.

Other indirect dynamic effects can be vibrations transmitted through foundations into other buildings (and people living in them). They can be very disturbing. Vibrations transmitted through the air into other buildings are usually negligible.

This chapter on machinery-induced vibrations is structured into the following sub-chapters:

2.1 Machine foundations and supports
2.2 Bell towers
2.3 Structure-borne sound
2.4 Ground-transmitted vibrations

Important fundamentals are given in the appendices.

2.1 Machine foundations and supports

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2.1.1 Problem description

Machinery can affect many different parts of civil engineering structures such as foundations, pedestals or structural members (slabs and beams), and even whole buildings in several ways with quite different types of dynamic forces. In the following context the term *"foundation"* refers to engineered structures supporting all kinds of machines and resting directly on the soil, whereas the term *"support"* covers all other structures and structural members supporting machines.

A machine causes distinct dynamic forces depending on its manufacturing purpose, conditions of operation, state of maintenance, design details, etc. These forces depend primarily on the type of motion the machine parts describe, whether it is of a rotating, oscillating or an impacting nature. According to their time function, dynamic forces from within machines can be *periodic or non-periodic* (see Figure 2.1). A periodic excitation may sometimes be harmonic. A non-periodic excitation can either be of a transient or of an impulsive nature. As a first approximation it is sometimes possible to model repeated impulsive excitations as quasi-periodic. In some cases the time function of an excitation may not be sufficiently described by a deterministic mathematical approach and therefore may be more effectively described by a stochastic approach.

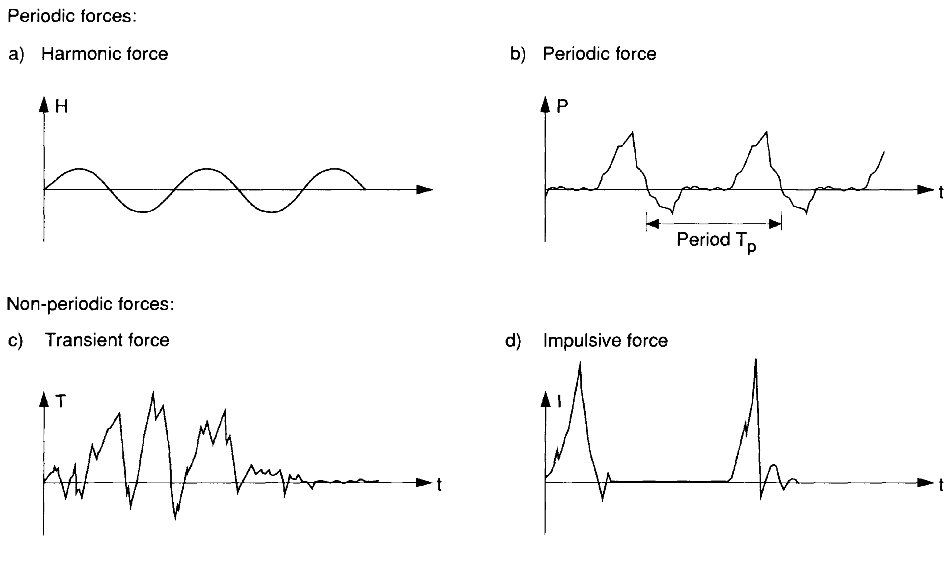


Figure 2.1: Typical time functions of dynamic forces

Machine foundations and supports can be as varied as the kind of dynamic forces. Machine foundations can mainly be divided into block, box and framed foundations. Either direct mounting or soft supports for machines on foundations is possible. Furthermore, there are many possibilities for machines, especially for small and medium sized machines, to be mounted without any additional measures on the structural members of the building, e.g. mounting a machine on a floor of a building. In this case, supports may be rigid or elastic. Sometimes it is advisable to consider an additional, stabilizing mass. Waves induced by machines may be transmitted into neighbouring buildings or adjacent rooms in the form of vibrations and special attention has to be paid to these problems.

2.1.2 Dynamic actions

a) Causes

Rotating parts of machinery cause non-negligible dynamic forces if they are insufficiently balanced or if electrodynamic fields are present. Out-of-balance forces arise whenever the centre of mass of a rotating part does not coincide with the axis of rotation. The product of mass and eccentricity is called static unbalance. The resulting dynamic load depends on the flexural rigidity of the axle and its support. Unbalance is usually more noticeable in machines that have been in operation for a considerable time. Examples of machines with predominantly rotating parts are fans and ventilators, centrifugal separators, vibrators, washing machines, lathes, centrifugal pumps, rotary presses, turbines and generators.

Oscillating parts of machines always excite dynamic forces. The causative motion can be translational, rotational with small angle, or a combination of both (pendular motion). Furthermore, dynamic forces depend on the state of maintenance and age of the machine. Examples of machines with predominantly oscillating parts are weaving machines, piston engines, reciprocating compressors, reciprocating pumps, emergency power generators (diesel engines), flat-bed printing presses, frame saws, crushing and screening machines.

Impacting parts of machines often develop large intermittent dynamic forces. Skilful design, however, will attempt to balance (e.g. with a counter-blow hammer) the major part of the force within the machine frame. This reduces the residual forces on the structure. Examples of machines with impacting parts are, for instance, moulding presses, punching machines, power hammers and forging hammers.

Apart from the types of motion the machine parts describe when in use, dynamic forces are also created by start-up and shut-down operation of the machine, through short circuiting and parts of machinery failing.

Generally it is of great advantage if the manufacturer can give details of the various types of dynamic forces caused by machines in use and probable vibration limits (for safe operation, threshold, etc.). If this is not possible the resulting dynamic forces have to be assumed based either on experience with similar machines or on acceptable vibration limits causing no damage to the machines and/or the personnel. Assumed time functions for accelerations or velocities of the exciting forces or energy data may also be used as criteria for assessing vibrations.

As mentioned above, the force time function can be periodic, transient or in exceptional cases even harmonic. In any case a periodic force can be decomposed by means of a Fourier analysis into a number of harmonic components (see Appendix A). Some forces can only be described in a stochastic way.

b) Periodic excitation

Periodic excitations are mainly the result of either rotating or oscillating parts of machinery. The resulting force time functions are briefly described:

Machines with rotating parts may cause non-negligible dynamic forces if they are insufficiently balanced. The centrifugal force depends on the square of the rate of rotation. This kind of excitation is referred to as *"quadratic excitation"*. A constant rate of rotation equals a constant amplitude of the force, often referred to as *"constant-force excitation"*.

The amplitude of a single rotating part out-of-balance is the typical case of a *quadratic excitation* and leads to:

$$F_o = m' \cdot e \cdot \frac{4\pi^2}{3600} \cdot n^2 = m' \cdot e \cdot 4\pi^2 \cdot f_B^2 = m' \cdot e \cdot \Omega^2 \tag{2.1}$$

where F_o = centrifugal force [N]
m' = mass of the rotating part (unbalanced fraction) [kg]
e = eccentricity of the unbalanced mass fraction [m]
n = rate of rotation (speed of revolution of the unbalanced mass expressed in revolutions per minute) [r.p.m.]
f_B = operating frequency ($f_B = n/60$) [Hz]
Ω = angular velocity of the rotating part (= $2\pi \cdot f_B$) [rad/s]

In an arbitrary direction the harmonic force may thus be defined as

$$F(t) = F_o \cdot \sin(\Omega t) = m' \cdot e \cdot \Omega^2 \cdot \sin(\Omega t) \tag{2.2}$$

which acts on the total mass M of the machine (including the mass m' of the rotating, unbalanced fraction). Detailed information on machinery with regard to data on unbalanced parts is given in [ISO 2372] or [ISO 3945].

If *several rotating parts* with individual unbalances are mounted on the same shaft, they rotate with identical speed of revolution but different phase angle, and hence they produce a resulting harmonic force.

If *several (n) unbalanced parts* rotate with different rates of rotation, a periodic force results due to the superposition of their individual harmonic forces in a state of unbalance. Thus

$$F(t) = \sum_{i=1}^{n} A_i \cdot \sin(2\pi i \cdot f_B \cdot t - \phi_i) \tag{2.3}$$

where i = order of the harmonic
A_i = amplitude of i-th harmonic
ϕ_i = phase lag of the i-th harmonic relative to the 1st harmonic.

The forcing function of *constant-force excitation* is of the form

$$F(t) = F'_o \cdot \sin(\Omega t) \tag{2.4}$$

where F'_o = amplitude of the force (constant)

Machines with oscillating parts always excite periodic forces in the direction of the moving parts. Although all reciprocating machines or engines exert primarily an oscillating force in the direction of the piston motion, they also give rise to a rotational component due to the eccentric hinging of the connecting rod to the crankshaft. Either component is of a quadratic excitation type. At constant operating frequency, however, the resulting amplitude depends on the number of pistons and of their suitable arrangement with respect to each other which may compensate the resulting forces or at least minimize them. The dynamic forces have to be defined by the manufacturer (see Section 2.1.6).

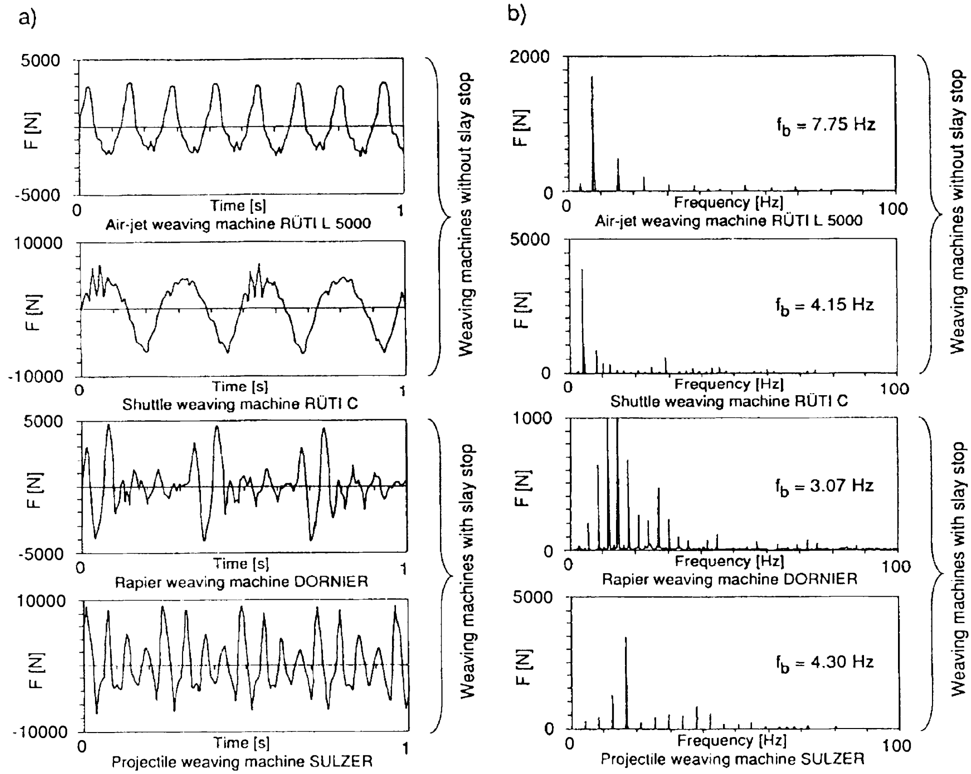


Figure 2.2: Vertical forces from various types of weaving machines with predominantly oscillating parts; a) force-time function, b) derived discrete Fourier amplitude spectrum [2.1]

Typical force-time functions caused by weaving machines, i.e. from machines with oscillating parts, and transmitted to their supports, are shown in Figure 2.2. This figure shows the time function of the vertical force transmitted to the footings and the corresponding spectra of Fourier amplitudes applicable for various types of machinery. Only in air-jet machines and shuttle looms do the maxima of the transmitted forces coincide with the operating frequency; on all other types the maximum occurs at a frequency of a higher harmonic.

c) Transient excitation

Machines with impacting parts often develop large intermittent dynamic forces. The loading can be characterized by the following parameters of the impact phase (see Figure 2.3):

- duration of impact t_p, compared to the first natural period of the oscillator T_1
- momentum I
- rise time of the force t_a
- peak impact force $F_{p,max}$.

The combination of these parameters determines the shape of the force-time function, for instance semi-sinusoidal, triangular or even rectangular. In most cases the maximum of a dynamic quantity resulting from the impact is of primary interest, e.g. the peak displacement x_{max} of the centre of mass of the affected structural member. Assuming an SDOF oscillator, for various shapes of force-time functions Figure 2.3 shows the dynamic magnification factor V_s versus the ratio t_p/T_1 (duration of impact to the natural period of the oscillator).

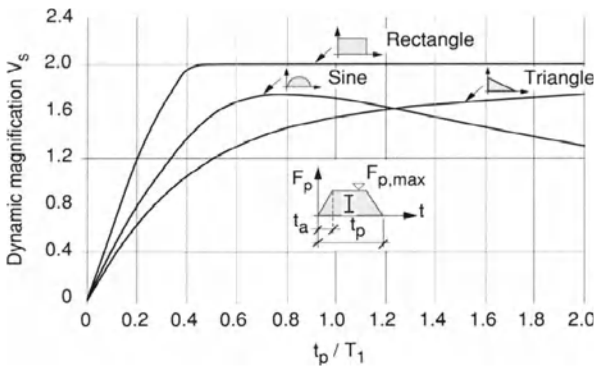


Figure 2.3: Dynamic magnification for various forcing functions as a function of the ratio between duration of impact and structural period [2.2]

The dynamic magnification factor for the displacement is defined as:

$$V_s = \frac{x_{max}}{F_{p,max}/k} \tag{2.5}$$

where V_s = dynamic magnification factor
$F_{p,max}$ = peak impact force
k = stiffness of the impacted structural member
$F_{p,max}/k$ = static displacement under the peak impact force

Figure 2.3 leads to the following conclusions:

1. In the range $t_p/T_1 > 1$, the magnification factor V_s is dominated by the rise time t_a. Instantaneously applied forces induce the highest possible value $V_s = 2.0$, whereas gradually applied forces lead to a lower bound of the magnification factor of $V_s = 1.0$.
2. In the range $t_p/T_1 < 0.40$, the magnification factor V_s depends strongly on the shape of the force-time function, but the peak displacement is largely determined by the applied momentum I. The relationship can be approximated by

$$x(t') = \frac{1}{M \cdot \omega_1} \cdot I \cdot \sin(\omega_1 \cdot t') \tag{2.6}$$

where M = mass of the equivalent SDOF oscillator (structural member)
ω_1 = circular natural frequency of the equivalent SDOF oscillator
$I = \int_o^{t_p} F(t) \cdot dt$ = applied momentum
$t' = t - t_p$

and for the peak displacement

$$x_{max} = \frac{1}{M \cdot \omega_1} \cdot I \tag{2.7}$$

Because of the typically short rise time t_a and impact duration t_p, significant force components are present over a wide frequency range. Structural damping does not have much effect on the peak displacement under these forcing conditions. More details on the force-time functions can be found, e.g. in [2.3] or [2.4].

For power and forging hammers the force-time function depends not only on the operating mode of the machine but also on the moulding properties of the processed material. One can distinguish the forcing phase (duration of impact) from the subsequent, sometimes much longer, phase without external excitation (quiet phase). In general, this phase of vibration decay is not exactly of constant duration, hence the nature of the forces is transient (single impulses). There are some machines on the market, however, which do have a constant quiet phase, resulting in a periodic type of excitation. The resulting dynamic forces exhibit a periodic peak followed by a more or less free vibration decay of the impacted body. As the operating frequencies of these machines can be rather high, complete vibration decay between impacts may not always be possible, especially in cases where structural damping is minimal.

d) Stochastic excitation

In many cases as, for instance, in coal or clinker mills it is not possible to describe the force-time function deterministically as it depends on random events. It is then necessary to use stochastics with characteristic parameters determined statistically (see e.g. [2.5], [2.38]).

Due to the large amount of uncertainty concerning the force-time function, machines of the impacting type should only be mounted on vibration isolating supports (spring-damper elements).

2.1.3 Structural criteria

a) Natural frequencies

A coincidence between the fundamental frequency of a structure or structural member and of the operating frequency of a machine must be avoided if possible. Due to the large variety and ranges (2 decades or more) of excitation-frequencies together with the large variety of structures and substructures concerned (building elements, type of support, stabilizing mass) it is not possible to define a common range of adequate structural natural frequencies. However, even for the case of a relatively rigid structure, the fundamental frequency will probably not exceed 25-30 Hz.

b) Damping

Typical equivalent viscous damping ratios ζ for machine-supporting floors of industrial buildings are given in Table 2.1.

	damping ratio ζ		
Construction type	min.	mean	max.
Reinforced concrete	0.010	0.017	0.025
Prestressed concrete	0.007	0.013	0.020
Composite structures	0.004	0.007	0.012
Steel	0.003	0.005	0.008

Table 2.1: Common values of damping ratio ζ for machine-supporting floors of industrial buildings

2.1.4 Effects

Effects of machine-induced vibrations can be of a wide variety, the basic distinction being drawn between effects on structures or structural members on the one hand, and those on people, installations, machinery and their products on the other.

a) Effects on structures

Effects of machine-induced vibrations on structures may include:

- appearance of cracking, crumbling plaster, loosening of screws etc.
- problems of fatigue of steel girders or steel reinforcement with consequent damage, ultimately leading to collapse
- loss of load-bearing capacity (in rare cases of overstressing).

b) Effects on people

People working temporarily or permanently near to machines emitting vibrations or near to co-vibrating structural members could be affected in various degrees. The intensity may range from barely noticeable to slightly or severely disturbing to harmful. They can occur in three different ways:

- as mechanical effects (e.g. vibration of a floor or ceiling)
- as acoustic effects (e.g. noise from reverberating installations and pieces of equipment, also structure-borne or air-borne sound)
- optical effects (e.g. visible motion of building elements, installations or objects).

c) Effects on machinery and installations

These include:

- problems of material behaviour in the machine itself (deformations, strength, fatigue)
- problems of material technology in the manufactured goods (e.g. excessive tolerances due to unplanned motion of tools and installations).

d) Effects due to structure-borne sound

See Sub-Chapter 2.3.

2.1.5 Tolerable values

a) General Aspects

The above mentioned effects lead to the following criteria:

- structural criteria
- physiological criteria
- production-quality criteria.

Vibration bounds may be given as physical quantities such as

- displacement amplitude
- velocity amplitude
- acceleration amplitude

or otherwise derived quantities including natural frequency (e.g. KB intensity of a structure or an affected structural member, see [DIN 4150] and also Appendices I and J).

b) Structural criteria

Vibrations induced by machines may cause deformations and smaller or larger forms of distress in buildings and structural members (see also Appendix J). Non-structural elements are particularly vulnerable. Continuous vibration can also lead to problems of fatigue and overstress in principal load-bearing members. However damage to non-structural elements is in most cases predominant. Damage patterns may be:

- cracking of walls and slabs
- aggravation of existing cracking in structural members and non-structural elements, which can lead to secondary damage such as leakage, corrosion etc.
- continued vibration causing subsidence of buildings leading to cracking as a secondary effect.
- collapse of equipment or cladding, thereby endangering occupants.

Various codes and standards contain data for acceptance criteria and tolerable values.

Detailed data are to be found in the following codes and standards: [ISO/DIS 4866], [DIN 4150/3], [SNV 640312], [VDI 2056], [VDI 2062], (as an example see also Figure 2.4) .

When new machinery with dynamic forces in a non-negligible range is to be integrated into existing buildings or mounted nearby and probably exerting non-negligible dynamic forces it is advisable to carry out vibration measurements at the planning stage to get more insight into the expected behaviour of the existing structure due to the anticipated additional excitations.

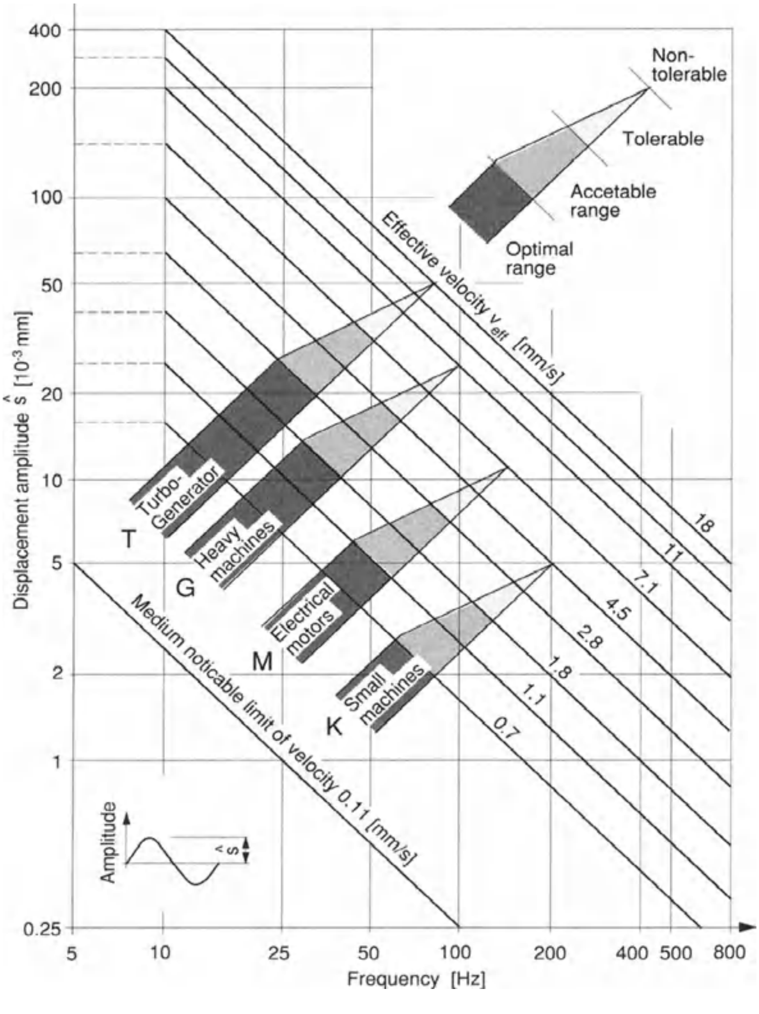


Figure 2.4: Effective velocity levels for operation of different machines (according to [VDI 2056])

c) Physiological criteria

Parameters influencing the human sensitivity are listed in detail in Appendix I. Additional and more specific data can also be found in the following codes and standards: [ISO 2631], [BRE], [BS 5400], [BS 6472], [DIN 4150/2], [NBCC 90], [VDI 2057].

d) Production-quality criteria

Universally applicable criteria cannot be given but need to be specified individually for different types of machinery. The trend for the future, however, shows a definite development towards establishing general regulations. Instructions given by manufacturers are important. At the planning stage, bounds to vibration effects need to be formulated with respect to the following questions:

1. Does the installation of new machines into the existing structure have any negative effects on the machinery itself or on the immediate neighbourhood?
2. Will secondary vibration of the existing structure cause material-technological problems to newly installed machinery?

General statements try to group various types of machinery into different sensitivity classes. More detailed information can be found in the following codes, standards and recommendations: [ISO 2372], [ISO 2373], [BS CP 2012/1], [DIN 4024/1], [DIN 4024/2], [VDI 2056] (see Figure 2.4), [VDI 2063].

e) Tolerable values relative to structure-borne sound

Some indications for tolerable values relative to structure-borne sound be found in [DIN IEC 651], [2.15] and [2.16].

2.1.6 Simple design rules

a) General

In addition to providing support for the static forces, the main task of machine foundations and supports is the safe transmission of the dynamic forces from the source of excitation through the machinery mountings down to the subsoil or the adjacent structural members. In addition foundation and support must have sufficient rigidity to avoid unacceptable deformations to the machine or parts thereof. For example, the supports of the machine axle should have approximately equal rigidity. Relative displacements of the supports due to shrinkage of structural elements have to be avoided. Tolerable values for relative displacements of supports, eccentricity of supports, etc. have to be given by the machine manufacturer.

Large machines, as for example a steam turbo generator, should preferably have their foundation separated by joints from other building parts to avoid vibration transmission to adjacent structural elements. Today, the preferred material for machine foundations is reinforced concrete. Rules applicable to reinforced concrete used in construction work have to be followed. Additional considerations are e.g. regulations for mounting the machine, secondary mounting aids, coupling of pipes and ducts, position of mounting, etc.

Costs of machine foundations have to be in proportion to costs of machinery. It is usual that costs of machinery exceed costs for the foundations and supports by a large factor. Costs arising from operational interruptions due to machine damage or a possible need for the installation of vibration isolation must be considered also. In specific situations it may be necessary to include additional parts of surrounding structural members and equipment into the dynamic system just to be sure that none of the frequencies of this enlarged system becomes dominant.

If structural measures cannot avoid the risk of vibration transmission it is advisable to introduce vibration isolation measures. In any case preliminary measures should be taken so that vibration isolation can be mounted with only minor problems when operation of the machine is impaired. However, stiffening of a support system may lead to excessive cost.

Vibrations due to rotational and oscillatory motions are more or less similar in cause, effects, and hence also in countermeasures; thus they are treated together. Only vibrations due to intermittent motion of machinery with impacting parts are fundamentally different and these require special measures.

b) Data desirable for the design of machine supports

The following data should be furnished by the *machine manufacturer:*

- loads and forces of rigid and moving parts of machines including the centre of gravity of these parts
- forces due to short-circuiting
- loads of additional installations if they are standing on the foundation or if they are attached at the bottom of the machine
- forces caused through operating the machine and through disturbances (e.g. start-up and shut-down of the machine), and the operating frequency
- loads of pipes, vacuum pressure of capacitor, etc.
- forces caused through changes in temperature on the machine itself (e.g. friction on supports due to changes in casing dimensions caused by changes in temperature)
- changes in temperature through heat radiation of machines or additional installations and piping
- force on bearings due to unbalance and frequency content of dominant force amplitudes
- possible dynamic forces caused by disturbances in additional installations and in piping.

Data to be furnished by the *plant manager:*

- expected forces that exceed those given in standards, e.g. designing for earthquakes
- possible additional loads such as operating loads or loads resulting from installation.

Data to be furnished by *geotechnical investigations:*

- In many cases data supplied are approximate. Lower and upper bounds of
  - subsoil classification and material parameters
  - position of different subsoil-layers
  - groundwater level.

  Selected soil parameters have to be used for specific calculations. For special cases it is recommended that the corresponding soil parameters be determined by detailed investigations, including soil dynamics.

c) Measures for rotating or oscillating machines

Vibration tuning

Separating the frequencies by tuning (see also Appendix A) is the most effective measure against machine-induced vibrations. Some cases may require the calculation of a forced vibration case or warrant special measures.

Since the objective is to completely avoid any state of resonance, the following parameters become important (see Figure A.2):

- the dominant natural frequencies of the structure as a whole and of the member immediately under the machine, taking into account existing spring-damper elements or a base slab (called in the following the machine base)
- the frequencies of the dominant dynamic force components (operating frequency of the machine and relevant higher harmonics).

If the fundamental frequency f_1 of the isolated system falls clearly below the operating frequency f_B of the machine the case of *low tuning* exists; the machine is said to run at over-critical speed. For effective *high tuning*, the fundamental frequency f_1 of the isolated system must be established well above the highest frequency component for which an appreciable contribution to the dynamic force exists (i.e. higher harmonics of the operating frequency); the machine is said to run at under-critical speed.

Tuning between machine and support frequencies is commonly referred to as vibration isolation. In the present context of preventing forced vibrations from being transmitted to neighbouring zones, one speaks of *active isolation* (not to be confused with the term "active controlled isolation" which means in its simplest form an isolation achieved by means of a feedback control system). The opposite action of preventing external vibrations from reaching an object is termed *passive isolation*. The effectiveness of vibration isolation is expressed in terms of the degree of isolation.

Figure 2.5 shows various types of machinery and their respective average operating frequencies and compares them with the natural frequencies determined by subsoil conditions and isolation materials. Problems with existing support systems may be eliminated by re-adjustment of out-of-balance forces.

The choice of supports for machines depend on the type of machine and its operating frequency. Figure 2.6 shows the three principal possibilities of low tuning with mounting of machines on floors or slabs. Principles b) and c) in Figure 2.6 need a sufficiently stiff substructure compared to the spring stiffness of the isolating elements. Table 2.2 shows various types of foundations and the related type of machines they are used for. For more details see also references [2.6], [2.7] and [2.8].

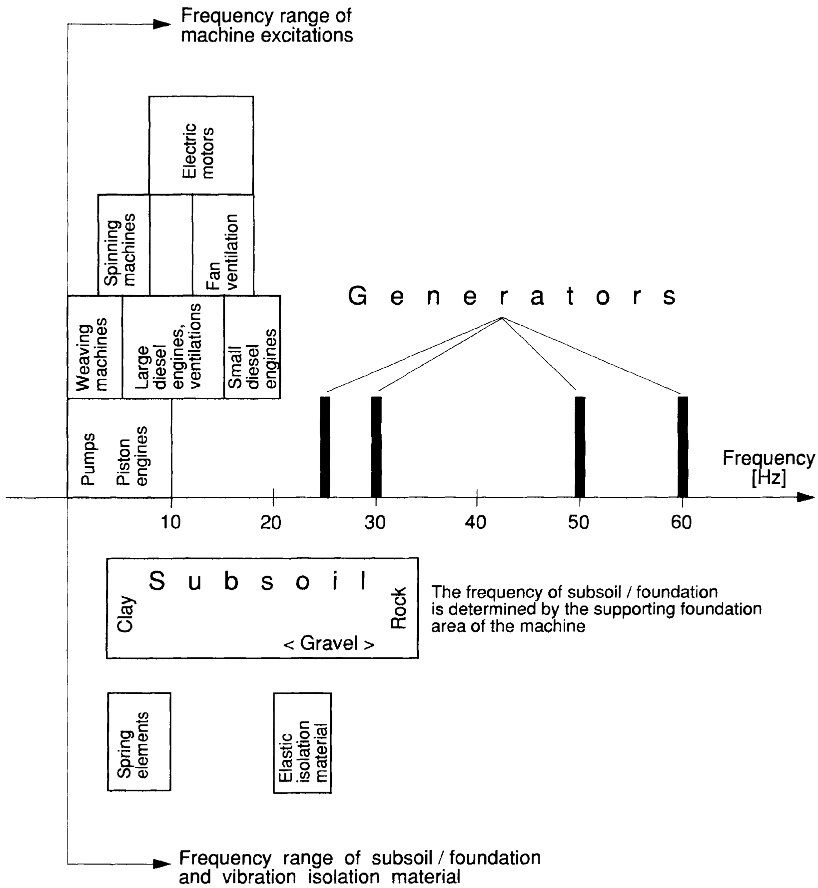


Figure 2.5: Overview of the operating frequency ranges of machines and achievable frequencies of foundations and supports on different subsoils and spring-damper elements

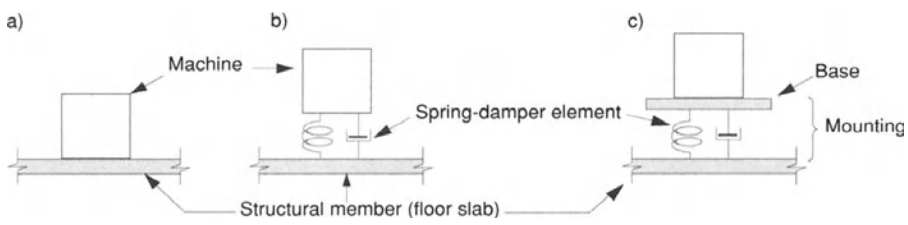


Figure 2.6: Principles for low-tuning of machinery with rotational or oscillatory motion [2.2]:
a) direct mounting on a structural member b) support on spring-damper elements
c) mounting on a vibration-isolated base

Type of foundation	Mainly used for:	Space needed under machine?	Tuning	Uncertainties in design and construction
Block (box)	· pumps · ventilators · diesel units · mills · compressors	not necessary	high or low according to sub-soil-conditions	natural frequencies and moduli of elasticity of the sub-soil are usually estimated values only; calculations are mostly done with estimated values for upper and lower bounds
Slab	· ventilators · gas turbines · compressors			
Frame-support	· turbo engines · ventilators · compressors · saw frames	necessary for special devices, maintenance, access, etc.	low or in some specific cases high	calculated modulus of elasticity of reinforced concrete cannot be obtained reliably (values at construction site are too high); uncertain sub-soil conditions
Vibration isolation	· turbo engines · mills · diesel units	technically possible for installing special devices, maintenance, accessibility of isolators (spring-damper elements)	low	best suited for calculation; corrections of wrong interpretations simply by exchanging of vibration isolators

Table 2.2: Types of foundation and domain of application

Low tuning

On the one hand low tuning reduces the reaction forces substantially, while on the other the resulting soft support yields large displacements even under static load. Certain conditions must be fulfilled for low tuning to be effective:

(1) The machine should be operated at a frequency of more than 4 to 6 Hz.
(2) Higher structural natural frequencies must not coincide with the dominant harmonics of the dynamic force if there is a risk of mode excitation.
(3) The soft base of the machine must not cause manufacturing problems.
(4) No intolerably large deflections or vibration velocities are allowed during start-up and shut-down of the machine.

Condition (1) reflects the constructional limitation for the degree of softening of the base that can be achieved. The limit is often reached with a stiffness resulting in an isolation-system frequency of approximately 1.5 to 3 Hz; this will require a minimum operating frequency of approximately 4 to 6 Hz. The idea is to achieve a tuning ratio of about 2. However, a tuning ratio between 2.5 and 4 is preferred since a larger ratio yields smaller reaction forces. Tuning below 2 Hz can lead to problems if the operating frequency of the machine is reduced. Ideally, the operating frequency of a machine should be above 12 Hz for successful low tuning.

	Helical springs made from steel, single or multi positioning	Rubber elements	Reinforced cork elements	Air cushions	Plate springs
Frequency domain	horizontally and vertically designed natural frequencies 4 to 10 Hz	5 to 20 Hz	5 to 20 Hz	very low, limited possiblility for variations 0.5 to 3 Hz	works only in one direction 4 to 10 Hz
Spring characteristics	linear	progressive since rubber is elastic but non compressible			
Range of application	in cases of urgent need for excellent isolation	in cases of high frequencies, low loads, minor vibrations	isolation of high speed machines (high revolutions)	in low frequency domain, and where small displacements are required	
Damping	separate damping elements must be supplied if damping is required	damping effect is larger than for steel springs but the extent of damping is material and temperature dependent			
Remarks	possibility of access and exchange of elements; tuning of the system possible		no access or exchange possible	sustaining the air-cushion is very expensive, therefore not often used for machine foundations	

Table 2.3: Outline of vibration isolation elements (spring-damper elements)

Condition (2) prevents resonance phenomena in the higher frequencies of the base.

Condition (3) results from production requirements, as a softer support yields larger displacements of the machine.

Condition (4) can often be ensured by precise instructions on start-up and shut-down of the machinery, possibly in combination with braking devices in the slowing-down phase or enhanced damping of the base.

If applicable, low tuning can be put into practice in three possible ways (Figure 2.6 and [2.2]):

- Direct mounting of the machine on the structural member if its frequency is below 2-3 Hz.
- Mounting of the machine on spring-(damper) elements. The spring-(damper) elements differ with the machine characteristics (dimensions, mass, stiffness, operating frequency, etc.). The different types on the market comprise steel springs, elastomeric or natural rubber pads and air suspension (see Table 2.3).
- Mounting on a vibration-isolated base. This measure is suitable for machines with:
  - very large dynamic forces
  - insufficient stiffness for isolation
  - very small mass which would demand extremely soft springs for low tuning
  - very eccentric centre of mass of the machine
  - asymmetrical machines.

Due to additional mass of the base one can employ stiffer spring-damper elements than one could in direct support, thereby reducing the vibration amplitudes. The stabilizing mass should have at least the same weight as the machine alone and preferably it should be larger (up to a ratio 8:1)

High tuning

High tuning is chosen in those cases in which low tuning is impractical (see Table 2.2). The requirements for high tuning are:

(1) The frequency of the highest relevant force component does not exceed 20 Hz approximately.
(2) Production requires that machine vibrations be kept small.

Condition (1) results from the fact that a structure or structural member cannot be arbitrarily designed for high stiffness within given project design conditions. Since the computational prediction of the structural frequency contains some uncertainty, the tuning ratio is augmented by an additional safety factor. The lower bound of the fundamental frequency of the structure or the respective structural member thus becomes:

$$f_1 \geq f_B \cdot n_h \cdot a_b \cdot s_f \qquad (2.8)$$

where f_1 = required fundamental structural frequency [Hz]
f_B = operating frequency of the machine [Hz = r.p.m./ 60]
n_h = order of the highest relevant harmonic of the forces
a_b = reciprocal of the tuning ratio (> 2.0)
s_f = safety factor (usually taken as 1.1 to 1.2)

In general, high tuning is only feasible with a rigid connection of the machine to the structure.

In calculating the expected design frequency of stiff structures the flexibility of the ground may need to be taken into account. This is especially true for pile foundations. Under some circumstances it can substantially reduce the combined frequency of the soil-structure system and thus limit the success of high tuning.

In calculating the expected natural frequencies of structures it is usually sufficient to examine independent partial systems. The bounds given are for the uncracked state (state I). Loss of flexural rigidity in case of localised cracking (state II) does not have any significant influence on the dynamic behaviour of block-foundations. The flexural rigidity can be reduced by half of the uncracked state in framed foundations and when calculating the eigenfrequencies of floor slabs. For a more precise evaluation of the cracked state refer to [2.2]. Machinery, casings, piping etc. are treated as mass without stiffness.

Condition (2) is covered by the need of stiff support and mounting of the machine resulting in small vibrations.

d) Measures for machines with impacting parts

The solution of a practical problem is largely determined by the force-time function resulting from the interaction of the impacting part with the static part of the machine and the substructure or subsoil. The important parameters of the force-time function are the duration of the impulse, the momentum and the peak force.

Vibrations of machines with intermittent motion are best counteracted by low tuning, i.e. by providing low stiffness of supports. High tuning, in contrast, is not likely to be an alternative as the force spectrum is typically rather wide, i.e. relevant force components are exerted over a wide frequency band.

Low tuning is applicable under the following conditions:

(1) The resulting displacement amplitudes of relatively large magnitude must not interfere with production requirements.
(2) The static loading on the structure must be accommodated without difficulty.

Condition (1) is frequently not met for machines in the metal-working or plastic-producing industries because of the usual production requirement of a very stiff mounting. This problem can be mitigated considerably by using a stabilizing mass and thereby rendering low tuning feasible. Low tuning finds a common application in hammer machines, coal and clinker mills. Low tuning may be achieved most efficiently by isolation.

Condition (2) can normally be achieved by an appropriate design of the structure or the respective structural member.

The above conditions being met, low tuning of machines with predominantly intermittent motion can proceed along the same lines as discussed in the previous section for rotating or oscillating motion.

Direct mounting of the machine on a relatively soft structure or member is only feasible for small impacting forces. Attention has then to be given to the possibility of substantial excitation of higher structural frequencies.

e) Rules for detailing and construction

General aspects

The machine itself is decisive for foundation and support design. Details about the minimum size of the required foundation and its influence as stabilizing mass have to be furnished by the manufacturer. Maximum tolerable displacements as specified by the manufacturer can be adhered to with the correct stabilizing mass.

The basic rule is to design simple systems that have clear and short load transmission into the foundation. Abrupt changes in cross section, grooves, cantilevered parts, notches and openings at points of concrete compressions should be avoided. The possibility of vibrations in sub-systems of the structure is to be considered too.

The slab of framed concrete supports should be as stiff as possible so that the relative deformation within the structure, and thus the effects on the machine shaft, remain small. The distribution of supports beneath the slab should be such that the bearings of the shaft should, as far as possible, be equally stiff.

In designing the machine-support one should always keep in mind the uncertainties in the calculation. Also, preventive measures for a possible subsequent stiffening (i.e. tuning) of the foundation should not be forgotten.

Joints

Joints in block slabs of pedestals should be avoided. If this is not possible, rules for construction and remedial measures should be adhered to. Joints should be positioned in places with minimum stress. A possible splicing of the reinforcement should be located away from the concrete joint. Under some circumstances it is better to have additional joint reinforcement.

Reinforcement

In general, the rules for the arrangement of the reinforcing in the construction should apply (simple installation, few constrictions when concreting, adequate support especially of the upper reinforcement in deep foundation blocks).

The arrangement of the reinforcement has to be carefully adjusted to fit the supports of the machine bearings and their mountings (position of the overlapping bars, etc.).

Frequently in the case of massive foundations or parts of foundations the reinforcement is distributed along the three main axes of the foundation (cubic or spatial reinforcement). This reinforcement serves the purpose of carrying tensile stresses that are not accounted for in the calculations and should prevent any crack formation - deformation restraint during the setting process - so that the corresponding structural component acts as a single unit during its life span, as is assumed in the calculations.

Several standards and guidelines give values for the spatial arrangement of the reinforcement. It is advisable, especially for larger foundations, to verify that the expected tensile forces and their distribution are adequately accounted for.

Machine anchoring

The position and size of the forces acting at the base of the machine, as well as the envisaged method of fastening, have to be supplied by the machine manufacturer. These details influence the design of the foundation and of its reinforcement, in particular for finely proportioned foundation members. As well, these details affect the positions of the fasteners with respect to one another and to the edges of the foundation or possible openings and recesses in the foundation. These statements also apply analogously for fixing spring-damper elements to the machine and to the stabilizing mass or the machine base. Subsequently drilled fasteners offer, in contrast to pre-installed or embedded fixtures, the advantage of later support adjustments, and the avoidance of support tolerance problems, etc. The individual fastening elements must be able to accommodate the high load variations. An optimum design solution (limiting notch stress concentration effects, geometry of bolt or female anchor), the right choice of material and material processing as well as the possibility of applying prestress to the fastening elements increase its life span considerably. A periodic check on the prestressing is advisable, since the relaxation behaviour can be important.

Aspects of construction

The concrete for block and slab foundations, column and frame supports should, if possible, be placed continuously in one working cycle. In the case of massive structural elements (block and slab foundations) it is advisable to use cement with low thermal properties in order to reduce the amount of heat introduced into the system and so restrict the build-up of tensile stresses and the development of cracks due to the heat of hydration.

2.1.7 More advanced design rules

If the success of frequency tuning is somewhat doubtful (e.g. because of a wide frequency spectrum of the dynamic forces), calculation of the structural behaviour under forced vibration is recommended. Such a calculation yields information on the maximum expected vibration amplitudes, which could then be compared with the acceptance criteria for evaluating the tolerance of vibration effects. However, such a calculation is in itself subject to considerable uncertainty, especially as far as the actual dynamic forces from the machine and realistic damping properties are concerned.

The modal influence of neighbouring vibrating structural parts must be included in the evaluation when simplifying a complex dynamic system.

Supports as shown in Figure 2.6b and c can be simplified as 2-DOF (two degree of freedom) models. It is possible to decouple the two masses if

- the natural frequencies of separate systems differ from each other and
- the mass of the machine (including stabilizing mass) is small in comparison with the equivalent support model.

The system then becomes a set of independent SDOF models (for more details see [2.2], [DIN 4024/2]).

A periodic force-time function can be decomposed by Fourier analysis into its constituent harmonics, of which subsequently those with non-negligible amplitudes are selected as forcing functions. If the loading is of a transient nature and exhibits strongly the features of Section 2.1.1 then the computation of a forced vibration must proceed in two phases:

- *Phase 1:* Computation for the effective duration of the force (impact)
- *Phase 2:* Computation of the subsequent free vibration (after the force has ceased), for which the current values of the state variables of motion at the end of phase 1 serve as initial conditions for evaluating the integration constants of phase 2.

If the calculation is carried out with the aid of the finite element method then attention must be paid to suitable modelling (element choice and mesh subdivision). When considering the demands on accuracy of results the ever-present scatter with respect to material strength, modulus of elasticity, damping characteristics, physical dimensions etc. has to be taken into consideration. Frequently, the interaction between the foundation or the structure and the sub-soil influences the results.

2.1.8 Remedial measures

There may be occasions where the cases presented in Section 2.1.6 and 2.1.7 do not, for technical reasons, lead to the desired result. In any case a reduction in possible out-of-balance forces should be attempted by the manufacturer before considering design improvements to the machine structure or its foundation. An improvement of the tuning factors by means of subsequent design measures (e.g. stiffening of the structure or a part of the structure) is only possible in a limited number of cases and is normally very expensive.

If at the design stage a possible later installation of vibration isolation or a mass tuning system is planned, then this can keep the costs of such a measure low. It may be favourable to carry out measurements during the construction process to get more insight into relevant eigenfrequencies and damping behaviour of the structure in progress. With regard to remedial measures, earlier design assumptions have to be re-evaluated, preferably on an experimental basse.

Special devices, which may be either passive or active, can be used in special situations. Active devices include servo-hydraulic driven masses, controlled by means of an electronic/hydraulic circuit to adjust the movement of the machine or its base support. They require additional external power for their operation.

A passive device would be the tuned mass damper described in Appendix D. In contrast to the protective measures against structure-borne waves it only acts in a narrow frequency band. A tuned mass damper consists of one (or, depending on the requirements, on a number of) mass-spring-damper system(s). In addition, it is possible to employ highly damped dampers. These are additional vibrators, which dissipate the energy in the system by means of relative displacement damping. However, it must be remembered that an oscillation damper or a highly damped damper is in themselves a vibrating mechanical system requiring maintanence. For impact-type forces, shock absorbers are also used. They take up the force impacting on the object and vibrate until the motion dies out.

2.2 Bell towers

H.G. Natke, J.H. Rainer

2.2.1 Problem description

Bell towers are excited dynamically by bell movements and by wind. This section treats vibrations from bell movements whereas vibrations of towers from wind are treated in Sub-Chapter 3.2.

Bell motions occur at low frequencies and cause flexural and torsional vibrations of the tower. The forces associated with bells are those of the periodic suspension forces which can be considered as externally applied excitations [2.9].

For forced vibrations of the tower in flexure and torsion, resonance with the harmonic components of the bell forces - particularly that of the third harmonic - should be avoided. Resonance can result in tower movements that are perceived as being excessive. The tower motion depends greatly on the base support. In the case of a flexible foundation a nearly rigid-body rotation about the base may occur. In the case of a stiff or rigid foundation, nearly pure bending or torsional modes of the tower can be activated.

The impact of the bell clapper also causes high frequency vibrations which are transmitted into the belfry. The older wooden bellfries filter the high frequencies whereas the steel ones do not. Thus, they introduce additional stresses into the belfry which are then transmitted to the tower.

2.2.2 Dynamic actions

The governing parameters for the dynamic forces of ringing bells are (see Figure 2.7):

- bell diameter d
- weight of bell G
- swing angle β
- shape factor c
- clapper rate (equal to the strike rate)
- strike rate N: Bells are often used in the range between musical notes $b = b^o$ with 42 impacts/min. and e'' with about 68 impacts/min. To account for inadequate maintenance of bell ringing controls, the strike rate used for calculations is varied by ± 3 impacts/min.
- swing rate n: Number of bell swings per minute equal to $n = N/2$ [min^{-1}]; then $\omega = \pi \cdot n/30$ [s^{-1}];
- swing frequency $f = n/60 = N/120$ [Hz].

The swing angle β is the maximum angular displacement ϕ. For $\phi > \sim 7°$ the motion is no longer harmonic, yet it is still periodic. In England, swing angles of 360° are common, in Germany 70° to 80°, in Europe generally 80° to 110°, in the USA up to 160°. The bell suspension usually consists of a straight yoke in order to achieve a large Doppler effect.

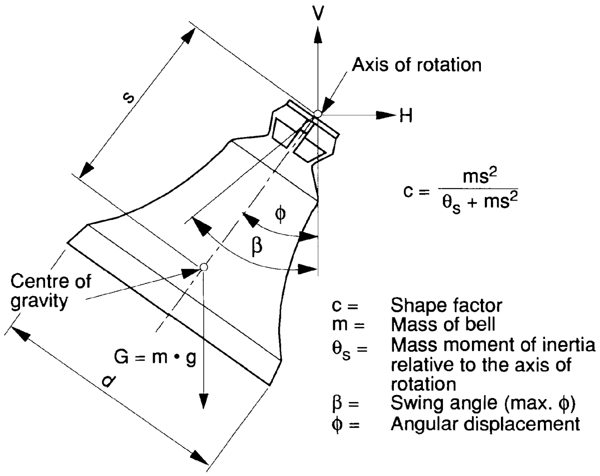


Figure 2.7: Parameter definitions without the clapper

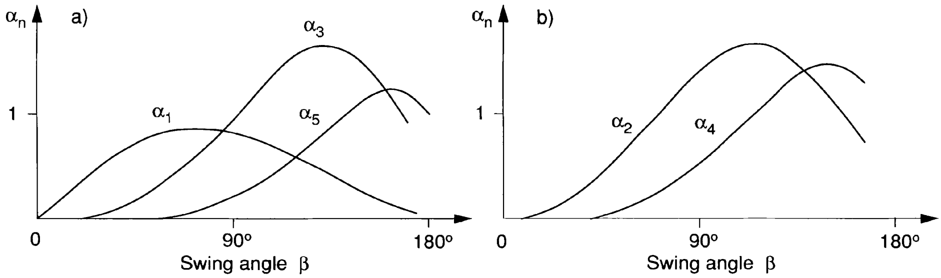


Figure 2.8: Normalized force amplitudes of the harmonics of the force-time function from bell ringing: a) horizontal forces, b) vertical forces

The forces arising from bell ringing can be taken from [DIN 4178]. The excitation forces are periodic and can therefore be represented by Fourier series. The horizontal forces are proportional to the Fourier coefficients $\alpha_n = \alpha_{2m-1}$, $m = 1, 2, \ldots$ (Figure 2.8a), the vertical forces to the Fourier coefficients $\alpha_n = \alpha_{2m}$ (Figure 2.8b) (ratio of the dynamic force amplitude to the weight of the bell, see also Appendix A).

For a swing angle greater than 80° the coefficient α_3 predominates. Thus danger of resonance is not at the fundamental frequency of the bell oscillations, but rather at the frequency of the third harmonic.

2.2.3 Structural criteria

a) Natural frequencies

The need for avoiding resonance has already been mentioned. The range of the swing frequency f of normal bells is given from 0.3 Hz to 0.6 Hz. As the third harmonic is dominant the critical frequency range of a bell tower structure is 0.9 Hz to 1.8 Hz. Slender bell towers often have their fundamental bending and/or torsional frequency in this range.

b) Damping

Equivalent viscous damping ratios ζ of modern reinforced concrete bell towers (nearly bare structures, see Appendix C) are given in Table 2.4.

	damping ratio ζ		
Construction type	min.	mean	max.
Reinforced concrete (~ uncracked or only few cracks)	0.007	0.01	0.014

Table 2.4: Common values of damping ratio ζ for bell towers

2.2.4 Effects

Effects on people from vibrations, noise, etc. may cause annoyance and thus affect the serviceability of the structure. Especially church towers that open into the nave, with choir loft or choir balcony, have limited serviceability on account of vibration sensitivity of people. For old church towers attached to old churches the horizontal motion of the tower may be transmitted to the church nave. As a result, cracks frequently occur at the connection between tower and nave and also shear cracks at the windows, at least in the first bay of the nave. The relative horizontal motion between the heavy tower and the light nave results in leaning of the tower and causes the highest soil pressures at the foundation joint. Furthermore this motion may result in axial forces in the ridge beam of the nave and the transmission of the vibrations to the entire church roof. Modern bell towers which are not designed for dynamic bell forces may cause similar damage.

2.2.5 Tolerable values

Church towers with motion amplitudes of ± 1 cm at the top are intolerable to people on the tower, even if there is no concern about structural safety. The same applies to tower motions in the order of a few centimeters observed from outside the tower.

Tolerable values for annoyance to people by vibrations are given in Appendix I. Tolerable values for church towers are given in [DIN 4178] (see also Appendix J).

2.2.6 Simple design rules

First, a frequency investigation for the tuning of the structure needs to be carried out. The tower must be modelled together with the foundation and (static) mass of the bells. Initially, bending is considered in both directions (first order). The objective should be to achieve a fundamental tower frequency that is at least 20% higher than the excitation frequency of the third harmonic. If, for example, one assumes a strike rate of 50 impacts per minute (a value that lies within the mentioned range of 42 - 3 impacts/min. and 68 + 3 impacts/min.), then the swing rate becomes $n = 25$ swings/min. and results in $f = 25/60 = 0.42$ Hz. Thus the fundamental frequency should be at least $3 \cdot 0.42 \cdot 1.2 = 1.5$ Hz. In many cases of modern bell towers this requires already a relatively stiff foundation. In general, a resonance separation of at least 20% should be obtained for the least favourable natural frequencies. If the natural frequencies are determined from measurements on the existing structure, then the resonance separation should be at least 10%.

2.2.7 More advanced design rules

If more sophisticated considerations are desirable a forced vibration investigation has to be made. If in a tower several bells are hung above one another, then it is recommended that analytically the forces be brought to one common level by means of transfer moments (Figure 2.9). If bells hang beside one another, then they should be referenced analytically to the tower axis. The transfer moments are then the torsional moments.

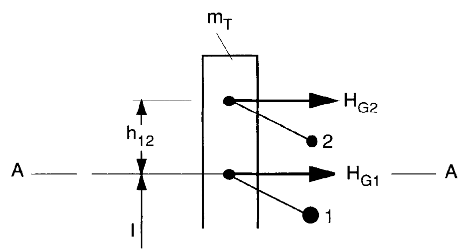


Figure 2.9: Illustration for transfer moment $h_{12} \cdot H_{G2}$

For bells that are intended to be referenced to the plane A-A (elevation l) in Figure 2.9, the equilibrium conditions are:

$$Q(l) = H_{G1} + H_{G2} \tag{2.9}$$

$$M(l) = H_{G2} \cdot h_{12} \tag{2.10}$$

where $Q(l)$ = shear force in the tower at height l
$M(l)$ = bending moment at height l
H_{G1} and H_{G2} = horizontal forces of the bells.

If the tower is modelled as an elastically supported cantilever beam, then the inertia forces of the bells are included in the boundary conditions and therefore in the calculation of Q and M. If, in addition, one wishes to consider the tower mass m_T above the force reference point, then the equations of motion of the referenced centre of gravity of the bell is to be augmented by the equation

$$m_T \ddot{w}_T = \sum_{i=1}^{2} H_{Gi} \tag{2.11}$$

where $w_T(t)$ = horizontal motion of the referenced centre of the top tower mass m_T.

Stress calculations in the horizontal direction have to consider the bells in motion. For bell locations in the upper third of the tower an equivalent static force procedure is presented in [DIN 4178]. If the bells are located below the top third of the tower, the actual dynamic loading can be greater than that calculated by the equivalent static forces, and a dynamic analysis is therefore required. According to [DIN 4178] it is adequate to substitute the maximum bell forces as static forces when the tower height is less than 20 m, the tower has a closed cross section and a slenderness ratio $h/b \leq 4$, and when a slab foundation is located on dense, non-cohesive soils or rock.

Because of uncertainties in the results, the fundamental natural frequency is to be determined by measurements after completion of the structures (even without bells), so that the bell assembly can then be properly sized.

For the design in the vertical direction it is generally sufficient to consider the maximum vertical bell forces. The same applies in general to the verification of the belfry and of the substructure under the belfry; a load safety factor for the static design of 1.3 is indicated.

A sophisticated mathematical modelling of the tower proceeds along established methods of structural dynamics ([2.2] [2.10] [2.11]). The tower dynamics is characterized by the interaction of the bells and the tower; in most cases this interaction can be neglected. A difficult part is the modelling of the bell motion ([2.12] [2.13]). Nonlinear differential equations apply to the bell and the clapper. It is common to idealize the bell as a physical pendulum with initially a non-moving pivot point, i.e. neglecting the action of the clapper. The periodic suspension forces, transferred from the belfry to the upper end of the tower, can be introduced into the energy formulation by means of the horizontal tower motion $w_A = w(l)$ for the suspension points (see Figure 2.9). From there the Lagrangian equations can be derived.

2.2.8 Remedial measures

If after completion it is found that a tower is excessively excited or that the tower motion is unacceptable, then remedial measures need to be taken. Six different measures can be considered; note that the measures 3. to 6. affect the sound quality of the bell.

1. Change of the natural frequency (tuning) of the tower by an increase or a reduction of stiffness. A larger separation from the resonance frequency reduces the amplification factor and consequently reduces the stresses at the critical location. This requires a major effort, and architectural features may be affected.

2. Attachment of a tuned compensation pendulum. This produces forces that counteract the horizontal forces produced by the bell. The objective is to attain a zero resultant force.
3. Change in the distance between centre of gravity of the bell and its axis of rotation. In general a reduction of the distance to the centre of gravity, through shortening the bell support collar, changes the frequency of ringing and consequently reduces the bell forces.
4. Change in the mass moment of inertia of the bell. An added mass is applied to the bell yoke which increases the rotational moment of inertia and as a consequence reduces the strike rate. Care needs to be taken that the swing frequency of the bell and the frequency of the clapper do not coincide, otherwise the clapper won't hit the bell. The new natural frequencies and the horizontal forces need to be investigated in relation to the tower properties.
5. Attachment of fluid dampers to the tower in the neighbourhood of the belfry. A liquid in a container serves as damped absorber [2.14]. This is one of the add-on measures that reduces tower motions.
6. Lastly, the bell could be fixed and the clapper would impact this bell. This affects the sound quality to the greatest extent.

2.3 Structure-borne sound

H.G. Natke, E.U. Saemann

2.3.1 Problem description

Local time-dependent displacements of a solid propagate by waves in a finite time with finite wave velocity. Waves propagating in a structure and being radiated as sound into the air are called "structure-borne sound". Sound with frequencies of more than 16 Hz can be heard by the human ear.

Sub-structures affected by structure-borne sound can be floors, walls, windows, roofs, pipes.

One has to distinguish between travelling waves and standing waves (vibrations) in structures. The standing waves result from reflections at boundaries (in geometry and material) so that the induced deflection varies in shape only by a time factor.

Since solids can transmit shear strains, wave propagation in structures are accompanied by a transverse component, in contrast to that in the air. Just as for longitudinal waves, transverse waves and the combination of the longitudinal and transverse waves can lead to:

- high number of stress reversals in the deflected material;
- flexural waves, and in consequence to radiated sound from structural surfaces having particular properties;
- transmission of structure-borne sound into and within buildings via foundations and other connections (e.g. pipes).

Of greatest importance for the radiation of sound to the air are those types of waves that have a component of motion normal to the surface of the solid, since only through this component the surrounding air can be excited.

2.3.2 Dynamic actions

The dynamic actions of sources producing structure-borne sound may be manifold: machines, human body motions, fluids in pipes including valves, etc.

2.3.3 Structural criteria

Natural frequencies and damping properties of structures affected by structure-borne sound can vary in a wide range. Whether a structure shows structure-borne sound depends on its properties (frequency response function) as well as on the parameters of the dynamic action (frequencies, amplitudes, etc.).

2.3.4 Effects

The effects of structure-borne sound on people and structures consist of those with respect to vibrations (see Sub-Chapter 2.1) and those with respect to noise.

Depending on the particular application the effects will be described in different ways. For a description of problems concerning ageing, service-life and fatigue limit of a structures, displacement amplitudes are employed. The effect on buildings is described in terms of peak values of vibration velocity and of impedances. The effects on people with respect to vibrations are described by means of frequency band-limited vibration velocities assessed in the frequency and time domain, and with respect to noise by means of weighted sound pressure levels (see Appendix B).

2.3.5 Tolerable values

The discussions of tolerable values in Section 2.1.5 and in Appendices I and J are also valid for this sub-chapter. Tolerable values for structure-borne sound in the sense of emission do not exist, of course. For immission, however, [VDI 2058/1] gives such values (acoustic levels) due to structure-borne sound. In addition, [2.18] defines tolerable values for flats which are connnected with an industrial plant.

2.3.6 Simple design rules

Concepts and methods of vibration insulation is treated in a general way by [VDI 2062/1], whereas [VDI 2062/2] deals with insulation elements and their properties and with elastic pipe connections for controlling high frequency vibrations (structure-borne sound).

Calculation of the magnitudes of structure-borne sound from vibrations induced by machinery cannot be done accurately. It is, however, possible by means of approximate assessments to achieve an overview of the processes involved in propagation of structure-borne sound. If the sound levels are of a magnitude that requires noise protection, or if the radiated sound causes annoyance, then it is most effective first to exhaust all possibilities of reducing the sound at the source. If further steps are required, then room acoustical measures can be applied.

The formation of structure-borne sound can be inhibited by:

- influencing the initiation of structure-borne sound by means of
  - changing the excitation forces
  - changing the input impedances
- influencing the transmission of structure-borne sound by means of
  - insulation of structure-borne sound
  - absorbtion of structure-borne sound
  - auxiliary devices
- influencing the structure-borne sound at the position of radiation from the surface (this is not treated here).

Room-acoustical measures do not reduce the generation of structure-borne sound, but affect only the sound level at the receiver. This employs principles of room acoustics, shielding or enclosures of machinery, or ancillary counter measures; these are, however, not discussed here.

a) Influencing the initiation

Changes of the excitation forces

Data on the type and magnitude of forces on the machinery mounts should be provided by the manufacturer. A complete description of the forces requires data about amplitude, direction, time-dependence and frequency content. One distinguishes among deterministic (periodic, transient) and stochastic excitations. The most important transmission path of machinery-induced vibrations occur via the machinery mounts. From the frequency content of the forces in the machinery mount it can be determined whether the building will likely be excited. This happens when one of the excitation frequencies coincides with one of the natural frequencies of the building, considering standing waves (see Sub-Chapter 2.1). Accurate knowledge of the natural frequencies of the structure and especially of the machine room (changes of local resonance) allows to avoid resonance by suitable choice of machines (among various types with different dynamic behaviour), or for existing machines, by a change in operating conditions. The latter is generally possible only within narrow limits.

Changing the input impedances

If uncertainties exist, it is recommended that already during the design phase the vibration amplitudes be determined by an expert. A simplified description of the relationship between excitation force and the dynamic response of various points of observation on the structure is possible via the concept of impedance, which is the ratio of the applied force to the resulting vibration velocity at the point of force application (see Appendix A.6). Detailed considerations on impedance for low-noise design were presented in [2.15]. It contains a proposition for a systematic procedure of assessments and a presentation of numerical examples supported by experimental results.

In summary, the measures for a reduction of machinery-induced structure-borne sound in buildings are changing the excitation forces and/or changing the input impedances.

b) Influencing the transmission

The following attenuation measures, also called "structure-borne sound insulation", are based on limiting the transmission of structure-borne sound by means of changes in impedance of the transmitting structure. For greatest effectiveness such measures should be taken as close to the sound source as possible. The changes in impedance are achieved by means of

- added masses
- resilient supports and layers (inserted construction element, e.g. floating floor)
- changes in cross section

as well as through other discontinuities.

Effectiveness of sound reduction

The effectiveness of sound reduction of a certain insulation method can be described by various quantities. A possible description is the difference L_K in sound pressure level L_p

$$L_K = L_{p\ \text{before}} - L_{p\ \text{after}} \tag{2.12}$$

from before and after the attenuation measure (see Appendix B). Other commonly used descriptions include the ratio of the forces acting in front of a support (i.e. an inserted construction element) and behind it, F_1/F_2, or the level $10 \cdot \log (F_1/F_2)^2$ (see Figure 2.10a, where a spring is shown as a support).

Another method of describing the sound reduction is the insertion loss L_e. For inserted construction elements whose dimensions are less than $\lambda/6$ (where λ = flexural wave length), the insertion loss is given by [2.16] as follows (see Figure 2.10b):

$$L_e = 20 \cdot \log \left| 1 + \frac{\frac{1}{Z_{1k}}}{\frac{1}{Z_i} + \frac{1}{Z_a}} + \frac{Z_{2l}}{Z_i + Z_a} \right| \text{[dB]} \tag{2.13}$$

where Z_i = source impedance above the point of insertion
Z_a = terminal impedance below the point of insertion
Z_{2l} = input impedance on plane 2 for freely moving plane 1
Z_{1k} = input impedance on plane 1 for fixed plane 2.

For point masses, massless springs and large impedances (low frequencies), the attenuation can be estimated reliably. For other cases (high frequencies) this becomes very complicated. A large insertion impedance can be achieved by inserting heavy masses or soft springs.

Added masses

For reduction of structure-borne sound added masses ("blocking masses") can be inserted so that they act as points of reflection to body waves. This illustrates the use of mass inertia for reducing vibration amplitudes. For a compact (relatively small) mass m the following approximation applies (for harmonic motion):

$$\frac{1}{Z_{1k}} = 0 \quad \text{and} \quad Z_{2l} = i\omega m \tag{2.14}$$

where $i = \sqrt{-1}$
ω = frequency variable.

From Equation (2.13) it can be seen that the insertion loss L_e increases with larger added mass and lower impedances Z_i and Z_a. Low impedances Z_i and Z_a are achieved through light, soft components (adjacent to the insertion element). The insertion loss increases with frequency by $40 \cdot \log (f/f_0)$ (where f = frequency, f_o = reference frequency) when both connected regions (connected to the inserted construction element) exhibit an impedance having the characteristics of a spring. For high frequencies a spring no longer acts like a massless

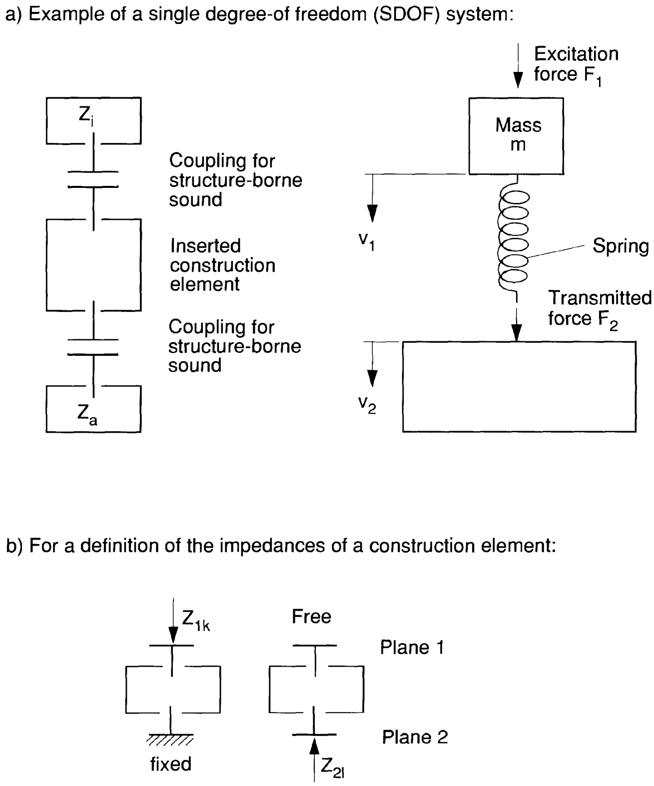


Figure 2.10: Construction element [2.16]

spring, and the resulting insertion loss is governed by mass m_b that moves with it. The moving mass m_b is defined as the mass within radius $\lambda/4$ around the point of excitation of the spring. $L_{e,\,min} \approx 20 \cdot \log\,(m/m_b)$. The qualitative variation of the insertion loss of an added mass is shown in Figure 2.11.

Resilient supports and layers

Insulation of structure-borne sound for various building elements (e.g. floating floors) or entire structures can be achieved by insertion of resilient supports or layers which corresponds to a change in material. For particularly high performance requirements, foundations need to be decoupled above the ground; even vibration-insulated rooms within the building can be contemplated.

Inserted elastic supports or layers may affect the stability of a structure. This stability must therefore be verified. Also, it is necessary to consider the deformation of the structure with respect to its serviceability.

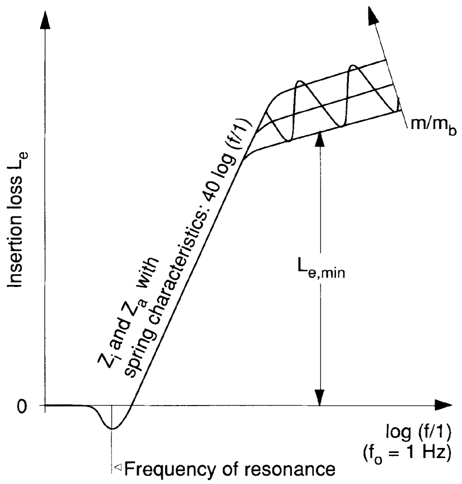


Figure 2.11: Insertion loss of a heavy mass [2.16]

Resilient supports and layers may be modelled by a spring. A spring with a spring constant k is described by:

$$\frac{1}{Z_{1k}} = \frac{i\omega}{k} \quad \text{and} \quad Z_{2l} = 0 \tag{2.15}$$

It can be seen from Equation (2.13) that the effectiveness of a spring as an attenuation element increases with increasing impedances Z_i, Z_a and with decreasing spring constant k. Large impedances Z_i, Z_a are achieved by means of heavy and stiff structures. The insertion loss increases with increasing frequency as $40 \cdot \log{(f/f_0)}$ when the adjacent structures of the insertion element (spring) have an impedance with mass characteristics. For higher frequencies, however, often at least one connected element to the spring will exhibit an impedance with spring characteristics. The insertion loss is then given by

$$L_{e,\,max} \approx 20 \cdot \log{(k_{eff}/k)} \tag{2.16}$$

where k_{eff} = effective spring constant of the structure at the point of insertion.

The qualitative variation of the insertion loss of a spring used as an insulation element is shown in Figure 2.12.

In addition to using these compact structural elements for attenuating structure-borne sound, combinations of these elements are also being employed as added elements; their vibration behaviour is to be modelled as a continuous system. Details for the calculation of such types of construction are presented in [2.11].

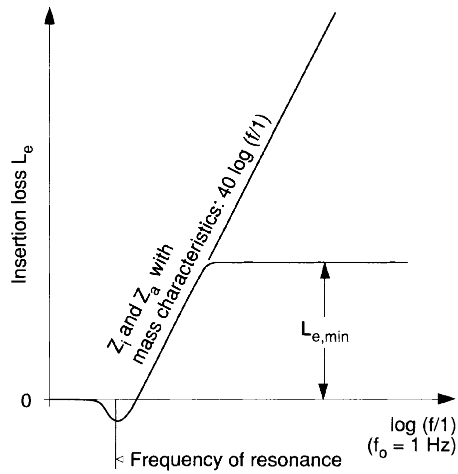


Figure 2.12: Insertion loss of a soft spring [2.16]

Change in cross section

Besides changes in materials, a change in cross section can also contribute to a change in impedance and consequently to an attenuation of structure-borne sound. For longitudinal waves the insertion loss is given by

$$L_e = 20 \cdot \log\left(\frac{q^{-1/2} + q^{1/2}}{2}\right) \text{ [dB]} \tag{2.17}$$

where q = cross section ratio: A_2/A_1 for beams, h_2/h_1 for plates
A = cross section area
h = cross section height

The ratio $q = 2/1$ yields $L_e = 0.5\text{dB}$.

For flexural and longitudinal waves Figure 2.13 shows the variation of the insertion loss as a function of the cross section ratio.

Where possible, one can of course combine a change in cross section with changes in material. If vibration-insulated structures contain connecting elements (for transmission of force, or cables, or pipes), they also need to be insulated if vibration bridging is to be avoided. If a machine is already minimized with respect to structure-borne sound, then there still remain pipes, hosepipes and cables to be considered. Here, flexible connections are available if the connections themselves (such as hosepipes) are not already elastic enough. Hoses need to be connected in bows and not simply straight and tight. From Equation (2.13) a theoretical attenuation of structure-borne sound can be estimated to be between 5 and 15 dB.

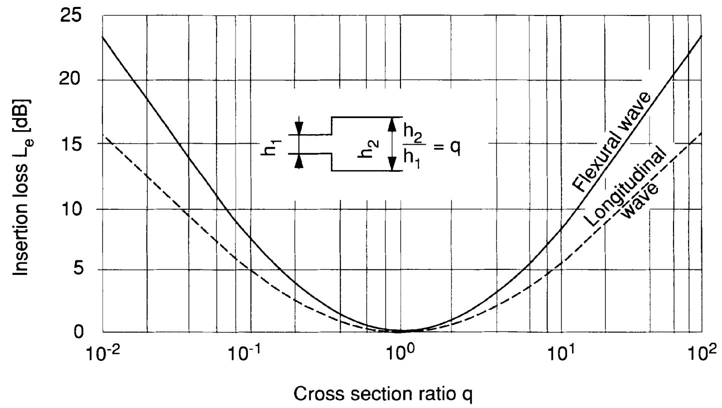


Figure 2.13: Insertion loss for a change in cross section

Absorption of structure-borne sound

Absorption of structure-borne sound means damping of flexural waves during propagation, with a resulting reduction in sound radiation. This measure is based on the principle of removal of structure-borne sound energy from the structure. Internal damping of the absorbing material occurs as a result of molecular deformations and to a lesser extent through heat conduction. The damping mechanisms of materials vary greatly. Besides linear viscous damping, frictional damping also needs to be considered. This can be achieved by friction of fixed (usually metallic) surfaces and in structures, by employing sand fills. Sand fills should not be of uniform thickness. The lowest frequency for which damping takes place is determined by the thickest layer. The internal damping can be described by the damping factor ψ as the ratio of the total damping to the total potential energy at maximum deflection. For a model (specimen) with uniform stress distribution the relationship between ψ and the damping ratio ζ is $\psi \equiv 2\zeta$ (see Appendix C).

The application of high-polymer materials with their potentially very high damping factors depends on temperature and on the frequency. Figure 2.14 shows the variation of the damping factor with temperature. The principal frequency dependence of the damping factor is shown

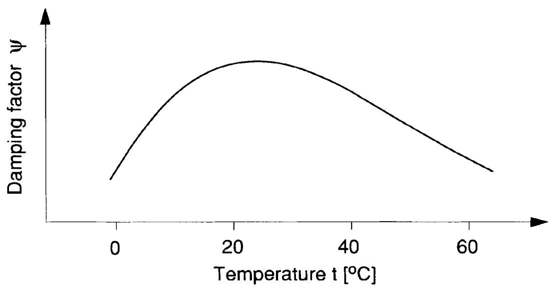


Figure 2.14: Temperature dependence of the damping factor of polymer materials [2.17]

in Figure 2.15. Combinations of different viscoelastic materials can counteract the large decrease of the damping factor on frequency.

For polymer materials, Figure 2.16 shows qualitatively the variation of the modulus of elasticity with temperature.

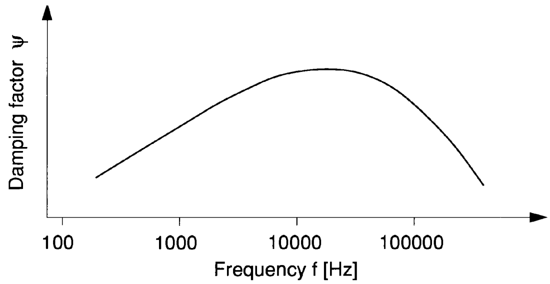


Figure 2.15: Frequency dependence of the damping factor of polymer materials [2.17]

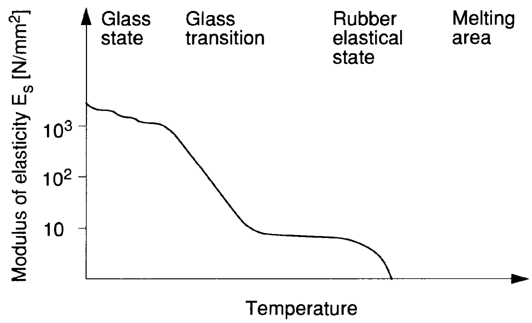


Figure 2.16: Schematic temperature dependence of modulus of elasticity of polymer materials [2.17]

In applying damping to structure-borne sound one distinguishes among

- single layers, applied to one side or both sides of the vibrating body, that utilize damping due to extension and compression;
- inserted (sandwiched) layers, in which an elastomeric material is constrained between the vibrating body and a rigid cover member, and for which shear deformations are utilized.

The combination of these characteristics allows for a wide range of applications, but requires detailed knowledge of the materials employed. Simple design rules can therefore not be established.

A direct assessment of the reduction of sound level of structure-borne sound by an increase in damping is only possible for simple cases. Thus, for purely sinusoidal excitation at discrete resonance frequencies, a reduction of the structure-borne sound pressure level can be expected to be $L = 20 \cdot \log(\psi_1/\psi_2)$ as a result of raising the damping factor from ψ_1 to ψ_2.

Ancillary devices

Ancillary devices can be active or passive. Active devices require added energy for their operation, and also a system of active control. Here, for example, piezo transducers serve for measurement as well as for excitation (for light structures as panels in the high frequency range). A passive ancillary device is, for example, a resonant vibration absorber (also called resonator). The possible measures discussed previously for avoiding or reducing structure-borne sound act over a broad frequency range. Vibration absorbers, on the other hand, are effective only over very small frequency intervals (see Appendix D).

Passive ancillary devices consist of one or more mass-spring-damper systems, depending on the requirements. Undamped vibration absorbers act at one resonance frequency; damped absorbers act over a small frequency band. Shock absorbers are highly damped vibration absorbers. They operate on the principle that they take over the structure-borne energy and thus vibrate themselves; as a result, the main system is quieted through energy extraction.

2.3.7 More advanced design rules

These derive from system analysis. For simple systems one would employ simple elements (bars, plates, etc.) for the mathematical model; more complex systems need in general be modelled by discrete methods such as the finite element method. For more accurate results detailed considerations such as modal analysis [2.20] are only reasonable in the lower frequency ranges up to about 100 Hz, starting with the natural frequency in the order of 1 Hz. Above that range, impedance approaches should be employed.

Besides deterministic calculations, for stochastic excitations correlation methods in the time domain and energy density methods in the frequency domain can be carried out. The input-output relationships are formally the same ones as for deterministic processes, see [2.22]. To carry this a step further, one can try to minimize the structure-borne velocities systematically in the design stage. Literature references for this approach are rare, however.

Structure-borne sound is treated extensively in [2.19], fundamentals are given in [2.24] to [2.28].

2.3.8 Remedial measures

Bars, plates and shells that radiate sound exhibit variations that are too complex for a detailed mathematic treatment. Thus, despite simplified calculations and experience, remedial measures are required relatively often. It is recommended that a stepwise approach be arranged. After initial assessments and completion of the structural system, the dynamic properties should be measured; based on these verified values, sound reduction measures can be applied if the design objectives have not been achieved. It is important that such a planned stepwise procedure be arranged with the building owner right from the start of the planning process.

The remedial measures are the same as those already discussed:

- for machines: changes in operating conditions
- attenuation of structure-borne sound
- damping of structure-borne sound
- add-on devices.

2.4 Ground-transmitted vibrations

G. Klein, J.H. Rainer

2.4.1 Problem description

The excitation of structures through the soil caused by machinery-induced vibration sources outside these structures ("indirect" excitation) is influenced primarily by the transmission properties of the soil. Because of the inhomogeneity of the ground these transmission mechanisms are very complicated. Predictive calculations can therefore only be viewed as *approximations*. By means of on-site experiments, however, the reliability of such results can be improved. An elementary view of the problem can be found in Appendix E.

The intensity and frequency content of the vibrations of a structure induced by ground transmitted vibrations are governed by the excitation source, the transmission path and the properties of the structure itself as a receiver. The multitude of parameters involved precludes an exact solution of the vibration problem. Therefore the effects of pertinent parameters need in general be determined by experience and/or measurements. The transmission medium, i.e. the soil, plays an important part in this process [2.29], [2.30], [2.31].

Four typical questions can be considered:

(1) What is the effect of an *existing* vibration source on a particular *existing* structure?
(2) How can the effect of an *existing* vibration source on a *planned* structure be determined?
(3) How can the effect of a *future* vibration source on an *existing* structure be determined?
(4) How can the *governing vibration parameters* be evaluated for the *design* of industrial and residential neighbourhoods?

It is assumed here that the characteristics of the vibration source are known. Question (1) can then be answered by means of measurements on the existing structure. For an answer to question (2), field measurements are indicated; here the transmission from soil to structure is important. For question (3) the effect of the future source on an existing structure can be evaluated by means of test excitations. For question (4) an assessment can be made once the dynamic characteristics of the transmitting ground are known.

For the cases associated with questions (2), (3) and (4) a preliminary rough assessment can be achieved by means of calculations. Emission, transmission and immission of vibrations are to be investigated (see Figure 2.17). For that purpose the structure and the structural components are modelled by suitable mechanical substitute systems. This analysis consists of determining the natural frequencies, the response spectrum characteristics and the time-varying vibration behaviour of the substitute system. If the boundary conditions cannot be clearly defined then additional cases with external or limited boundary conditions have to be calculated. Of particular significance here is the vibration transmission across the various foundation elements into the structure which in general results in a reduction of vibration amplitudes. Transmission inside the building, on the other hand, can lead to an amplification of vibrations due to near-resonance excitations.

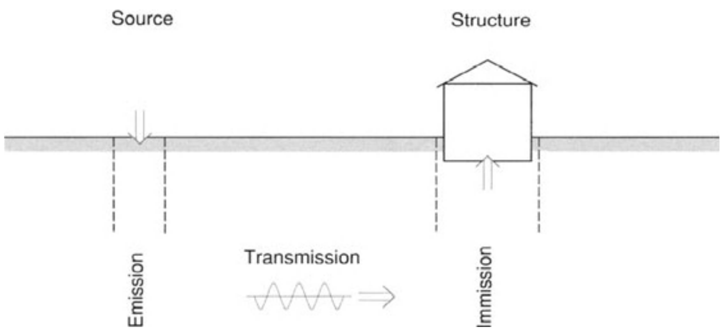


Figure 2.17: System for the propagation of vibrations

To achieve more *realistic results,* vibration measurements and dynamic soil investigations are in all cases necessary. Because in computations frequent encounter of complex boundary conditions arise (such as deformation of properties and layering of the ground, artificial disturbances in the ground and the presence of other structures) the measurement of propagation characteristics of the soil [2.22] can provide significant information for the area to be investigated. This applies particularly to propagation velocities (for soil stiffness) and damping parameters (for amplitude reduction) as a function of frequency. The measured quantity is usually the velocity; it can also be the impedance, that is the ratio of force and velocity (see Appendix A).

2.4.2 Dynamic actions

See the respective sections in Sub-Chapters 2.1, 4.1, 4.2, 4.4.

2.4.3 Structural criteria

a) Natural frequencies

Slabs and walls in a building structure are frequently excited to near-resonance amplitudes. The fundamental frequency of these elements lies between 10 Hz and 30 Hz, whereas the horizontal fundamental frequencies of entire structures often are between 5 Hz and 10 Hz. Some calculation methods are presented in [2.10], but more reliable values can be obtained by vibration tests.

b) Damping

Also for evaluation of damping quantities vibration tests are strongly recommended. Such tests will lead to more reliable values because of the variability of material properties, the effect of finishes and the connections between slabs and walls. But these parameters are generally difficult to quantify.

2.4.4 Effects

Evaluation of the indirect excitation will invariably be targeted towards people or sensitive systems in the structure as they are generally less tolerant to vibrations than the structure itself.

2.4.5 Tolerable values

The discussions of tolerable values in Section 2.1.5 and in Appendices I and J are also valid for this sub-chapter. In addition, [DIN 4150/1] may be considered.

2.4.6 Simple design rules

The following rules are only applicable for simple cases in uniform soils. They give only a rough estimate because of the variability of the soil parameters. In other cases one should study references [2.32], [2.33] or consult an expert.

a) Emission

The vibration energy introduced into the ground by a vibration source propagates in the form of various types of waves (see Appendix E). These are body waves (compressional waves and shear waves) and surface waves (Rayleigh waves) which are excited in varying amounts depending on the type of source and the energy coupling with the ground. The influence of Love-waves can be neglected for uniform soils.

The vibration source can be a point source or a line source. For the propagation of vibrations from point and line sources it is advantageous to separate a near field from a far field. In the near field of a source the energy is mainly transported by body waves. With increasing distance from the source the energy transport takes place by surface waves. The boundary between near field and far field, that is the distance R, can be assumed to be one wave-length λ.

In the case of *stiff foundations* with plan dimensions $a \times b$, where b is always the longer side (e.g. a diesel power machine on its stiff foundation mat) with in-phase excitation at the ground surface, two regions need to be considered that have different propagation characteristics:

- If the propagation direction is perpendicular to the longer side b, the propagation in the near region is similar to the propagation from a line source. From a distance $R = b^2/\lambda$ the propagation is more and more similar to the propagation from a point source.
- If the propagation direction is perpendicular to the shorter side a, the propagation is similar to the propagation of a point source: in the near region according to body waves, from a distance $R = \lambda$ according to surface waves.

In the case of *flexible foundations*, plates, beams as well as other distributed sources (e.g. railway ties) no definitive expressions are available for the distances R. For these cases, however, a decomposition into contributing sources having stiff properties could be considered.

As for sources located near the ground surface (machines, traffic) the surface waves dominate in the far field, the predominant wave length given by $\lambda = v_s/f$ is seen to be a function of the propagation velocity v_s of the shear wave or the surface wave, and the wave frequency f.

b) Transmission

In general the intensity of the ground-transmitted vibrations attenuates with increasing distance from the vibration source. This attenuation is mainly governed by geometric amplitude reduction (geometric or radiation damping) and material damping of the ground.

In the *far field* the attenuation can be calculated by the attenuation laws presented in Appendix E. These attenuation laws are, however, to be viewed only as approximations to the more complex situation that prevails in the real ground. Locally, considerable deviations can occur from this approximate assessment, particularly when the ground is strongly layered, when other structures or discontinuities in the ground are present, or when more vibration sources exist. In these cases additional considerations would be appropriate; in particular, measurements in the field can be very helpful.

Ground water and frozen soil facilitate the propagation of vibrations, particularly of compressional waves. However, changes in the ground water level do not significantly affect the spreading of surface waves.

In the *near field* the approximate relationships presented in Appendix E are not valid. For the prediction of such cases, special analytical and experimental investigations are required.

c) Immission

The intensity of vibrations inside building structures depends on various parameters such as: duration and frequency of the vibration signal, mass and stiffness of the building or the building component, foundation and type of soil, damping [2.2], [2.34].

The transmission of vibrations from the ground into the building foundation results in an attenuation of amplitudes. For smaller buildings the particle velocities measured in the ground are reduced to approximately one half, for larger buildings to approximately one third to one quarter. The propagation of vibrations into floor slabs or other building components is usually accompanied by an increase in vibration amplitudes due to near-resonance excitations. In extreme cases these amplitudes can reach 10 to 20 times the values of the reduced foundation amplitude. For purposes of prediction an amplification of 5 times can be assumed; however, the scatter is quite large. The resulting values are then used for comparison with tolerable values from codes and other values from personal experience.

2.4.7 More advanced design rules

More accurate investigations for the propagation of vibrations are based on modelling the ground by means of finite element and finite difference methods [2.35]. Difficulties encountered are those of the boundary of the finite domain because of energy radiation and the unknown dynamic characteristics of the ground. Boundary element methods have also proved to be useful. It should, however, be carefully considered whether for practical problems an expenditure on such methods is cost effective and, above all, whether the reliability of the results obtained is compatible with the effort expended.

2.4.8 Remedial measures

If the vibrations in a building from ground-induced excitations are larger than the tolerable values, then remedial measures need to be adopted. These can be carried out at the source, in the surrounding medium, or at the affected building itself. In any case a systematic approach is recommended that permits an assessment of the effectiveness of each step in this procedure.

The most effective method is always a *reduction* of the energy at the *excitation source*. This can be achieved by a reduction in machine vibrations, as for example, by improved balancing methods (see e.g. [VDI 2060]). Furthermore the excitation source can be treated as an entity by considering the process of isolation, damping and attenuation with the objective of reducing the propagated vibrations (see Sub-Chapter 2.1 and e.g. [2.36]). Finally, a modification of the foundation of the building that contains excitation machinery can lead to improvements in the vibration environment.

In the transmitting soil the vibration amplitudes can be partially attenuated by *artificial barriers* such as piles, open trenches, bentonite trenches, rigid walls [2.37]. The depth of a trench needs to be approximately one wavelength of the offending frequency component. For stiff walls greater reductions are achieved with increasing product of wall thickness and depth.

Measures of vibration reduction are most difficult to carry out at the affected building. These measures are also very difficult to implement from a legal point of view. A substantial reduction is only rarely possible. Most likely one can achieve an attenuation effect at the interface between soil and foundation. Sensitive stationary equipment in the affected building can, however, again be decoupled by means of the process of isolation, damping and attenuation (isolated and controlled systems; see Sub-Chapter 2.1).

References to Chapter 2

[2.1] Natke H.G., Thiede R., Elmer K.: "Untersuchungen der von Webereien ausgehenden Schwingungsemissionen und Hinweise zu Websaal-Bauplanungen". ("Investigations of Vibration Emissions emanating from Weaving Mills and Hints in Planning the Construction of Weaving Sheds"). Verband der Nord-Westdeutschen Textilindustrie, Schriftenreihe Heft 66, Münster, 1985.

[2.2] Bachmann H., Ammann W.: "Vibrations in Structures - Induced by Man and Machines". Structural Engineering Documents No.3e, International Association of Bridge and Structural Engineering (IABSE), Zürich, 1987.

[2.3] Korenev B.G., Rabinovic I.M.: "Baudynamik Handbuch" ("Handbook on Structural Dynamics"). VEB Verlag für Bauwesen, Berlin, 1980.

[2.4] Müller F.P., Keintzel E., Charlier H.: "Dynamische Probleme im Stahlbetonbau. Teil 1: Der Baustoff Stahlbeton unter dynamischer Beanspruchung" ("Dynamic Problems in Structural Concrete Design. Part 1: Reinforced Concrete under Dynamic Excitation"). Deutscher Ausschuss für Stahlbeton, Heft 342, 1983.

[2.5] Clough R.W., Penzien J.: "Dynamics of Structures". McGraw-Hill, New York, 1975.

[2.6] Makhult M.: "Machine Support: Design based on Vibration Calculus". Collet's Ltd., London and Wellingborough, 1977.

[2.7] Major A.: "Dynamics in Civil Engineering". Vol. I-IV, Akademiai Kiado, Budapest, 1980.

[2.8] Macinante J.A.: "Seismic Mountings for Vibration Isolation". John Wiley & Sons, New York, 1984.

[2.9] Müller F.P.: "Berechnung und Konstruktion von Glockentürmen" ("Calculation and Construction of Bell Towers"). Wilhelm Ernst & Sohn, Berlin, 1968.

[2.10] Eibl J., Henseleit O., Schlüter F.-H.: "Baudynamik" ("Dynamics of Structures"). Betonkalender 1988, Teil II, S. 665 - 774. Wilhelm Ernst & Sohn, Berlin, München, Düsseldorf, 1988.

[2.11] Natke H.G.: "Baudynamik - Einführung in die Dynamik mit Anwendungen aus dem Bauwesen" ("Structural Dynamics - An Introduction to Dynamics with Applications in Civil Engineering"). Leitfäden für Angewandte Mathematik und Mechanik (LAMM-Reihe), B.G. Teubner, 1989.

[2.12] Baumgart F.: "Ein Beitrag zur Ermittlung der Erregerkräfte bei Glockentürmen", ("A Contribution to the Determination of Excitation Forces of Bell Towers"). Zeitschrift für Angewandte Mathematik und Mechanik (ZAMM) 51, T 183 - T 185, 1971.

[2.13] Baumgart F.: "Ermittlung der Erregerkräfte bei Glockentürmen" ("Determination of Excitation Forces of Bell Towers"). Der Bauingenieur, Heft 12, 1970.

[2.14] Savci M.: "Ein Beitrag zur Bekämpfung von Glockenturmschwingungen" ("A Contribution to Vibration Abatement of Bell Towers"). Dissertation, Rheinisch Westfälische Technische Hochschule (RWTH) Aachen, 1969.

[2.15] Kurze U.J., Dietz P.: "Lärmarm konstruieren XIII, Schalltechnische Regeln und Konstruktionsmethodik" ("Minimum Noise Design XIII, Acoustical Rules and Design Methodology"). Schriftenreihe der Bundesanstalt für Arbeitsschutz - Forschung - Fb 424, 1985.

[2.16] Kurze U.J.: "Lärmarm konstruieren XII, Mechanische Impedanzen", ("Minimum Noise Design XII, Mechanical Impedances"). Schriftenreihe der Bundesanstalt für Arbeitsschutz - Forschung - Fb 398, 1985.

[2.17] Gahlau H.: "Geräuschminderung durch Werkstoffe und Systeme" ("Noise Reduction by Materials and Systems"). Expert-Verlag, Sindelfingen, 1986.

[2.18] TALärm: "Technische Anleitung zum Schutz gegen Lärm" ("Technical Instruction for Noise Protection"), Beilage zum Bundesanzeiger (BAnz) Nr. 137 vom 26.07.68.

[2.19] Cremer L., Heckl M.,: "Körperschall: Physikalische Grundlagen und technische Anwendungen" ("Structure-borne Sound: Physical Fundamentals and Technical Applications"). Springer-Verlag, Berlin, 1967.

[2.20] Natke H.G.: "Modalanalyse, Fortschritte der Akustik" ("Modal Analysis"). Deutsche Arbeitsgemeinschaft für Akustik (DAGA) 1987, S.137 - 155.

[2.21] Henn H., Sinamhari G.: "Ingenieurakustik" ("Acoustics for Engineers"). Friedrich Vieweg & Sohn, Wiesbaden, 1984.

[2.22] Natke H.G.: "Einführung in Theorie und Praxis der Zeitreihen- und Modalanalyse, Identifikation schwingungsfähiger elastomechanischer Systeme" ("Introduction to The Theory and Practice of Timeseries and Modal Analysis, and Identification of Vibrating Elastomechanical Systems"). Friedrich Vieweg & Sohn, Braunschweig, 2. Auflage 1988.

[2.23] Cremer L., Heckl M., Ungar E.E.: "Structure-Borne Sound". Springer Verlag, Berlin and New York, 1973.

[2.24] Morse P.M.: "Vibration and Sound". 2nd Ed. McGraw-Hill, New York, 1948, reprinted by the Acoustical Society of America, 1981.

[2.25] Pierce A.D.: "Acoustics: An Introduction to its Physical Principles and Applications". McGraw-Hill, New York, 1981.

[2.26] Beranek L.L.: "Noise and Vibration contro'". McGraw-Hill, New York, 1971.

[2.27] Skudrzyk E.: "Simple and Complex Vibratory Systems". Pennsylvania State University Press, 1968.

[2.28] White R.G., Walker J.G.: "Noise and Vibration". Ellis Horwood Ltd., John Wiley & Sons, New York, Brisbane, Chichester, Toronto, 1982.

[2.29] Richart F.E., Hall J.R., Woods R.D.: "Vibrations of Soils and Foundations". Prentice-Hall, Englewood Cliffs, New Jersey, 1970.

[2.30] Studer J., Ziegler A.: "Bodendynamik" ("Soil Dynammics"). Springer, Berlin, 1986.

[2.31] Haupt W.: "Bodendynamik" ("Soil Dynamics"). Friedrich Vieweg & Sohn, Braunschweig, 1986.

[2.32] Rücker W.: "Schwingungsausbreitung im Untergrund" ("Vibration Propagation in Subsoil"). Bautechnik 66 Heft 10, 1989.

[2.33] Haupt W., Herrmann R.: "Dynamische Bodenkennwerte" ("Dynamic Soil Characteristics"). Querschnittsbericht 1986, Landesgewerbeanstalt Bayern, Nürnberg, Heft 48.

[2.34] Umemura H., Takizawa H.: "Dynamic Response of Reinforced Concrete Buildings". Structural Engineering Documents No. 2, International Association for Bridge and Structural Engineering (IABSE), Zürich, 1982.

[2.35] Haupt W.: "Numerical Methods for the Computation of Steady-state Harmonic Wave Fields". Proceedings "Dynamical Methods in Soil and Rock Mechanics", Karlsruhe, 5-16 September 1977, Balkema, Rotterdam, 1978.

[2.36] Harris C.M., Crede C.E.: "Shock and Vibration Handbook". McGraw- Hill, New York, 1976.

[2.37] Dolling H.-J.: "Die Abschirmung von Erschütterungen durch Bodenschlitze" ("Screening of Vibrations by Soil-trenches"). Bautechnik 67, Hefte 5 und 6, 1970.

[2.38] Yang C.Y.: "Random Vibrations of Structures". John Wiley & Sons,New York, 1986.

3 Wind-induced vibrations

G. Hirsch, H. Bachmann

This chapter deals with wind-induced structural vibrations from the point of view of structural engineering. Basically, no civil engineering structure is safe from wind loading effects. Of critical importance are the non-stationary characteristics of natural wind and the dynamic properties of the structure it acts upon. Simple rules for the construction industry can only be defined by making radical simplifying assumptions. For complex structures it is necessary to resort to the results of wind tunnel tests.

This chapter deals with forced vibrations due to

- gust actions (turbulence effects), in the wind direction
- buffeting, in the wind direction
- vortex shedding, (without "lock-in" effect) in the across-wind direction

as well as across-wind self-induced vibrations (aeroelasticity) due to

- vortex shedding ("lock-in" effect)
- galloping
- flutter (bridge flutter).

Wind-induced vibrations may strongly affect either the serviceability or the fatigue behaviour and safety of structures or both, depending on the type of structure.

In this chapter wind-induced vibrations for the following types of structure are treated in the following sub-chapters:

3.1 Buildings
3.2 Towers
3.3 Chimneys and Masts
3.4 Guyed Masts
3.5 Pylons
3.6 Suspension and Cable-Stayed Bridges
3.7 Cantilevered Roofs

The fundamentals of the dynamic effects of wind loading, which are required for several types of structure, are presented in Appendix H. Further basic theory is to be found in the other appendices. Wind-specific aspects are covered in several sub-chapters.

3.1 Buildings

G. Hirsch, H. Bachmann

3.1.1 Problem description

Although the effects of natural wind on structures are in general of a dynamic nature, not all structures react with pronounced vibrational behaviour, i.e. the mass-inertia forces and the damping forces do not play a significant role. A classification of "vibration-sensitive" ("flexible") and "vibration-insensitive" ("rigid") structures is possible by defining certain limits. A possible criterion for this is given in the draft of the Eurocode Wind [EC 9/1990], according to which a structure subjected to wind is defined as "rigid" if the dynamic reaction to the effects of gusts or turbulence does not exceed 10% of the reaction to the static force.

Figure 3.1 shows corresponding limits for buildings with different heights and widths together with damping values, whereby the concept of stochastic wind loading for excitation in the wind direction was followed (see Appendix H) and the fundamental frequency was determined from the approximate formula given in [3.1],

$$f_e = 0.4\,(100/h)^{1.6}\ \text{[Hz]} \tag{3.1}$$

where h = height [m].

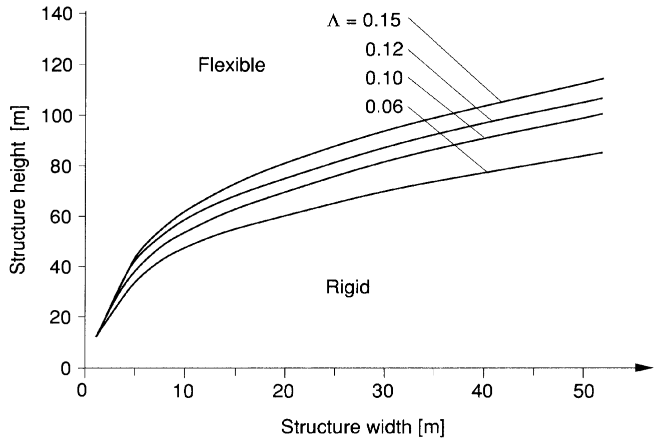


Figure 3.1: Demarcation line between "flexible" and "rigid" structures (Λ = logarithmic decrement)

One may draw the general conclusion that buildings of height h greater than about 50 m may be regarded as "flexible" in the above sense. For building heights between 50 and ~100 m a calculation can be carried out according to Appendix H.2.2, which includes dynamic effects.

At greater heights more exact investigations may be required, especially using suitable wind tunnels (i.e. boundary layer tunnels capable of simulating the turbulence at the specific site).

Buildings undergoing wind-induced vibrations suffer above all with respect to serviceability, since user comfort is reduced. The movements, however, are not great enough to impair the structure's safety (with the exception perhaps of local damage to facades and roofs).

3.1.2 Dynamic actions

For ordinary or high-rise buildings the dynamic forces in the wind direction due to gusts and turbulence of natural wind must be considered. The special geometry of such structures is so that vortex-induced vibrations like galloping and flutter are in general of no real significance.

3.1.3 Structural criteria

a) Natural frequencies

The fundamental frequency of a building depends above all on its height. An estimate of the fundamental frequency erring on the flexible side is possible using Equation (3.1). The investigation [3.2] has shown that exact calculations, even with the aid of complicated computer programs, when compared with observations on the original structure, do not provide better natural frequencies than those obtained with the approximate formulas, Figure 3.2 shows the fundamental frequency of tall buildings as a function of height using a formula from [3.2], which is similar to that given in Equation (3.1), namely:

$$f_e = 46/h \text{ [Hz]} \tag{3.2}$$

where h = height [m].

b) Damping

Ordinary and tall buildings exhibit relatively large structural damping values mainly due to non-loadbearing elements (see Appendix C). Table 3.1 presents suggested values in terms of equivalent viscous damping ζ based on the draft [ISO TC 98/SC 3/WG 2].

	damping ratio ζ		
Construction type	min.	mean	max.
Tall buildings (h > ~100 m):			
• Reinforced concrete	0.010	0.015	0.020
• Steel	0.007	0.010	0.013
Buildings (h ~50 m):			
• Reinforced concrete	0.020	0.025	0.030
• Steel	0.015	0.020	0.025

Table 3.1: Common values of damping ratio ζ for buildings

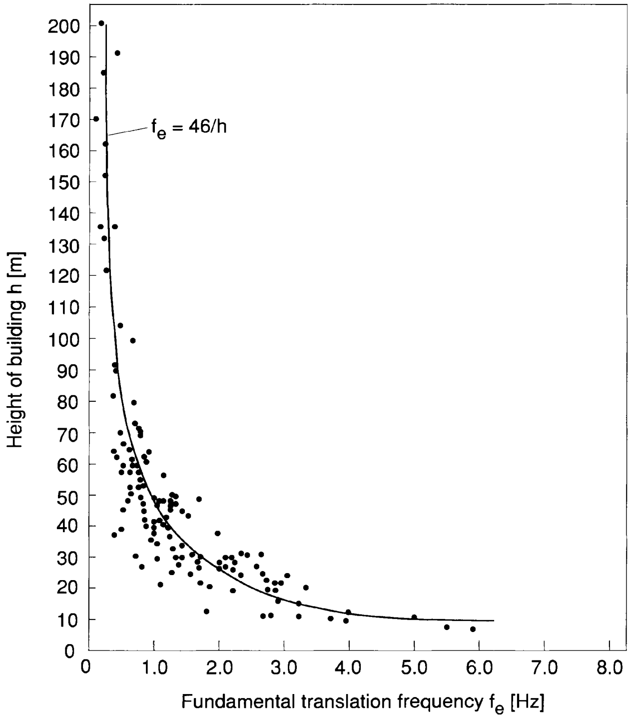


Figure 3.2: Fundamental frequency of tall buildings

c) Stiffness

The stiffness of the structure of tall buildings can be decisive for serviceability, i.e. for the comfort of occupants. This criterion usually governs over the design requirements of strength and stability.

3.1.4 Effects

The effects of low-frequency vibrations on people are those of annoyance, apprehension regarding the structural safety of the building, loss of mental concentration, and an unwell feeling resembling sea-sickness. None of these effects are considered "harmful" to people, but because of the annoyance factor, buildings with vibration problems may receive more complaints than others. There are no known cases, however, where such vibrations have impaired the structural safety of a building.

3.1.5 Tolerable values

In Figure 3.3 for low-frequency wind-induced building vibration the perception and judgment of persons are given in terms of limiting values of acceleration as a function of frequency.

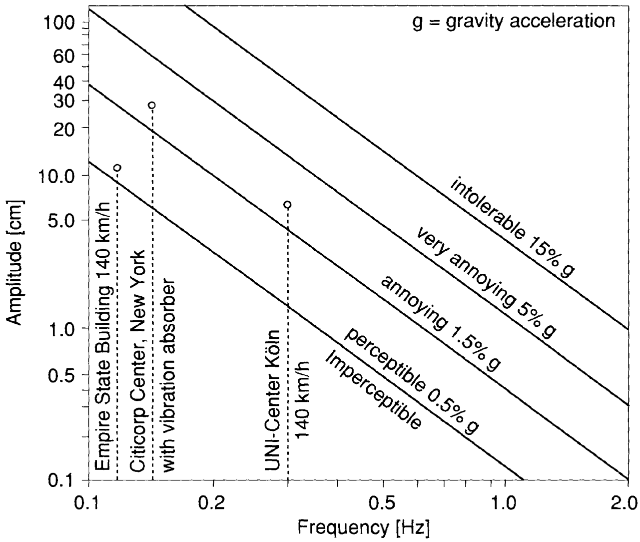


Figure 3.3: Human perception of building vibration due to wind [3.3]

In [H.2] the following acceleration values may be used:

Perception	acceleration limits
Imperceptible	$a < 0.005g$
Perceptible	$0.005g < a < 0.015g$
Annoying	$0.015g < a < 0.05g$
Very Annoying	$0.05g < a < 0.15g$
Intolerable	$a > 0.15g$

Obviously, the frequency of occurrence of a particular acceleration is of great importance. For example, an acceleration of 1.5% g on the top floor of a tall building, which is normally found to be disturbing, may be acceptable with a recurrence period of some years.

Further information is given in Appendix I.

3.1.6 Simple design rules

The approximations in Equations (3.1) and (3.2) show that the fundamental frequency of tall buildings - no matter what the type of construction - depends above all on the building height. If additional stiffness is introduced, then inevitably the mass is increased, so that in general the "dynamic stiffness" remains more or less constant. A distinct advantage in the application of such approximations for the fundamental frequency lies in the fact that a fairly reliable estimate of the fundamental frequency for the final configuration can already be given when only the main dimensions of the structure are known even though the complete design of the structure has not yet been carried out. The initial calculations of the effect of gusts can thus be made with some confidence.

To determine the force in the wind direction a method of estimating an equivalent stochastic force and a maximum acceleration is given in Appendix H.2.2, which corresponds more or less to today's state of the art in the international standards (e.g. Eurocode, ISO). By applying different national standards, however, rather different conclusions concerning the design of the structure could be reached. This is due to the fact that until now dynamic wind effects have not been sufficiently researched to permit a unified understanding of the complex aerodynamic and structural mechanisms to be found.

3.1.7 More advanced design rules

Detailed investigations are possible with the help of the literature given in Appendix H.

3.1.8 Remedial measures

To influence the wind-induced dynamic forces in the building various passive (and lately also active) vibration control measures have been implemented to reduce vibration intensity. The more important of these are mentioned in the following.

a) Installation of damping elements

The World Trade Centre in New York has a number of damping elements as shown in Figure 3.4.

b) Vibration absorbers

The first tall building in which a tuned vibration absorber was installed for reducing the wind-induced forces was the 280 m high Citicorp Centre in New York. The system is in fact a semi-active damper. The mass of the hydrostatically supported system amounts to 410,000 kg [H.2]. Figure 3.5 shows its principle of operation. (However, since in theory damping exhibits no significant effect for vibrations in the wind direction, the effectiveness of application of the damper is a matter of contention. See Appendix H.2).

Today, it is mainly in Japan that completely active vibration control is applied for tall buildings subject to both wind and earthquake loading. The principle of active vibration control is as follows: the motion of the structure is measured, compared to a reference value (practically zero) and brought to rest by inertia forces resulting from the controlled motions of added masses or by using active tendon control. An overview of the state of knowledge of active vibration control is given in [3.4] and [3.5].

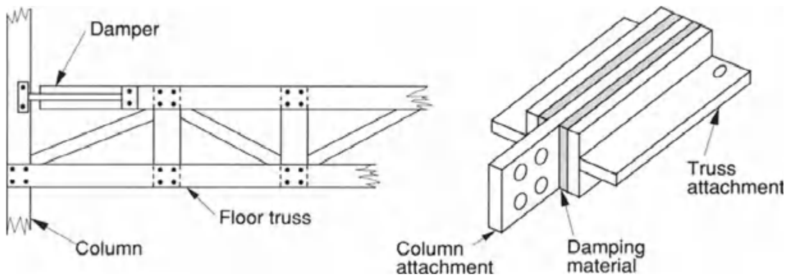


Figure 3.4: Friction dampers in the load bearing structure of the World Trade Centre in Manhattan (New York) [H.2]

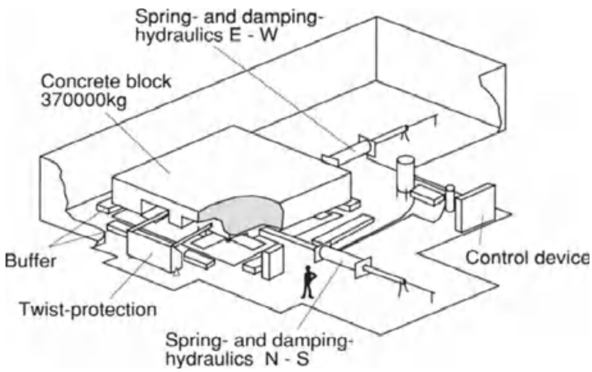


Figure 3.5: Vibration absorber, Citicorp Centre, New York

3.2 Towers

G. Hirsch, H. Bachmann

3.2.1 Problem description

Tower-like structures are understood, in general, to be slender, tall structures (television towers, lookout towers, etc.). Figure 3.6 shows a few examples of telecommunication towers, which can be seen to exhibit some differences in structural form. Often the basic contour shows distinct structural components jutting out, which can significantly influence the dynamic behaviour of the tower under wind loading. In addition, different cantilever systems (e.g. to support an antenna) may be built onto a tower, which can be incorporated as a substructure into the total structural system, but nevertheless still exhibit their own local behaviour. Examples of such added cantilever systems are shown in Figure 3.6, b and c.

A slender residential building may also be classified as a tower. Nowadays this often includes hotels whose structures consist of tall buildings of cylindrical form. A bell tower, despite its name, does not belong to the category of structures considered here but instead is treated in Sub-Chapter 3.1 since the major dynamic excitation derives from the motion of the bells and not from wind.

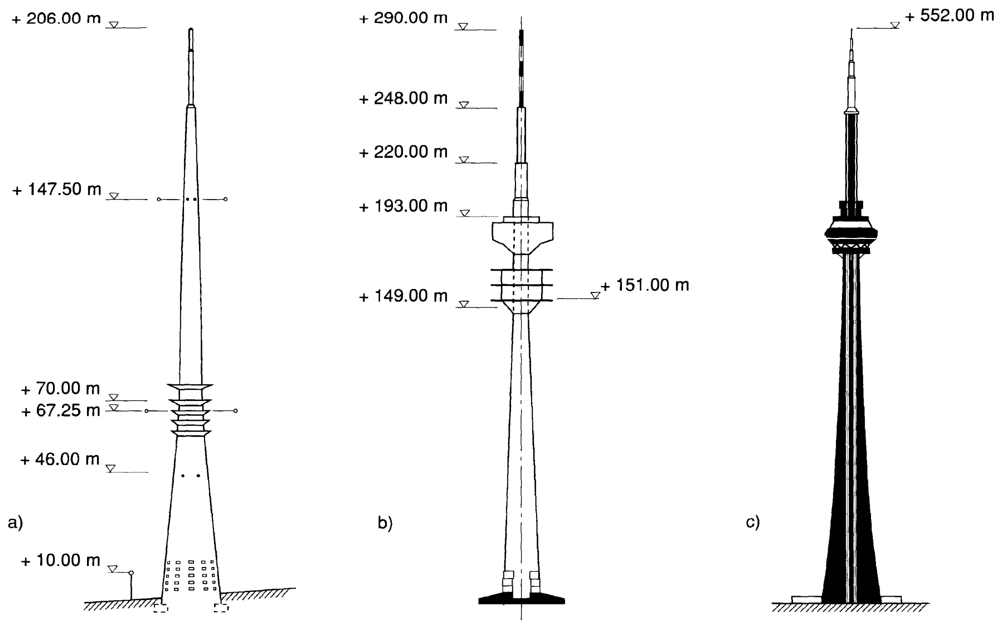


Figure 3.6: Telecommunication towers: a) Hornigsrinde, Germany [3.6], b) Munich [3.7], c) CN Tower, Toronto [3.8]

The vibrations of tower-like structures in natural wind are characterized by a random (stochastic) motion, whereby it is observed that the structure vibrates not only in the wind direction, but also normal to it. Thus dynamic effects are superimposed due to gusts in the wind direction and to vortex-induced vibrations in the across-wind direction. Also in the case of vortex resonance, vibrations of towers in general are not exactly harmonic, which may be attributed to the relatively large structural damping and the associated mass distribution (see Appendix H.4 and Figure H.12a).

Wind-induced vibrations of towers result above all in a reduction of serviceability. Persons at places of work or in restaurants located high up (because of the spectacular view) can feel uncomfortable. For telecommunication towers the antennae can exhibit large deviations from the static position. The safety of the structure, however, is rarely endangered.

3.2.2 Dynamic actions

Figure 3.7 shows a typical dynamic behaviour for a tower subjected to wind loading. Usually, gust-induced vibrations in the wind direction predominate, especially those at the fundamental bending frequency. Vibrations also occur in the across-wind direction due to vortex-shedding, but do not govern the design. Figure 3.7 is also characteristic for chimneys built of reinforced concrete or masonry, which from the point of view of wind engineering are likewise classified as towers, and are treated together with highly vibration-sensitive steel chimneys in Sub-Chapter 3.3.

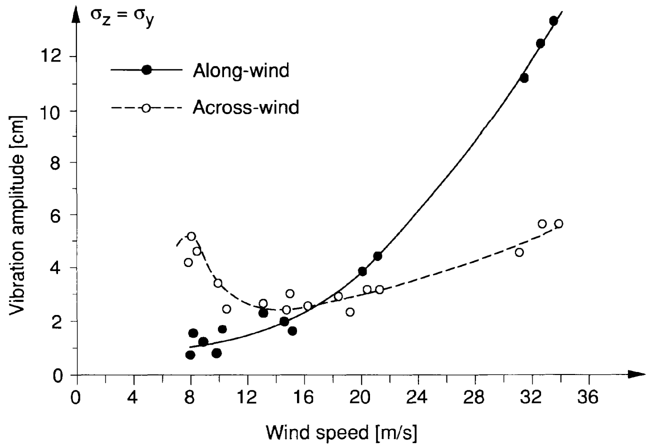


Figure 3.7: Vibration in a tower-like structure in the along-wind and across-wind directions

3.2.3 Structural criteria

a) Natural frequencies

Slender towers vibrate relatively slowly in their fundamental mode. They exhibit fundamental frequencies as low as about 0.15 Hz, with a corresponding period of ~7 s. The calculation of the bending frequency is best carried out by Rayleigh's method:

$$f_e = \frac{1}{2\pi}\sqrt{\frac{\Sigma m_j \cdot g \cdot y_j}{\Sigma m_j \cdot y_j^2}} \text{ [Hz]} \tag{3.3}$$

where m_j = mass of j-th discretized section of the tower
y_j = deflection caused by the applied horizontal inertia force $m_j \cdot g$

In the calculation of the deflection it is necessary to consider the possible flexibility of the tower's foundation. The deformations due to the elasticity of the tower structure have to be added to the contribution due to a rotation of the foundation in the plane of bending. The influence of flexible supports can be considerable.

An improved calculation using Equation (3.3) may be achieved by replacing in a second iteration the term $m_j \cdot g$ by the mass-inertia forces obtained from the first approximation:

$$T_j = m_j \cdot y_j \cdot (2 \cdot \pi \cdot f_e)^2 \tag{3.4}$$

Thereby one obtains a new $y_j{}^*$, with which the calculation using Equation (3.3) can be repeated. Often there is no significant deviation from the first approximation, but in some special cases (e.g. as in Figure 3.6b) marked differences can be obtained. Then the calculation has to be repeated until there is agreement between successive results.

It is possible that cantilevered portions of a tower structure deform primarily in flexure. In such a case the rotation of the added mass in the plane of bending has to be considered. This is given by the slope angle at the height of the mass of the deflected shape. The rotating mass Θ_j contributes an angular momentum in the form of an "added mass". The denominator in Equation (3.3) must then be extended to include the rotational energy $\Sigma\,(\Theta_j \cdot y_j'^2)$, where y'_j is the slope of the deflection curve.

In general, only the fundamental bending mode is considered for tower-like structures, but cases may arise in which e.g. the second mode is significant.

While Rayleigh's method is sufficient for the calculation of the fundamental bending frequency, for the determination of higher frequencies a computer program (e.g. employing the finite element method) or a classical approach (e.g. matrix iteration) may be required.

b) Damping

The damping of towers derives mainly from material damping and a possible radiation damping into the ground (see Appendix C). Values of equivalent viscous damping in the form of the logarithmic decrement Λ are given in Table 3.2.

The influence of damping is not as critical for tower-like concrete structures as it is for steel chimneys. The transverse vibrations, for which damping plays an important role, are also influenced by the distribution of masses, which for concrete towers is fairly favourable.

	Logarithmic decrement Λ		
Construction type	min.	mean	max.
Reinforced concrete - uncracked - cracked	 0.03 0.08	 0.04 0.10	 0.05 0.12

Table 3.2: Common values of logarithmic decrement Λ for slender concrete towers

c) Stiffness

The required stiffness of the tower structure is mainly determined by serviceability (comfort of persons on the observation platform, antenna movements for telecommunication towers, etc.).

3.2.4 Effects

Section 3.1.4 is applicable.

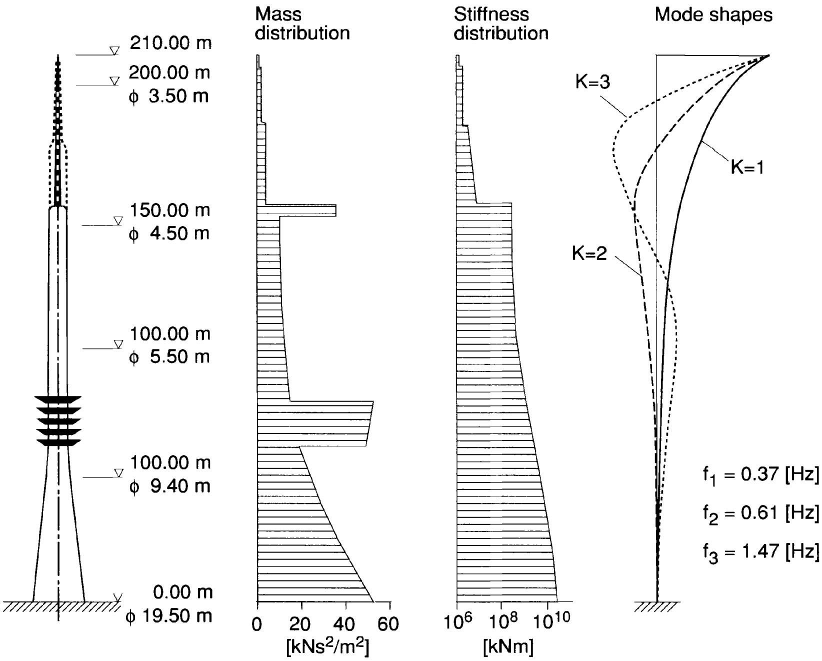


Figure 3.8: Telecommunications tower with mass distribution, stiffness distribution and natural modes of vibration [3.6]

3.2.5 Tolerable values

Section 3.1.5 is applicable.

Tolerable values of vibration velocity for evaluating wind effects on the structure are given in [DIN 4150/3] (see also Appendix J).

Antennae mounted on towers represent a special case. For their assessment the dynamic tilt angle can be decisive. Figure 3.8 shows that the fundamental bending mode (K = 1) may be particularly significant, but also in higher modes (K = 2 and K = 3) the rotation of the antenna can be very large, depending on the modal response. The permissible value of antenna rotation must be specified from case to case.

3.2.6 Simple design rules

Firstly, the fundamental frequency of the tower has to be calculated using Equation (3.3) above.

The gust-induced force corresponding to vibration in the wind direction can be calculated according to Appendix H.2.2 with the aid of an equivalent stochastic force. A simplified, and for towers more suitable, approximate formula for the gust factor is given in [DIN 1056] or in the draft of DIN 4133, Appendix A (1991):

$$\varphi = \eta \cdot \varphi_o \tag{3.5}$$

where the size factor $\eta = 1.05 - h/1000$ (for towers higher than 50 m) and the basic gust factor:

$$\varphi_o = 1 + (0.042 \cdot T - 0.0019 \cdot T^2) \cdot \Lambda^{-0.63} \tag{3.6}$$

where $T = 1/f_e$ = fundamental period
Λ = logarithmic decrement ($\Lambda \approx \zeta \cdot 2\pi$)

It may be seen that the "dynamic stiffness" is hardly influenced by the damping. At a vibration frequency of 1 Hz, doubling the logarithmic decrement from 0.1 to 0.2 results in a reduction of the basic gust factor by only 5%. The reduction of the dynamic displacement and increase of dynamic stiffness would be correspondingly low.

The equivalent wind force given by Equation (H.4) can be determined from Equations (3.5) and (3.6) above. The vibration in the across-wind direction can be calculated as shown in Appendix H.4.1. Another possibility is given in Appendix A of [DIN 4133] (draft 1991).

3.2.7 More advanced design rules

More detailed investigations, as for example to determine antenna rotation in telecommunication towers etc., requires the calculation of the natural frequencies and mode shapes for several modes (3 modes shown in Figure 3.8) with the aid of a finite element program as well as carrying out a modal analysis for the selected natural frequencies and mode shapes. The wind action has to be applied as, for example, in [3.17].

3.2.8 Remedial measures

For towers the use of a vibration absorber (see Appendix D) can be very economical. For telecommunication towers such a measure can be applied to the antenna. The vibration absorber has to be tuned to the predominant frequency of the sub-system "antenna construction". For the case shown in Figure 3.8 a vibration absorber can be attached to the top of the antenna whose frequency lies between the frequencies of K2 and K3. A much better result can be achieved if two vibration absorbers tuned to the frequencies of K2 and K3 are used. When designing vibration absorbers the mass of the main system is that of the tower's modal mass (generalized mass) at the position of the vibration absorber:

$$m_s = \sum_{j=1}^{n} m_j \cdot y_j^2 \tag{3.7}$$

where m_s = modal mass of the structure
n = number of discretized sections of the tower
m_j = mass of the j-th discretized section of the tower
y_j = normalized (dimensionless) modal displacement (of the second or third mode according to Figure 3.8).

It is important to note that for wind-induced vibrations the classical vibration case according to Den Hartog (Appendix D) is not present. The excitation force is not independent of the excitation frequency, but increases in proportion to the square of the excitation frequency (i.e. to the square of the wind speed). Thus with respect to damping a different optimization from that given in Appendix D has to be carried out [3.10]. The optimum natural frequency f_t of the added system is the same as for the classical case, namely

$$f_t = f_s \cdot \frac{1}{\sqrt{1 + m_t/m_s}} \tag{3.8}$$

where f_t = optimum natural frequency of the vibration absorber
f_s = structure's natural frequency to be absorbed
m_s = modal mass of the structure
m_t = mass of the vibration absorber

The optimum damping, on the other hand, is

$$\zeta_{opt} = \sqrt{\frac{3 \cdot (m_t/m_s)}{(1 + m_t/m_s) \cdot (2 + m_t/m_s)}} \tag{3.9}$$

Further details with regard to vibration absorbers for tower-like structures are dealt with in Sub-Chapter 3.3 on chimneys, since for such structures the use of added systems is especially relevant.

3.3 Chimneys and Masts

G. Hirsch, H. Bachmann

3.3.1 Problem description

Free-standing chimneys and masts that are not guyed are similar to towers from the point of view of wind-induced vibrations. This is true especially for the dynamic forces induced in such structures by gust action in the wind direction. The vibrations connected with vortex shedding transverse to the wind direction, however, can be more important. Particularly sensitive in this respect are steel chimneys (welded construction, not insulated or lined with masonry, fixed base). Not the very high ones, but particularly the steel chimneys of about 30 to 40 m height are endangered. Therefore these structures are given particular attention in the following.

Masts are often built as composite structures consisting of components of different materials and geometries (e.g. a glass-fibre tube on a lattice tower), so that a dynamic analysis of the complete system is necessary. This requires an interdisciplinary cooperation between the constructors of the mast and the antenna, as is the case e.g. for the design of a radio transmission tower. Tubular masts of circular section for carrying antennae have to be treated in the first instance as steel chimneys, whereby if external antennae are present, additional vibration problems may arise (e.g. galloping instability if iced-up).

Vibrations of chimneys and masts can lead to structural safety (fatigue) and serviceability problems. In the case of antenna masts, e.g. for telecommunication towers, excessive deviation of the antennae from their normal position can result.

3.3.2 Dynamic actions

The structural configuration is important for the dynamic effect of wind on chimneys and masts. Figures 3.9 to 3.12 illustrate this.

The normal case is that of a freestanding chimney of constant section (Figure 3.9a), while the stepped form of construction is less common (Figure 3.9b). In addition, chimneys in particular can be present in groups or rows (Figure 3.9c), whereby the distance of separation a in relation to the diameter d is important with respect to interference effects (mutual aerodynamic influence).

The tubular structure of chimneys may be provided with external insulation and thus be itself in direct contact with the smoke emitted, or it may be insulated internally (Figure 3.10a and b). The tubular structure may also have multiple layer flues, which affect the vibrating mass to a greater extent than the stiffness of the system (Figure 3.10c). Moreover, the structural damping is greater than for a simple tubular structure. Horizontal connection of chimneys at various heights is also possible (Figure 3.10d).

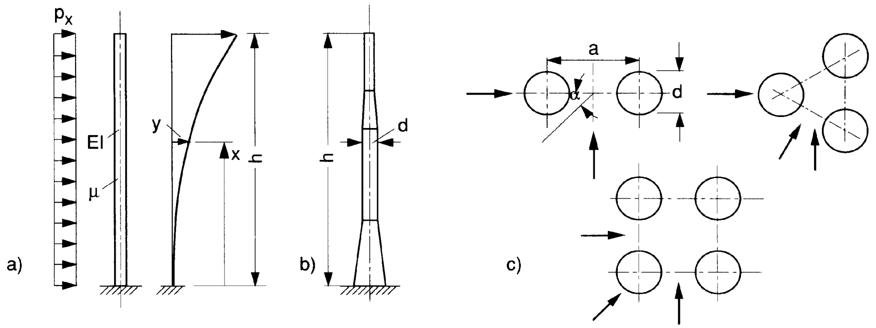


Figure 3.9: Chimney as structural system: a) dynamic wind action and deflected shape, b) stepped chimney, c) group arrangements

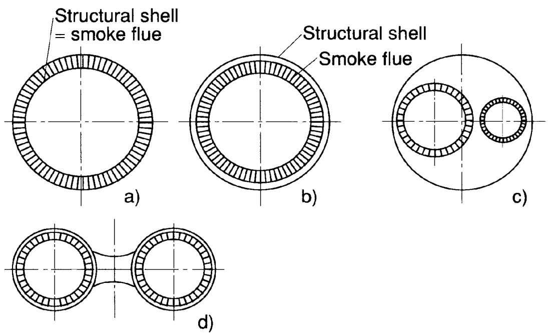


Figure 3.10: Chimney construction: a) with internal insulation, b) with additional smoke tube, c) multi-flues, d) connected chimneys

There are also chimneys which are fixed to a base or supporting structure (Figure 3.11). The base structure forms part of the system as a whole, whereby it may happen that a very stiff chimney is attached to a relatively flexible supporting structure. The wind excitation (due to vortex shedding) then acts in some circumstances mainly on the "rigid" tube. The dynamic response, however, is mainly governed by the flexibility and damping of the structure.

For a telecommunication mast, Figure 3.12 shows a "chimney"-antenna of fibre-reinforced plastic construction on top of a lattice tower which acts in the abovementioned sense as a base structure. The base structure is acted upon directly by the wind (gust action) and responds dynamically to the vortex-induced vibrations of the attached cylinder.

Massive chimneys (reinforced concrete construction) are in general relatively insensitive to vortex-induced vibrations due to their high structural damping. Thus, e.g., according to [DIN 1056] the dynamic force for this effect does not have to be investigated. If the chimney is fixed to a supporting structure, however, the dynamic force has to be determined for the design of this structure. For this purpose other codes (e.g. [DIN 4133], Appendix A, draft 1991) have to be consulted.

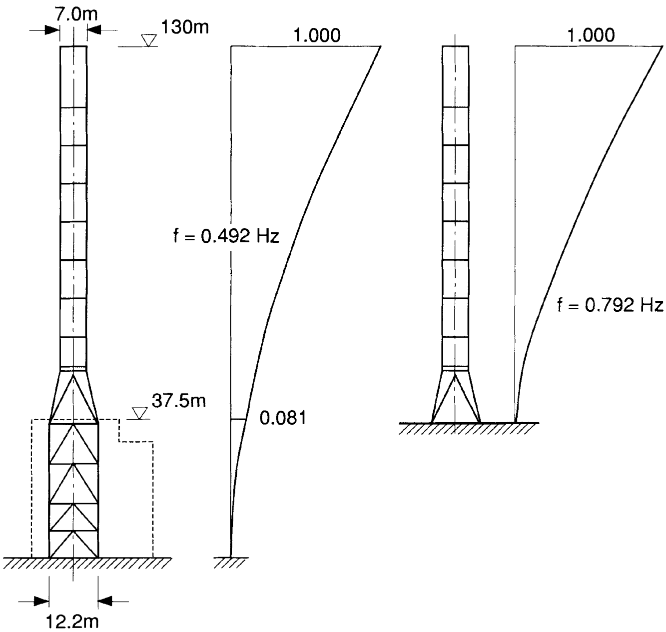


Figure 3.11: Examples of chimneys with and without base structure and the corresponding deflection curves

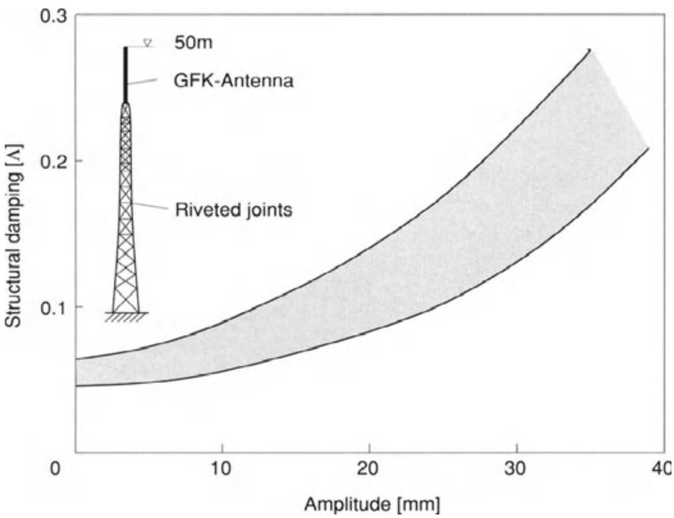


Figure 3.12: Lattice tower with fibre-reinforced plastic antenna construction: dependence of damping on displacement amplitude [H.6]

3.3.3 Structural criteria

a) Natural frequencies

For the calculation of the gust action in the wind direction it suffices in general to determine the fundamental frequency. For steel chimneys of the simplest form (Figure 3.9a) the following approximate formula may be used:

$$f_e = \xi \cdot 1010 \cdot \frac{d}{h^2} \tag{3.10}$$

where d = external diameter of the tubular part [m]
h = heigth [m]
ξ = $\xi_v \cdot \xi_m$
ξ_v = factor taking into account changes of the tube's wall thickness over heigth: in general $\xi_v = \sim 1.15$
ξ_m = (mass of tubular part/total mass)$^{1/2}$, accounting for any additional mass (e.g. insulation)

Exact calculations for stepped chimneys and those attached to a base structure can be carried out using Rayleigh's method (Equation (3.3)).

For rows of chimneys connected horizontally (Figure 3.9c) the simple cantilever lies at right angles to the row's axis, so that in the longitudinal direction a possible frame effect also has to be considered. The natural frequencies (and also the structural damping) are thus greater in the connected direction than orthogonal to the row.

Where a base structure is present (Figure 3.11) a coupled system exists with two important natural frequencies. Both constitute fundamental frequencies as far as vortex-induced vibrations are concerned, since the corresponding aerodynamic excitation forces act only on the superstructure (the tubular part) and since the base structure leads to the chimney having an additional "second" fundamental frequency.

The case may arise where the base structure is of unsymmetrical construction. Then different natural frequencies (and thus different critical wind speeds, see Appendix H) are dominant depending on the direction in which the wind is blowing.

The antenna support structure shown in Figure 3.11 has to be treated in the same way as a chimney on a base structure.

For complex structural systems the calculation of the natural frequencies, even when computer programs are used, only gives approximate values. Reliable values can only be obtained with the aid of vibration measurements on existing structures. It is important therefore that the displacement amplitude in such tests (e.g. measurement of the decay curve) corresponds approximately to the expected amplitude at the critical wind speed because, in general, both the stiffness and the damping behave nonlinearly with respect to displacement amplitude. Figure 3.12 illustrates this for the case of damping of a lattice tower.

b) Damping

The structural damping of chimneys and masts as freestanding structural systems depends to a large extent on special structural features. The lowest damping is exhibited by steel chimneys of welded construction without insulation. Equivalent viscous damping ratios ζ as low as about 0.002 have been observed. For groups of steel tubes joined together, the damping value increases to about 0.008 (measured on a group of five tubes, arranged with one inside and four outside). A more extensive summary is given in [3.8].

Corresponding with these damping ratios in the draft of DIN 4133, Appendix A (1991), the equivalent logarithmic damping decrement Λ for steel chimneys is given in the range of 0.015 to 0.04. If there is a masonry lining the range increases up to 0.07 to 0.1. Thus, the range of values typical for masonry and reinforced concrete construction are reached. Damping values for masts are of the same order of magnitude (see Figure 3.12).

3.3.4 Effects

Wind-induced vibrations of chimneys and masts are of no consequence to persons, since they do not normally spend lengthy periods of time on these structures.

The effects of vibrations on the structures themselves, however, can be very significant due to structural fatigue. This problem has to be carefully investigated, especially for chimneys.

3.3.5 Tolerable values

For chimneys and masts generally, safety with respect to fatigue of the materials is of prime interest. Admissible stress differences are given in national and international codes. They depend essentially on the structural form and on the roughness (cross-sectional details: ribs, sharp-edged shape, etc.). In the draft of DIN 4133, Appendix A (1991), a list of cross-sectional details is given.

As a rough guide it is reasonable to limit the amplitude of vibrations at the top of the complete structure to

$$(y_o/d)_{max} = 0.04 \tag{3.11}$$

where y_o = amplitude of deflections at the top
d = diameter of the chimney or mast

For steel chimneys and steel masts vortex-induced vibrations perpendicular to the wind direction are more important, whereas for reinforced concrete chimneys gust forces are usually more important.

For the dynamic tilt angle of antennae refer to Section 3.2.5.

3.3.6 Simple design rules

First of all, a natural frequency calculation with the aid of an approximate method has to be carried out. The damping has to be estimated in each case. With the critical wind speed according to Appendix H.4.1, it can be checked whether this is in the range of wind speeds at the construction site. If so, the dynamic forces due to vortex excitation in the across-wind direction can be determined using one of the methods given in Appendix H.4.2.

On the basis of this result it is then decided whether additional damping is needed. For steel chimneys without an internal lining additional damping measures are usually necessary (see Section 3.3.5). However, other measures, such as reducing the roughness effect should also be considered.

The gust forces in wind direction can be calculated using a simplified method (Section 3.2.6).

3.3.7 More advanced design rules

A more refined approach following [3.12] may be suitable especially for steel chimneys. It must be observed, however, that the topic of vortex-induced vibrations in the across-wind direction - in particular for weakly damped structures like steel chimneys - has not been thoroughly discussed among specialists at an international level [3.11]. Thus great differences are still to be found in the concepts and computed results of various codes and standards [DIN 4133], [CICIND], [ISO TC 98/SC 3/WG 2], [ISO/DIS 4354]. Therefore, in the treatment of vortex-induced vibrations of weakly damped structures great caution is necessary.

3.3.8 Remedial measures

For (steel) chimneys of low damping and mass the occurrence of unacceptable vortex-induced cross-wind vibrations cannot be ruled out. As remedial measures both aerodynamic and mechanical aids should be mentioned. An aerodynamic measure that is often employed is the Scruton helical stabilizing device with three strakes, a rise of 4.5 to 5, a helix width of $0.1d$ and helix height of $0.35h$ applied over the upper $0.35h$ extension of the chimney. One disadvantage lies in the increased wind resistance in the wind direction, so that for existing structures this device often cannot be considered. Besides, it has been observed that with low structural damping the aerodynamic efficiency of the Scruton device deteriorates. Such measures are also ineffective in reducing buffeting effects (see Appendix H.3). For the reasons stated, added systems of a mechanical type (i.e. vibration absorbers) are becoming more and more popular.

In [3.12] various possibilities for designing vibration absorbers, especially for steel chimneys, are presented. In general, suspended pendulum systems consisting of a circular steel mass provided with rubber damping elements (see Figure 3.13) are selected. For steel chimneys the mass ratio is often chosen to be 0.05 [3.10].

The design of vibration absorbers to reduce wind-induced vibrations may be carried out as described in Sub-Chapter 3.2. In practice, the optimum values of natural frequency and damp-

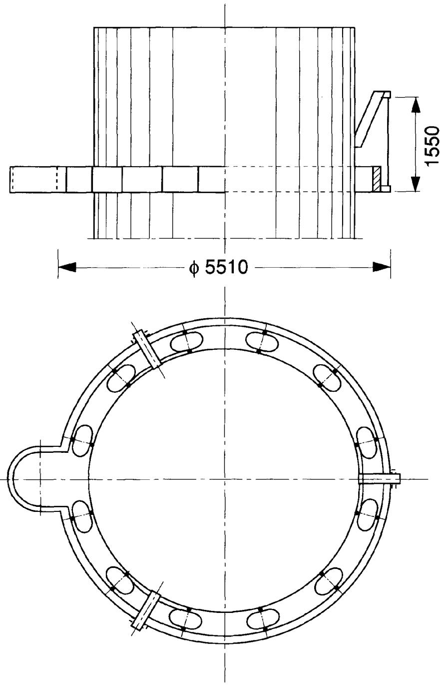


Figure 3.13: Pendulum-suspended vibration absorbers for chimneys [3.13]

ing are usually not attained precisely. However, the sensitivity of such added systems with respect to deviations from the optimum is relatively small.

Additional dampers also play an important role when the chimneys are arranged in a row or group and connected to one another (see Figure 3.10d). It is possible in this case to work with a single vibration absorber on one of the chimneys.

An illustrative example of the application of remedial measures to an existing group of steel chimneys when another chimney is added is given in [3.13]. All steps (problem analysis, suggested solution, execution, verification on site) are described and discussed there.

Further hints to the control of chimney vibrations are given in [3.34].

3.4 Guyed Masts

G. Hirsch, H. Bachmann

3.4.1 Problem description

Guyed masts are usually employed for mounting the antennae of transmitters. In the context of natural wind they present extremely complex structural problems due to the complex dynamic behaviour, which in general cannot be described accurately. The reason lies in the interaction of the substructures: mast, guys (often in groups) and antenna supporting structure, whereby the guys exhibit nonlinear behaviour with respect to the spring characteristic. In addition, the guys - similar to the masts - involve vibration systems, which couple with the other subsystems. More and more modern guyed masts consist of cylindrical antenna support structures of glass fibre construction (with the antennae lying inside) fixed to the steel tower. Vortices are shed from these circular cylinder structures and this can lead to significant vibration in the across-wind direction.

The treatment of the dynamics of guyed masts demands specialist knowledge and experience in the field of structural dynamics of complex systems [3.12]. Therefore, these masts - in accordance with the aims of the present document - are only discussed briefly.

3.4.2 Dynamic actions

The dynamic behaviour of guyed masts under wind excitation is stochastic even for critical wind speeds, which may be traced to the nonlinear influence of the guys [3.16]. The effective prestress in the guys plays an important role. For extremely high prestressing the system behaves almost linearly, whereas for smaller prestressing the response is irregular since the dynamic properties depend on the amplitude of vibration. In the latter case the system in part controls itself. Under some circumstances the vibrating guys contribute to passive vibration control, since with their numerous natural frequencies in the range of interest they represent many counter-oscillators with random tuning.

3.4.3 Structural criteria

a) Natural frequencies

For guyed masts in the frequency range of interest - i.e. vortex-shedding frequencies of about 4 Hz - several natural frequencies and associated mode shapes occur. At the planning and design stages they can only be determined by means of relatively time-consuming calculations (see Section 3.4.7).

The i-th natural frequency of a guy rope, which is anchored at two fixed points, is obtained from:

$$f_{e,i} = \frac{i}{2 \cdot c}\sqrt{T_c/m} \tag{3.12}$$

where m = the mass/unit length of guy
T_c,c see Figure 3.14.

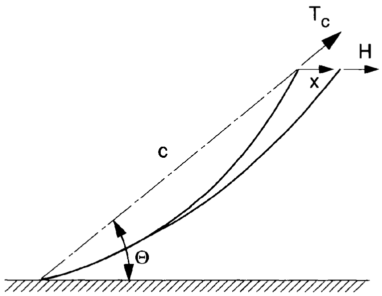


Figure 3.14: Symbols for guy calculations

Due to the continually changing displacements of the upper anchoring point of vibrating guyed masts, and the associated change of the force in the guy, the stiffness and thus the guy's natural frequencies change continually [3.17]. Therefore, it is not possible to carry out accurate calculations for guyed masts using simplified methods. From the point of view of the engineer it is, however, feasible to carry out a rough calculation of the natural frequencies for a simplified linearized model. By means of vibration measurements on the completed structure the dynamic response to wind loading can then be observed and used to verify the approximate calculations. Thus, actual values of the damping characteristics of the structure can be obtained.

b) Damping

The structural damping of guyed masts is greater than for freestanding slender structures. Damping depends in particular on the degree of prestressing. For glass fibre antenna construction an equivalent viscous damping ratio $\zeta = 0.0065$ (logarithmic decrement $\Lambda \approx 0.04$) may be assumed.

3.4.4 Effects

Effects on persons are in general unimportant. The actual influences on the structure can only be determined with great effort (theoretical and/or experimental).

3.4.5 Tolerable values

For guyed masts relatively large displacements may be permitted. Restrictions come from the operating conditions for the antenna. Some information can be found in [H.1]. Actual values have to be supplied by the antenna manufacturer.

3.4.6 Simple design rules

For masts there are no simple design rules within the scope of these guidelines. For the antenna structure the across-wind vibrations can be calculated according to Sub-Chapter 3.3. Based on this result it can be decided at the outset whether preventive measures are necessary.

3.4.7 More advanced design rules

The dynamic modelling of guyed masts depends to a large extent on the effort one wants to put into a theoretical investigation. Figure 3.15 shows various modelling possibilities. Often the simplified model a) is employed. The assumed massless guy is replaced by its (linear) spring stiffness in the plane. The mast is discretized into concentrated masses, e.g. at the upper anchor point of the guy. Wind effects must be considered for gusts and the vortex excitations associated with circular mast sections [3.15].

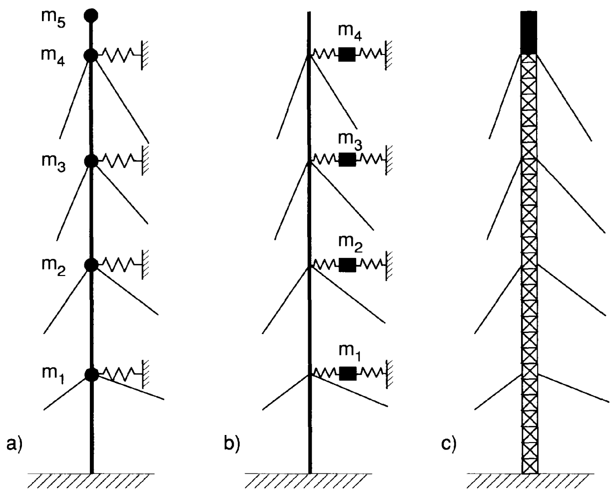


Figure 3.15: Dynamic modelling of a guyed mast

An improvement can be achieved using model b), where the guys are also considered to be vibrating systems. Therefore, in each case a generalized mass of the group of guys, i.e. those for a particular eigenvalue, is shown. The case may arise, however, where higher natural frequencies of the group of guys correspond to the natural frequencies of the mast, so that the guys' significance can be much greater. Besides, with this model the guy stiffness can be divided into a "quasi-static spring" and a "dynamic spring". Depending on the wind speed the mast can find itself in a "quasi-static position of rest", about which the vibrations occur. In reality, of course, there is no such "position of rest", since the wind is continually changing. In view of these considerations it is thus clear that only very simplified models are possible for the dynamic behaviour of guyed masts.

Finally, Figure 3.15c) is intended to give an idea of modern computational methods (finite element method). The abovementioned limitations apply in principle here also. More exact, i.e. nonlinear, calculations are indeed possible at great expense, but there is no assurance that the results preclude wind-induced vibrations of unacceptable amplitudes.

As a modern and refined approach an interplay of rough calculations and specific structural measurements (natural frequencies, damping in the mast, guys and antenna supporting structure) can be recommended. With the results of the measurements, the calculations can then be more refined or a decision regarding the use of additional measures, possibly for the antenna structure and the guys, can be taken. In addition, the dynamic forces that affect the point of force introduction (the guy-structure connection) must be checked. Attention must be paid to designing these locations for long-term resistance to fatigue.

3.4.8 Remedial measures

For guyed masts, measures to dampen out vibrations are cruical. A vibration absorber (see Section 3.3.8) may be placed at the top of the structure supporting the antenna. According to the present state of knowledge a vibration absorber of about1000 to 1500 kg is adequate. Vibration absorbers attached to the mast are in general not necessary and for practical reasons (space, access) they often cannot be placed where the modal response is at its maximum.

The guys are best brought to rest with simple dampers. Figure 3.16 shows a sketch of such a system. Information on determining the effective damping of a guy with additional dampers is provided in [3.18]. Practical cases of application show that it is possible to increase the very low damping (ζ = 0.0005) of the bare guy by a factor of about ten.

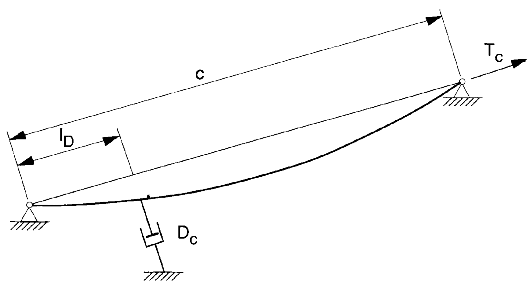


Figure 3.16: Guy with damper (D_c)

In the choice of the damper, attention must be paid to its suitability for long-term operation. Today, purely mechanical working elements are available (e.g. slit annular springs, disc springs, etc.), which are maintenance-free. The adjustment to the guy forces is quite simple, so that despite inadequate accuracy of theoretical investigations it can be ensured that the (unavoidable) vibrations remain within the admissible limits [3.19], [3.20], [3.21].

3.5 Pylons

G. Hirsch, H. Bachmann

3.5.1 Problem description

The pylons of cable-stayed and suspension bridges represent a special kind of tower structure. Steel pylons are particularly susceptible to vibrations, but slender concrete pylons can also experience large vibrations. The nature of the vibrations depends on the form of the bridge and the pylon (Figure 3.17). Apart from their response to stochastic gust effects, bridge pylons show a tendency both to forced vibration due to vortex excitation and - for steel pylons - to self-induced vibrations (galloping). In combination with cables, pylons also constitute guyed sub-systems (cf. Sub-Chapter 3.4). In general, different dynamic properties (natural frequency, damping) are exhibited in the planes of the bridge axis from those in the transverse direction.

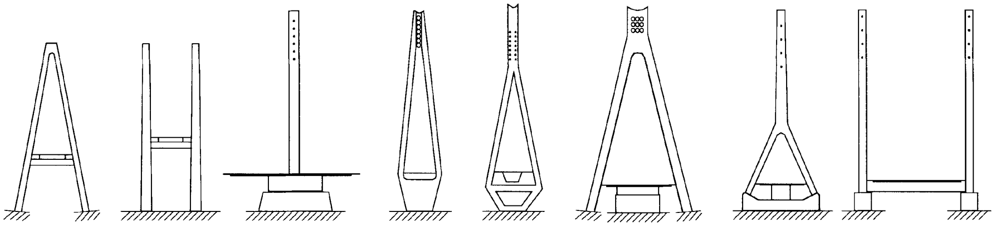


Figure 3.17: Various types of bridge pylons

For pylons the stage of construction is also important. Initially the pylon of a cable-stayed bridge is built as a freestanding cantilever without guys. Under wind loading the structure behaves like a tower or "chimney" of rectangular section. Vortex excitation and galloping can result for such rectangular shapes (Appendix H). Since the damping is small at this phase of construction it may be necessary to employ temporary damping measures. After completion of the cables it is in general possible to dispense with such anti-vibration measures in the plane of the bridge. However, it has happened that after completion of a bridge, unacceptably large wind-induced vibrations of the pylons have occurred in the plane of the bridge. Figure 3.18a shows a cable-stayed bridge with pylons that had to be stabilized using a vibration absorber subsequent to construction [3.22]. For vibrations outside the plane of the bridge, the presence of cables scarcely has any effect.

The pylon of a cable-stayed bridge constitutes a very complex aerodynamic vibration system. For long span bridges where modal responses of the substructures (main beam, pylons, cables) are present, a pronounced vibration transmission may occur within the overall system. This cannot be recognized with certainty in the design stage. Closely spaced natural frequencies of the substructure may be present, leading to this transmission effect. Due to the dynamic effect of the traffic flow, the bridge is excited at certain natural frequencies, and these vibrations are then transmitted to the substructures having similar eigenvalues. Wind-induced vibrations may be superimposed on these forced vibrations, which may then experience amplification to large amplitudes, and may combine with the so-called parametric-excited (self-excited) vibrations of the cables [3.20] that are attached both to the main deck beams and the pylons.

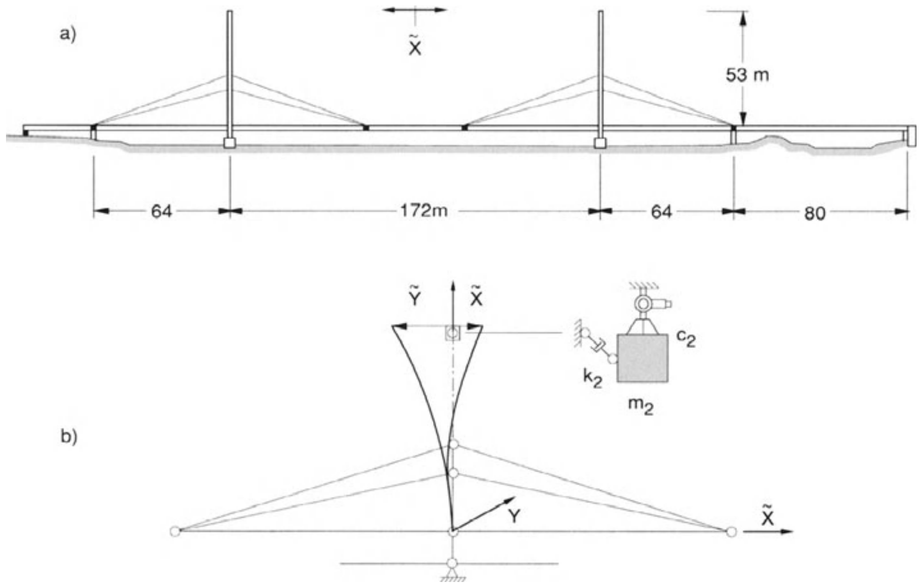


Figure 3.18: Cable-stayed bridge: a) overall system, b) pylon with vibration absorber

From this it follows that - as with guyed masts - the actual vibration behaviour of cable-stayed bridges, and thus also of the pylons, is usually difficult to describe accurately under natural wind excitation. Thus, from the engineer's point of view there remains no alternative but, with the aid of simplified calculations (including the use of advanced computer programs), to gain a certain insight into the behaviour and if necessary, to implement specific anti-vibration measures (see also [3.24]).

3.5.2 Dynamic actions

The dynamic action depends mainly on the shape of the pylons. Vertical supports are more disadvantageous than sloping ones, because vortex-shedding is more prominent in the former than in the latter. For large bridges, wind tunnel tests are often carried out. From these the aerodynamic and also - with the aid of aeroelastic models - the dynamic response of the structure are obtained with sufficient accuracy.

[3.23] presents a case history in which wind tunnel tests showed that for a particular cable-stayed bridge during the construction phase a freestanding pylon (without cables) was greatly endangered (vortex-shedding and galloping instability), but that leaving the scaffolding for the welding work in place at the top part of the pylon, the wind-induced vibrations could be reduced sufficiently to require no additional control measures.

3.5.3 Structural criteria

a) Natural frequencies

The bending frequency of a free-standing pylon (during construction) in both directions, or of a pylon with sloping cables for vibrations transverse to the plane of the bridge, can be roughly estimated for preliminary design purposes. As with buildings (Equation (3.2)), $f_e = 46/h$. In general, the second natural frequency is about four times higher. Exact calculations are possible using Rayleigh's method (see Section 3.2.3). For determining the natural frequencies of a cable-stayed pylon in the plane of the bridge - analogous to guyed masts - at least the cable stiffness (assumed linear elastic as a first approximation) has to be considered. Depending on the actual stiffnesses of the pylon, the cables and the main deck beam, as well as the degree of restraint of the base support and of the foundation, such calculations may only provide a rough estimate (see [3.24]).

b) Damping

The damping in steel pylons acting as freestanding structures is comparable to that in steel chimneys (see Section 3.3.3). For the steel A-shaped pylon of the bridge described in [3.23] an equivalent viscous damping ratio of $\zeta = 0.002$ (logarithmic decrement $\Lambda \approx 0.012$) was measured. It follows that the bridge during its construction phase is greatly endangered by vibrations. For the steel pylons shown in Figure 3.18 an equivalent viscous damping ratio of $\zeta = 0.003$ (logarithmic decrement $\Lambda \approx 0.02$) has been observed, whereby no substantial difference was shown in the principal vibration directions (x and y). Due to the rectangular cross-section and the stiffening effect of the cables, the natural frequencies were different (1.93 Hz in the x-direction and 1.1 Hz in the y-direction). The damping values for cable-stayed pylons are greater than those for freestanding pylons.

3.5.4 Effects

Normally, persons are not directly affected by wind-induced vibrations on bridge pylons, except when there is a viewing platform on the pylon, in which case an assessment can be made by referring to Sub-Chapter 3.1. However, the effects on the bridge structure as a whole also have to be considered. If the bridge deck is set in vibration this affects the comfort of users, especially pedestrians.

With respect to structural safety the effects of wind-induced vibrations of the pylons are mainly related to fatigue damage (see Sub-Chapter 3.3).

3.5.5 Tolerable values

Permissible values for specific objects should be laid down in the specifications for the bridge structure (contract documents). Generally applicable values cannot be given. In particular, separate considerations have to be given to safety and serviceability of the bridge structure.

3.5.6 Simple design rules

The response to gusts in the wind direction can be estimated using the dynamic properties of the pylon (frequency and damping) analogous to the case of buildings (see Sub-Chapter 3.1). Further, for preliminary design purposes with the aid of approximate upper and lower limits of natural frequency, estimates can be made of the upper and lower limits of critical wind speeds for vortex-shedding and galloping according to Appendices H.4.1 and H.5. If these critical speeds lie in the range of possible wind speeds, more detailed investigations have to be carried out.

3.5.7 More advanced design rules

For a more accurate investigation of the modal behaviour and of the dynamic response of the pylon as part of a complex bridge structure - at least in the case of large bridge projects - wind tunnel tests on aerodynamic models of the whole bridge together with the use of modern computer programs are necessary. This task belongs to the specialized field of structural dynamics.

3.5.8 Remedial measures

Measures to reduce the wind-induced vibrations of bridge pylons can be planned at the outset by aiming for stability with respect to vibrational effects. It may not be possible, depending on the circumstances, to make a decision about the necessity of remedial measures until observations and corresponding measurements on the completed structure are available. In the subsequent planning (and execution) of such remedial measures one has to reckon with much greater costs than if preventive measures had been taken initially.

In the first place - as with towers and chimneys - optimum tuned vibration absorbers should be considered. Figure 3.18b shows a sketch of such a system for a pylon. Since the natural frequencies in both principal directions are different, a pendulum suspension in the form of two torsional springs placed one behind the other was chosen, such that the pendulum system also exhibits different natural frequencies in the two directions of vibration. The damping was achieved by means of hydraulic pistons. Since easy access was provided - thus allowing later replacement of the elements - it was not necessary to give detailed consideration to their durability.

During construction, vibration of the pylon can also be controlled with the aid of auxiliary devices to increase damping, e.g. as in Figure 3.19, [3.25]. The friction damping of a thin cable under a relatively small tension force (several kN) is adequate for achieving the desired effect for weakly damped structural systems.

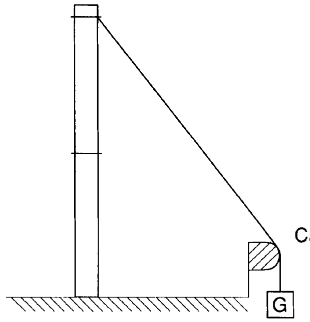


Figure 3.19: Auxiliary damping device for a cantilever system

Further possibilities can be found in [3.24]. Viscous dampers have the disadvantage that usually a substantial part of the added mass acts as a rigid mass and that one cannot dispense with special experimental optimizations.

3.6 Suspension and Cable-Stayed Bridges

G. Hirsch, H. Bachmann

3.6.1 Problem description

In the course of time, the spans of large bridges have continually increased. Long-span bridges are designed as either suspension or cable-stayed bridges. Figure 3.20 gives an overview of this development.

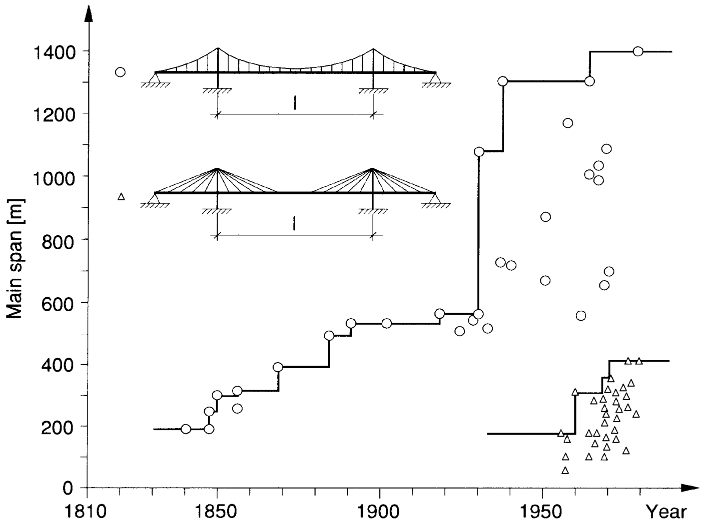


Figure 3.20: Development of main span lengths of suspension and cable-stayed bridges [3.22]

Figure 3.21 presents some modern bridge cross-sections which have been shown to meet the requirement of satisfactory dynamic behaviour with respect to the effects of gusts, vortices, galloping and bridge flutter (see Appendix H).

It is evident that in the planning and design of such large bridges, wind tunnel tests should be carried out together with competent structural dynamics calculations performed by specialists. Here it is possible to make only a few remarks regarding this highly specialised topic. Emphasis is placed on the main spans, the cables and the hangers. The pylons have been treated separately in Sub-Chapter 3.5. The construction stages are also discussed briefly.

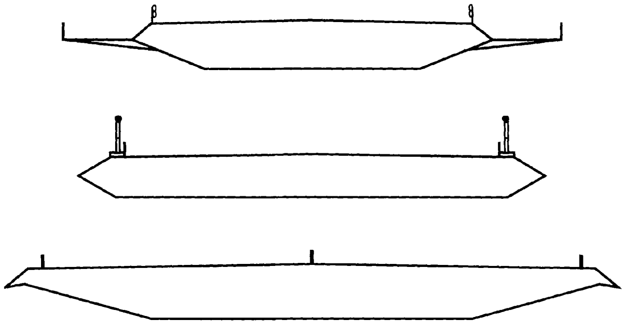


Figure 3.21: Bridge cross-sections

3.6.2 Dynamic actions

Dynamic wind action depends on a large number of parameters. Besides the aerodynamic design of the cross-section, of importance are the bending and torsional stiffnesses and the mass distribution of the main span (including the rotational inertia about the longitudinal axis of the bridge). The cables, hangers and pylons have to be considered as well. Cable-stayed bridges with only one plane of cables have to be handled with care as they exhibit a relatively small torsional stiffness and thus a correspondingly small ratio of torsional to bending natural frequencies.

In the especially critical case of bridge flutter, the same mechanisms are not present as in the classical case of the flutter of aircraft wings. For bridge flutter it is a question of the aeroelastic instability of a bluff body (not a streamlined form) with air-stream separation, in which the movement takes place in the bending and torsional degrees of freedom [3.26].

During construction there is a continuously changing vibrational system with changing dynamic properties and force response. It has to be verified what critical conditions will be encountered and when. Continuous checks with the aid of torsional and bending measurements may be appropriate (e.g. as was conducted on the Severn Bridge, England [3.27]).

3.6.3 Structural criteria

a) Natural frequencies

Figure 3.22 gives an indication of the bending natural frequencies (fundamental vertical frequency) of cable-stayed bridges as a function of the span length. For estimation purposes the following approximation can be used

$$f_e = \frac{110}{L} \tag{3.13}$$

where L = length of the main span [m]
f_e = fundamental bending frequency [Hz]

Normally, for cable-stayed bridges the bending natural frequency is higher for a harp-shaped arrangement of the cables than when the cables are bundled to the pylon.

In the case of cantilevering construction of a cable-stayed bridge, the calculation of the eigen-frequencies (horizontal transverse for gust forces, and vertical for vortex excitation, galloping and flutter) are carried out basically the same way as for the structural systems treated in Sub-Chapters 3.2 to 3.4.

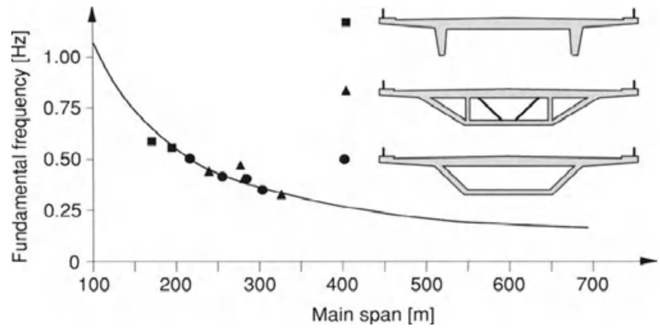


Figure 3.22: Fundamenta bending frequencies of cable-stayed bridges [3.22]

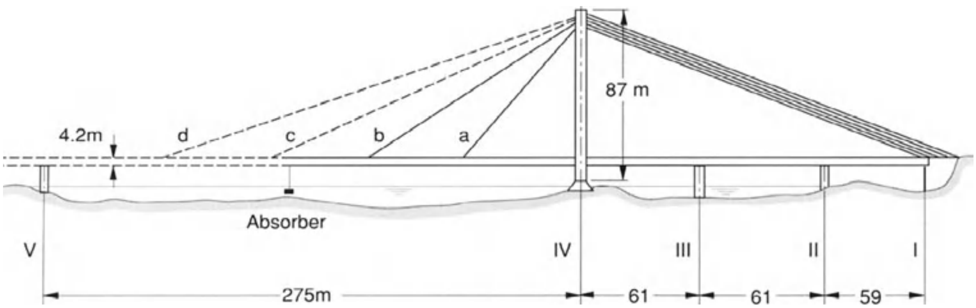


Figure 3.23: Cantilever construction of a cable-stayed bridge with preventive damping measures [3.23]

In the example shown in Figure 3.23, for every step in connecting a further group of cables new natural frequencies (and thus other values of critical wind speeds) result. With progress-ing cantilever construction the natural frequencies become smaller. The critical state is usually attained before the other river bank is reached.

b) Damping

Figure 3.24 presents a summary of damping values for long-span bridges.

For an accurate assessment and monitoring of construction stages, damping values can be ob-tained from the decay curves of rhythmical motions excited by the construction team [3.23].

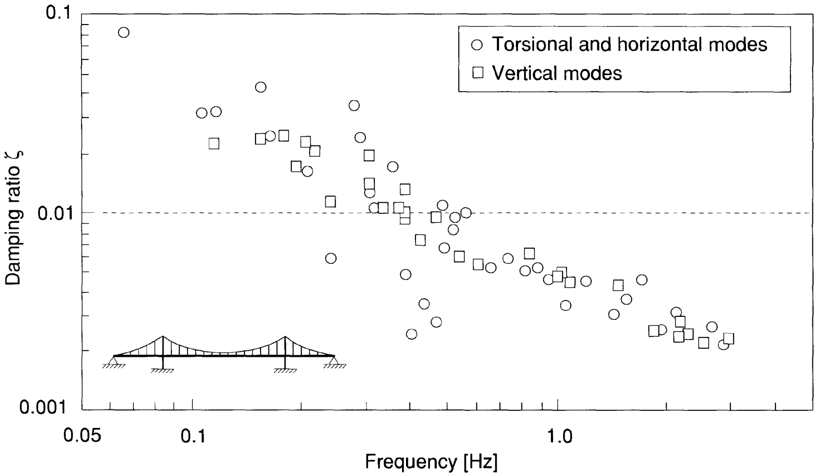


Figure 3.24: Damping values for long-span bridges [H.2]

3.6.4 Effects

Vibrations of the bridge deck can impair the comfort of the user, above all of pedestrians. With respect to effects on the structure (structural safety), the vibrations of the deck, cables and hangers are primarily of relevance for fatigue considerations.

3.6.5 Tolerable values

Tolerable values for serviceability and safety (fatigue) have to be established from case to case or taken from national and international codes and standards.

3.6.6 Simple design rules

Besides the choice of the most favourable aerodynamic cross-section ("streamlined profile"), one should attempt to obtain as high as possible a fundamental torsional frequency compared with the fundamental bending frequency.

With the aid of the given approximate values of natural frequencies and damping, rough estimates can be made of the expected wind-induced vibrations. This is the case in the first instance for wind action due to gusts (turbulence) and vortex shedding.

3.6.7 More advanced design rules

Using aeroelastic structural dynamics computer programs, calculations of flutter and galloping can be carried out. The aerodynamic parameters (see Appendices H.5 and H.6) have to be determined with the aid of wind tunnel tests. In this process, mostly aerodynamic measures (use of fairings, flaps and splitter plates, see Figure 3.25) are investigated. The effect of turbulence on flutter behaviour also has to be considered. Hangers and cables have to be investigated according to Sub-Chapter 3.4.

3.6.8 Remedial measures

The vortex-excited vibration behaviour of the main spans of bridges can be favourably influenced as is seen in Figure 3.25. Flutter and galloping stability can be improved by having a more favourable shape of the bridge profile. An increase of damping, on the other hand, has no significant influence.

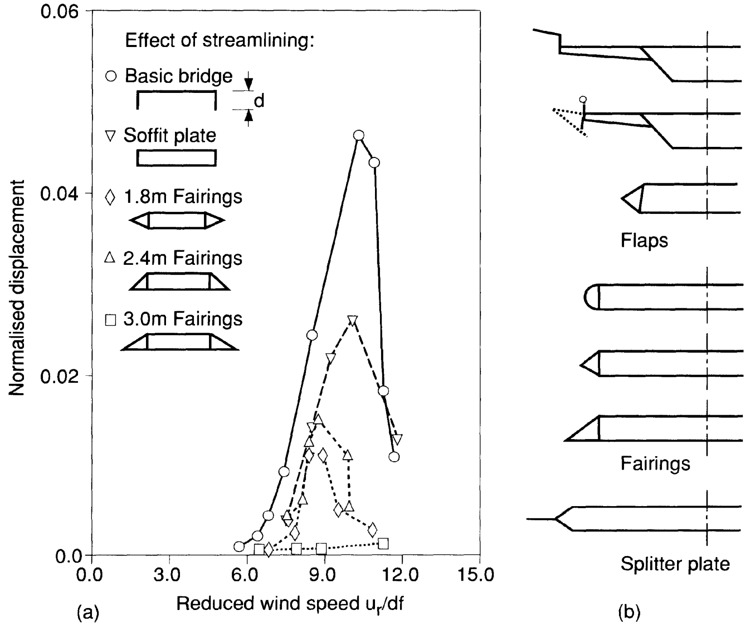
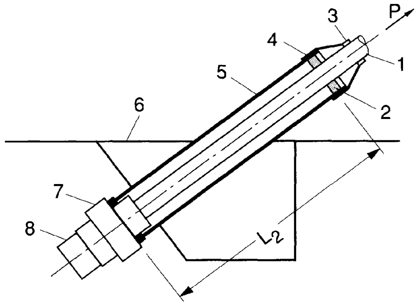


Figure 3.25: Influence of bridge profile shape with vortex-induced vibrations: a) results of investigation, b) examples of preventive measures [3.26]

Vibration absorbers (tuned-mass-dampers) can also be used to resist vortex-induced vibrations (see Appendix D, with additional information in Sub-Chapter 3.3).

For cantilevering construction of cable-stayed bridges a measure to dampen vortex-induced bending vibrations that can be easily implemented is to dip a cable-suspended "steel crossbar hedgehog" into the river, see Figure 3.23. A vibration absorber, comprising rubber buffers (or possibly air springs) with the mass consisting of concrete blocks or steel girders, can also be arranged at the end of the cantilever in the case of cantilever construction. By changing the size of the mass the natural frequency of the tuned-mass damper can be adjusted to correspond to the dynamic state of the cantilever.

Sloping cables can be further damped in the same way as the guy cables of masts (see Figure 3.16).



1 = Stay cable
2 = Neoprene washer
3 = Clamp
4 = Neoprene sleeve
5 = Steel tube
6 = Bridge deck
7 = Lock nut
8 = Anchor

Figure 3.26: Stay cable with lining and damping device [3.22]

In Figure 3.26 another means of obtaining additional damping of cables is shown, in which the cable lining in the region of the anchorage is used in order to activate a damping layer between the cable and the support.

Further applications may be found in [3.28]. Unfortunately it is usually only evident on the completed structure that, for example, the cable or the hangers is critically excited by the wind. Additional measures may be applied, but one should not forget at the planning and design stages that extra costs are involved in implementing such measures to a completed structure (e.g. the RAMA IX cable-stayed bridge in Bangkok, with introduction of damping after construction to reduce cable vibrations [3.29]).

3.7 Cantilevered Roofs

G. Hirsch, H. Bachmann

3.7.1 Problem description

Cantilevered roofs, e.g. roofs of grandstands, can be particularly sensitive to wind excitation. Steel structures are affected most, but problems with (lightweight) concrete cannot be ruled out. Depending on the wind direction, very different aeroelastic effects can be induced, e.g. galloping (see Appendix H). As shown in Figure 3.27, the roof plate for a wind angle $\beta = 0$ and a partial opening or closure of the rear side of the stand (blockage, e.g. $B = 50\,\%$ in Figure 3.27a) resembles an aerofoil with a certain incidence angle. For a negative gradient of the lift/incidence angle, this can produce "stall-flutter" (for structures with air stream separation = galloping).

In addition, vortex-shedding and vortex excitation is possible, whereby according to Figure 3.27, depending on the horizontal flow angle β, different types of excitation are possible. The data comes from the results of wind tunnel tests. For such structures these tests are indispensable, above all when the cantilevered roof construction is more complicated. The most important cases here would be closed, ring-shaped or oval roof structures of large sport stadia. Both, vibration damping and aerodynamic measures are possible.

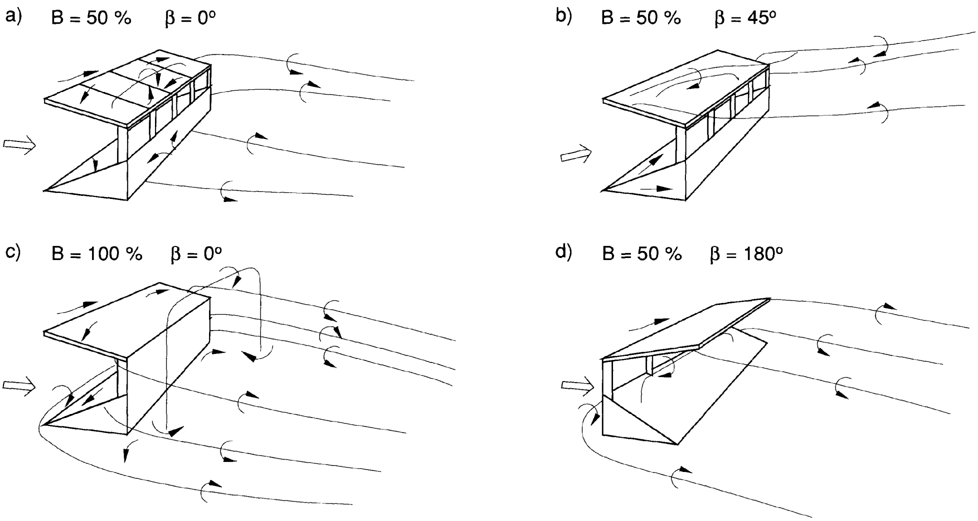


Figure 3.27: Cantilevered roof for various incident air flows and vortices [3.30]

3.7.2 Dynamic actions

Besides the geometric shape and structural dynamics parameters (natural frequencies, mode shapes and damping) the dynamic effect of wind on cantilevered roofs depends on the windstream (direction, speed). Of chief interest are the actions and dynamic responses with respect to vortex-shedding and galloping. Specific data for a structure, however, can only be obtained from wind tunnel tests [3.30], [3.31] and [3.32].

3.7.3 Structural criteria

a) Natural frequencies

The fundamental frequency of the roof can be estimated using formulae for cantilevers or using Rayleigh's method (see Section 3.2.3). For more exact calculations, methods must be employed which are suitable for non-uniform cross-sections and specific boundary conditions (partially-fixed support). From measurements on actual cantilevered roofs, natural frequencies in the range from 1 to 4 Hz have been observed.

b) Damping

Damping values for cantilevered roofs can vary very much, depending on the type of construction and possible energy dissipation due to non-loadbearing elements (roof tiles) and energy radiation into the ground. In [3.33] a logarithmic decrement of $\Lambda = 0.13$ is reported for a welded steel construction (cantilever beam with plates and roofing of corrugated sheeting).

3.7.4 Effects

Effects on persons may be of significance if the stadium is occupied under windy conditions. On the other hand, fatigue problems may develop due to the vibrations.

3.7.5 Tolerable values

Tolerable values for fatigue are supplied in national and international codes and standards.

3.7.6 Simple design rules

Possible problem cases can be identified by the mass-damping-parameter MDP:

$$MDP = \frac{2 \cdot m \cdot \Lambda}{\rho \cdot h^2} \tag{3.14}$$

where m = mass per unit length
h = thickness of roof
ρ = air density (1.2 kg/m^3)
Λ = logarithmic decrement

Care must be taken for MDP values below ~15. In such cases no simple design rules are available.

3.7.7 More advanced design rules

The modal behaviour of the structure can be estimated with the aid of a finite element computer program. Investigations on an aeroelastic model in a boundary layer wind tunnel (including simulation of turbulence effects) can provide the basis for a safe design.

3.7.8 Remedial measures

For a roof structure with low structural damping, the reduction of the dynamic response to wind excitation can only be achieved by using vibration absorbers (see Appendix D in connection with the relevant extensions regarding wind in Sub-Chapter 3.3). The frequency and the number of vibration absorbers to be installed depends on the number of critically excited modes in the roof and on the mode shapes themselves. In general, a greater number of small vibration absorbers is preferable.

Aerodynamic measures are also possible. According to [3.30] a reduction of the dynamic response can be achieved by employing trip wires causing turbulence underneath the roof construction. The effect of the wind in the region of the "critical" frequency is markedly reduced when turbulence is present as compared to uniform air flow.

References to Chapter 3

[3.1] Hirsch G., Ruscheweyh H.: "Newer Investigations of Non-Steady Wind Loadings and the Dynamic Response of Tall Buildings and Other Constructions". Proceedings 3rd. International Conference on wind effects on buildings and structures (3 ICWE), pp. 811-823. Saikon Co. Ltd., Tokyo, 1971.

[3.2] Jeary A.P., Ellis B.R.: "On Predicting the Response of Tall Buildings to Wind Excitation". Journal of Wind Engineering and Industrial Aerodynamics 13, pp. 173-182, 1983.

[3.3] Chang F.K.: "Wind and Movement of Tall Buildings". Journal of ASCE, Structural Division, pp. 70-72, 1967.

[3.4] Soong T.T.: "Active Structural Control in Civil Engineering". Engineering Structure 10, pp. 74-84, 1988.

[3.5] Leipholz H.H.E.: "Structural Control". North-Holland SM Publication, Amsterdam, 1980.

[3.6] Müller F.P., Charlier H., Kessler H.: "Wind- und Schwingungsmessungen am Sendeturm Hornigsrinde" ("Measurements of Windloads and Vibrations on Aerial-Tower Hornigsrinde"). Berichte Konstruktiver Ingenieurbau, Heft 35/36, pp. 11-15, Vulkanverlag, Essen, 1981.

[3.7] Take U., Schneider F.X., Schneider H.: "Vergleich der Ergebnisse der Wind- und Betonspannungsmessungen von den Fernmeldetürmen München, Brotjackriegel und Hornigsrinde" ("Comparison of Results of Measurements of Windload and Concrete- Stresses on Telecommunication Towers Munich, Brotjackriegel and Hornigsrinde"). Berichte Konstuktiver Ingenieurbau, Heft 35/36, pp. 35-42, Vulkanverlag Essen, 1981.

[3.8] Hirsch G.: "Aktive und passive Kontrolle dynamischer Verformungen von schlanken Strukturen unter Windbelastung" ("Active and Passive Control of Wind-Induced Vibrations of Slender Structures"). Berichte Konstruktiver Ingenieurbau, Heft 35/36, pp. 49-55, Vulkanverlag, Essen, 1981.

[3.9] Simiu E., Scanlan R.H.: "Wind Effects on Structures". John Wiley & Sons, New York, 1978.

[3.10] Hirsch G.: "Practical Experiences in Passive Vibration Control of Chimneys". Structural Control, pp. 278-296, Martinus Nijhoff Publications, Dordrecht, 1987.

[3.11] Ruscheweyh H., Sedlacek G.: "Crosswind Vibrations of Steel Stacks - Critical Comparison Between Some Recently Proposed Codes". Advances in Wind Engineering, Part 3, pp. 173-184, Elsevier, Amsterdam, 1988.

[3.12] Petersen C.: "Stahlbau" ("Steel-Construction"). Vieweg-Verlag, 1989.

[3.13] Adler P., Hirsch G.: "Dämpfung winderregter Schwingungen von Stahlschornsteinen in Gruppenanordnung" ("Damping of Wind-Induced Vibrations of Steel Chimneys in Group Arrangement"). Bautechnik 63 (1986), pp. 223-228. English Version in Proceedings International CICIND-Chimney Conference, Brighton, UK, 1988.

[3.14] Ruscheweyh H.: "Wind-Induced Vibrations of Towers and Stacks". Proceedings IUTAM-JAHR Symposium, Karlsruhe. Practical Experiences with Flow induced Vibrations, Springer, pp. 709-717, 1979.

[3.15] Gerstoft P., Davenport A.G.: "A Simplified Method for Dynamic Analysis of a Guyed Mast". Proceedings 6th Colloquium on Industrial Aerodynamics, Part 2, pp. 247-258. F.H. Aachen, 1985.

[3.16] Hirsch G., Nonhoff G., Volkmar H.: "Wind Induced Vibrations of Guyed Masts". Preprints EURODYN '90, SFB 151 Ruhr-Universität Bochum, Volume 1, pp. 815-823, 1990.

[3.17] Kärnä T.: "Dynamic and Aeroelastic Action of Guy Cables". Espoo 1984. Technical Research Center of Finland, Publication 18, 1984.

[3.18] Hirsch G.: "Kontrolle der wind- und erdbebenerregten Schwingungen von weitgespannten Schrägseilbrücken" ("Control of Wind- and Seismic Induced Vibrations of Long Span Cable-Stayed Bridges"). VDI-Bericht 419, pp. 101-109, 1981.

[3.19] Carne T.G.: "Guy Cable Design and Damping for Vertical Axis Wind Turbines". Sandia National Laboratory, Division 5523, Albuquerque, New Mexico, SAND 80-2669, 1981.

[3.20] Kovacs I.: "Zur Frage der Seilschwingungen und der Seildämpfung" ("On Questions of Cable-Vibrations and Cable-Damping"). Bautechnik pp. 325-331, 1982.

[3.21] Schmackpfeffer H.: "Rheinbrücke Emscherschnellweg" ("Rhine Bridge Emscherschnellweg"). Bauingenieur 65, pp. 453-462, 1990.

[3.22] Aschrafi A., Hirsch G.: "Control of Wind-Induced Vibrations of Cable-Stayed Bridges". Journal of Wind Engineering and Industrial Aerodynamics, 14 (1983), pp. 235-246, 1983.

[3.23] Hirsch G., Ruscheweyh H.: "Vibration Measurement on a Cable-Stayed Bridge Under Construction". Journal of Wind Engineering and Industrial Aerodynamics 1, (1975/76), pp. 297-300, 1975/76.

[3.24] Kanok-Nukulchai W.: "Cable Stayed Bridges, Experiences & Practice". CABRIDE AIT, Vol. 1 and Vol. 2. Proceedings International Conference on Cable Stayed Bridges, Bangkok, Thailand, 1987.

[3.25] Langer W.: "Querschwingungen hoher schlanker Bauwerke mit kreisförmigem Querschnitt" ("Cross-wind Vibrations of Tall Slender Buildings of Circular Cross Section"). Mitteilungen des Instituts für Leichtbau (IfL), Technische Universität Dresden, 8, pp. 184- 197, 1969.

[3.26]]Scanlan R.: "On the State of Stability Considerations for Suspended-Span Bridges Under Wind". Practical Experiences with Flow-Induced Vibrations, pp. 595-617. Springer, Berlin, 1980.

[3.27] Roberts G.: "The Severn Bridge, a New Principle of Design". Proceedings Symposium on Suspension Bridges. Lab. Nac. de Engenharia Civil, Lisboa, pp. 629-639, Lisboa, 1966.

[3.28] Modemann H.J., Schräder R.: "Rheinbrücke Emscherschnellweg" ("Rhine Bridge Emscherschnellweg"). Bauingenieur 65, pp. 343-350, 1990.

[3.29] Virlogeux M.: "Le projet du Pont de Normandie" ("Bridge Project Normandy"). Annales des Ponts et Chaussées, no. 2, pp. 7-23, 1989.

[3.30] Barnard R.H.: "Wind Loads on Cantilevered Roof Structures". 4th Colloquium Industrial Aerodynamic, FH Aachen, 1980. Part 1, pp. 63-73. See also Ph.D.-Thesis, Hatfield Polytechnic, England, 1979.

[3.31] Mankau H.: "Investigation of Flow Induced Oscillations of a Cantilever Roof Model". In [3.30].

[3.32] Sawyer R.A.: "Wind Load on the Roof of a Racecourse Grandstand". In [3.30].

[3.33] Fischer M.: "Schwingungsuntersuchung am weit auskragenden Tribünendach des Stuttgarter Neckarstadions" ("Vibration Investigation on Long Span Cantilever Roof of Neckar-Stadium Stuttgart"). Der Stahlbau 10, pp. 304-307, 1974.

[3.34] Józsa M., Hirsch G.: "Optimum Control of Chimney Vibrations". CICIND-Report, Vol. 10, No. 1, 1994.

4 Vibrations induced by traffic and construction activity

W. Ammann, F. Deischl, J. Eisenmann, G. Lande

This chapter deals with structural vibrations caused by road and railway traffic and by construction activities.

Vibrations induced by both traffic and construction activities affect mainly the serviceability of adjacent structures and only rarely cause problems related to structural safety. Problems with structure-borne sound may often become dominant, especially in underground railway track systems. Problems related to fatigue behaviour or to the overall safety of a structure arise more generally in bridges and are not treated in this chapter.

This chapter on vibrations induced by traffic and construction activities is structured into the following sub-chapters:

4.1 Roads
4.2 Railways
4.3 Bridges
4.4 Construction Work

Fundamentals are given in the appendices. An important reference is made to Appendix J with its recommended tolerable values of vibrations for structures.

4.1 Roads

W. Ammann

4.1.1 Problem description

Vehicles apply dynamic forces directly to the pavement. The forces are transmitted through the pavement to the sub-structure or through the subsoil to buried or even adjacent structures (see Figure 4.1a). A vehicle is a complex dynamic system which interacts with the pavement and the subsoil or sub-structure (Figure 4.1b). The magnitude and frequency content of the induced vibrations are dependent amongst other parameters on:

- mass of the vehicle m
- speed of the vehicle v
- vibrational behaviour of the vehicle
- characteristics of the tyres of the vehicle
- roughness of the highway pavement (asphalt or concrete layer)
- stiffness of the pavement or sub-structure (see also Figure 4.2)
- properties of the subsoil
- distance of the highway from affected structures.

4.1.2 Dynamic actions

Unevenness of the road surface and speed of the vehicle are the primary causes for vibrations as a result of the vehicle interacting with the pavement. The resulting interactive dynamic force consists of short-pulse impacts. Local and global deflections of the pavement and of the subsoil or sub-structure (see Figure 4.2) under the weight of the vehicle give rise to excitations of longer duration.

Asphalt and concrete pavements must be designed for wheel loads, temperature effects, etc. A continuous pavement is assumed to be a flexible plate resting on an elastic half-space. Segmented concrete pavements may be stressed due to different geometrical levels of adjacent segments. This leads primarily to damage of the joints where impacts of vehicle tyres initiate waves that are then transmitted through the soil to adjacent structures (Figure 4.1a).

Acceleration and deceleration of the vehicle leads to additional dynamic forces transmitted by shear in the direction of the vehicle's movement and these have to be taken into account primarily for pavement design near bus stops, junctions, etc.

Vibrations caused by the vehicle-pavement interaction may be transmitted through the subsoil to adjacent structures (Figure 4.1a). This transmission is governed by soil dynamic characteristics (see Appendix B). The vibrations that are observed at a remote point as a result of road traffic can be most suitably characterized by frequency spectra. In general, the resulting velocity spectrum is practically uniform across the frequency band from 1 to 200-400 Hz and does not - in contrast to rail-induced vibrations - exhibit pronounced frequency components.

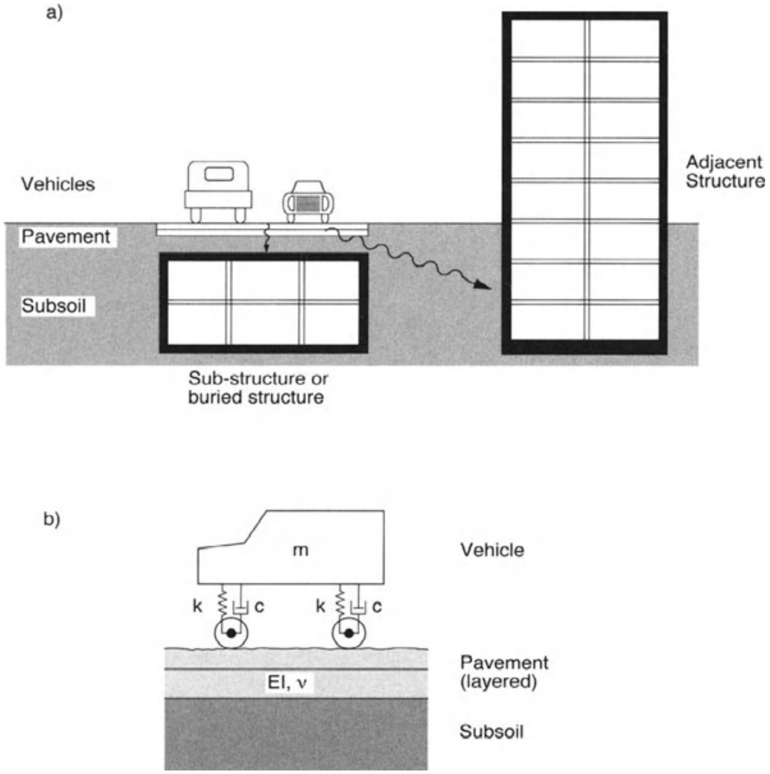


Figure 4.1: Dynamic excitation by vehicles: a) Excitation of a buried substructure and an adjacent structure through pavement and subsoil, b) Dynamic system of vehicle, pavement and subsoil

4.1.3 Structural criteria

Buried sub-structures or adjacent structures may be affected by the dynamic excitations. Considerable resonance effects may take place in the higher storeys of such structures (up to a factor of 5 and more). Floors with a natural frequency in the range from 8 to 25 Hz are particularly sensitive to this excitation. This applies also to the furniture placed upon the floors, which can also exhibit resonance effects. The damping ratio in such floors is usually rather modest (ζ = 0.01 to 0.02) and thus resonance amplifications are substantial and vibrations decay only slowly.

Tunnels for road traffic are rarely affected as long as the evenness of the asphalt pavement or of an inserted concrete slab are guaranteed. Special care has to be taken with highway systems integrated into structures for housing, hotels, assemblies, etc. Structure-borne sound problems may be greater than structural vibration problems.

4.1.4 Effects

Excitations due to road traffic primarily affect:

- structures (impaired functionality)
- persons (annoyance from vibrations and structure-borne sound)
- installations and other building contents.

Also, as a result of noise directly generated by vehicle vibrations, windows and other objects can rattle.

Fast moving vehicles may also cause wind forces especially on tunnel diaphragms or traffic lights and signals, possibly leading to fatigue failures.

4.1.5 Tolerable values

The main reference measure for the vibrations induced by road traffic is the velocity of vibration, and this is usually measured in the foundation of the affected structure.

To assess the effects on persons, installations and other building contents, however, it is not advisable to rely solely on vibration velocities measured in the foundation area of an affected structure. It is preferred to measure values directly at the affected places (i.e. at the various storeys) and to make a final assessment based on, for example the ISO-Standards or the KB-values ([DIN 4150], see also Sub-Chapter 2.1 and Appendix I).

For a general assessment of the effects on structures, different standards can be used. The values of tolerable velocity of vibration in the area of the foundation varies considerably, however. Table J.3 shows Swedish reference values (see [4.1]) for the vertical particle velocity relative to the building type and foundation conditions. The values given are very low in comparison with those of the Swiss Standard [SNV 640312] presented in Table J.2.

4.1.6 Simple design rules

Due to their random nature, vibrations induced by traffic can usually only be described qualitatively. When designing to minimize vibration emissions it is important to find an optimum cost-benefit relationship. It must not be forgotten that as a rule only a limited amount of engineering analysis is possible. Thus - if any calculation is done - the method employed has to be

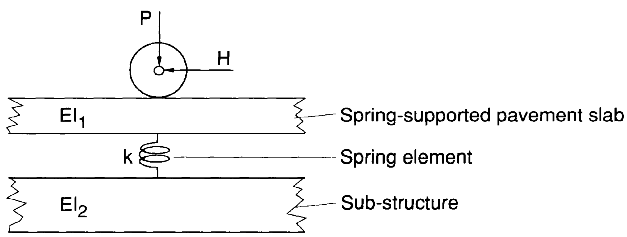


Figure 4.2: Simplified system of a spring supported slab on a sub-structure [4.2]

simple, safe and efficient. Instead of the calculations it is probably better to find improvements to the vehicle and the pavement or to look for other solutions in design.

The following measures are possible:

- *traffic:* limit on wheel load and vehicle speed
- *road:* repair, renewal or replacement of pavement or surface, insulation of pavement from supporting sub-structure or buried structures and from adjacent structures, separating the road from the structure by trenches to reduce the direct transmission of vibrations, repair of concrete pavement joints.

4.1.7 More advanced design rules

A number of detailed calculation methods are available, but their use is only justified if the measures given in Section 4.1.6 do not lead to the desired attenuation. Soil dynamic theory may be used to evaluate wave transmission from the emission source to the foundation of a given structure. Simplified dynamic models, (e.g. two or more degree-of-freedom systems, see Figure 4.2) may be used for affected structures.

One of the most efficient measures to protect a buried sub-structure (see Figure 4.1a) from vibrations is the system of the floating pavement slab. The floating slab system utilises elastomers, natural unvulcanized rubber, etc. as a flexible material between the underlying sub-structure (often also a slab) and the overlying pavement slab. Figure 4.3 shows a typical example. Special attention must be paid to the waterproofing and thermal insulation. The flexi-

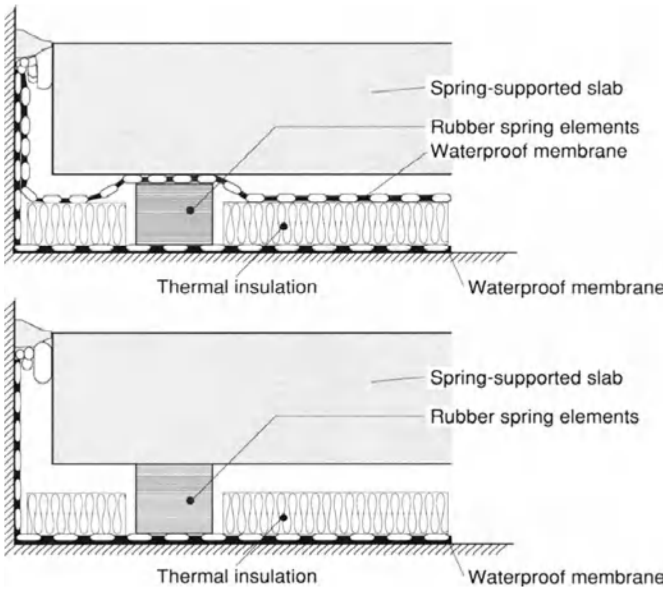


Figure 4.3: Spring-supported slab structure with a waterproof membrane on the sub-structure [4.2]

ble compliant material, depending on the application, is installed either over the whole surface, in strips or at points. For the dynamic analysis of the efficiency of the system a simplified model based on Figure 4.2 may be used. Decoupling of the spring supported pavement slab and the sub-structure can lead to further simplifications.

Large floating slab systems can only be constructed in situ. According to [4.2], if the lost-form-method is used to cast the slab, two types of construction can be used (see Figure 4.4):

a) Prefabricated, thin concrete panels can be placed on flexible elastic point supports or strip elements. In order to avoid vibration short circuits, which will impair the efficiency of the floating slab system, the slab joints have to be well sealed before casting. The disadvantages of this method are, on the one hand, that the base has to be levelled to millimetre accuracy, which is very difficult to achieve, and on the other hand, a relatively large part of the effective depth is lost.
b) Corrugated steel sheeting must be positioned on strip-shaped flexible springs, e.g. rubber spring elements, since a useful bending stiffness exists only in the direction of the corrugations. The stiffness of the sheet is still small enough to allow it to bed into the underlying elastomeric springs and thus guarantee a uniform loading. Irregular edge parts are easier to handle with corrugated steel sheeting than with prefabricated concrete panels. The total thickness of the floating slab is practically the same as the effective depth.

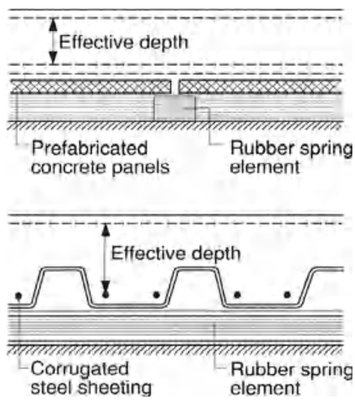


Figure 4.4: Floating slab constructed with the lost-form-method: a) prefabricated concrete panels, b) corrugated steel sheeting [4.2]

4.1.8 Remedial measures

Further measures to eliminate or reduce vibration effects include:

- tuned-mass dampers (installed on the sub-structure, see Appendix D)
- trench barriers (open, concreted, etc., see [4.3])
- frequency tuning (stiffening of the substructure by varying the moment of inertia or the span length).

4.2 Railways

F. Deischl, J. Eisenmann, L. Steinbeisser

4.2.1 Problem description

Noise and vibration caused by rail traffic can be transmitted through the air (railway lines at ground level) and through the soil (railway lines at ground level and underground). In adjacent structures the induced vibrations in the frequency range of around 8 to 20 Hz mainly affect timber ceiling structures or steel girder construction. Excitation frequencies of railway induced vibrations between 40 and 80 Hz may cause significant acoustic phenomena (structure-borne sound, [4.4], [4.5]).

4.2.2 Dynamic actions

Dynamic forces and thus vibrations and noise are caused by discontinuities and irregularities of both the running wheel and the rail- and track-construction. The critical frequency band extends up to about 150 to 200 Hz. If tunnel structures are affected by traffic induced vibrations, elastic waves are sent out into the surrounding ground which may force the foundations of adjacent structures to oscillate.

In addition, direct vibration effects due to vehicle-substructure interaction have to be taken into account. In general, they are treated in the relevant guidelines (see also Sub-Chapter 4.3). These effects include:

- dynamic impact on the sub-structure
- transverse horizontal excitation due to centrifugal forces and hunting.

4.2.3 Structural criteria

The behaviour of buildings when excited by rail traffic is almost the same as when excited by road traffic (see Section 4.1.3). However, as compared with road traffic, structure-borne sound may be increased due to a more pronounced level of excitation in the frequency range from 40 Hz to 80 Hz.

4.2.4 Effects

Excitations due to rail traffic may have the following effects:

- Resonances in buildings, especially due to low frequency excitations, may annoy people. The resulting, amplified vibrations may even cause malfunction and damage to delicate equipment.
- Resonance of parts of buildings, mainly ceilings and walls in a frequency range of 40 to 80 Hz, which may then act as a sound source (structure-borne sound [4.6]).
- Directly radiated noise may cause windows and other objects to rattle.

Damage to buildings or dynamic consolidation of subsoil as a result of operation of underground railways and insufficient bearing capacity of the sub-structure due to impact effects occur only very rarely.

4.2.5 Tolerable values

Regulations or codes for the limits of vibration and noise in buildings caused by underground railways exist only for general cases. Therefore in practice some guidelines are used, such as [VDI 2058] or [DIN 4150/2]. For the assessment of noise the usual dB(A)-values are used (see Appendix B), while low frequency vibrations as they affect people are classified by their so called KB-value (see Appendix I), which is derived from the velocity of vibration. As an example, these guidelines provide for the following limits for night-time between 22h and 6h:

- *Vibration limit:* 0.1 - 0.3 (KB) depending on the site of a building (e.g. industrial site or residential district). Here the problem is that human susceptibility to vibration varies greatly. There are persons with a limit in perception well below the band of 0.1 to 0.05 (KB). The limit of 0.15 (KB) will generally not be exceeded if the level of vibrations - taken on the foundation or basement wall - is lower than 40 to 45 dB_v on structures with timber ceilings or 50 dB_v with concrete ceilings (dB_v: dB-value related to the vibration velocity).
- *Noise limit:* usually 35 dB(A) peak value. This limit is of course much too high for concert halls, churches and similar buildings. Depending on the structure, the value of 35 db(A) will generally not be exceeded if the level of vibration at the foundation or basement wall is lower than 45 to 50 dB_v.

4.2.6 Simple design rules

a) General aspects

While vibrations can be measured precisely at a specific site, transmission from a place of emission to the site of the receiver (place of immission) is still difficult to predict. The most effective and economical measures for reducing ground transmitted vibrations are those performed on the track. At every critical location of a specific line certain specific measures can be carried out. Measures on the rolling stock primarily influence direct acoustic emission.

The need for emission reduction and immission protection has given rise to a lot of investigations of the causes of noise and vibration and the measures to reduce them. For almost every vibration problem effective measures are now available. For underground railways with axle loads ranging from 100 to 210 kN, guidelines for the application of protective measures have been established.

b) Measures against increased vibration level

In order to reduce the noise and vibration level in tunnels or at ground level the following measures need to be taken into account:

- *Rail surface:* Short and long pitch corrugations, insulating joints, turnouts etc. are frequently the cause of high noise levels and vibrations in the higher frequency range (100 - 200 Hz). A corrugated rail, for example, may raise the noise level by as much as 10 dB(A).

The same increase occurs on turnouts. In general good condition of the rail surface is important. High-strength rails may make maintenance less troublesome. Switches, points and crossings could be equipped with movable frogs. The great increase in service life may compensate for the higher initial costs.

- *Damping measures on the rail web*: Investigations have shown that covering the rail web with damping material decreases the noise level in a tunnel only by about 2 to 5 dB(A). This means that normally such small improvements are not cost effective.
- *Highly elastic (flexible) rail fastenings:* Highly elastic fastenings of the rail to the steel ribplate permit a larger deflection of the rail under the wheel; this reduces the mechanical impedance of the superstructure, which in turn reduces the excitation of vibrations. The limitations here are set by the fatigue strength of the rail, the geometric gauge widening and misalignment in the case of rail fracture. The attenuation effect here begins at about 30 Hz and reaches 6 - 10 dB at about 50 Hz.
- *Variation of thickness of ballast:* Ballast carries the track with wooden or concrete sleepers. Normally ballast is about 30 cm thick. An increase of ballast thickness has no measurable effect, while decreasing the thickness below 30 cm leads to a noticeable deterioration. There is also substantial difference between a recently tamped ballast and a ballast which has not been tamped for a long time; this again places emphasis on the necessity for track maintenance. Control measures for existing vibration levels should not be implemented shortly after a tamping procedure.
- *Sub-ballast mats:* Sub-ballast mats are elastic layers placed under the ordinary ballast bed (see Figure 4.5). A variety of products is available. Problems can arise with respect to the durability of the mats since this affects the track bed. High efficiency mats (thickness up to 80 mm) lower the vibration level noticeably, beginning at about 16 to 18 Hz and reducing it by 20 dB at 50 Hz. This reduction may even be increased by about 4 dB if the thickness of ballast is increased from 30 cm to 60 cm. (Such an increase in ballast thickness would be useless without sub-ballast mats).

4.2.7 More advanced design rules

The most efficient, but also the most expensive, solution is the appropriate design of a mass-spring system, for specific cases also called a floating-slab-system [4.7]. The basic concept is to prevent vibrations from penetrating into the soil by inserting a linear harmonic oscillator with a very low natural frequency (see Figure 4.6). This solution is primarily used for underground railways.

The natural frequency should be as low as possible in order to let the attenuation of vibrations start at the low end of the excitation frequency spectrum. Due to practical limitations one cannot go below a fundamental frequency of the system of 5 Hz, but it is also important not to exceed 14 Hz. Normally the calculated natural frequency of a mass-spring-system lies between 8 and 12 Hz, and this provides a sufficient gap from the natural frequency of the vehicles (1 to 2 Hz) on the one hand, and the frequencies of structure-borne sound on the other hand.

Many different types of construction have been built in the last few years: for example, tracks with and without ballast (noise problems in trains and stations) laid on prefabricated or cast-

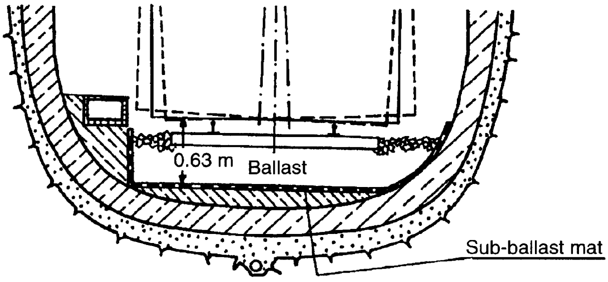


Figure 4.5: Ballast-track with sub-ballast mat

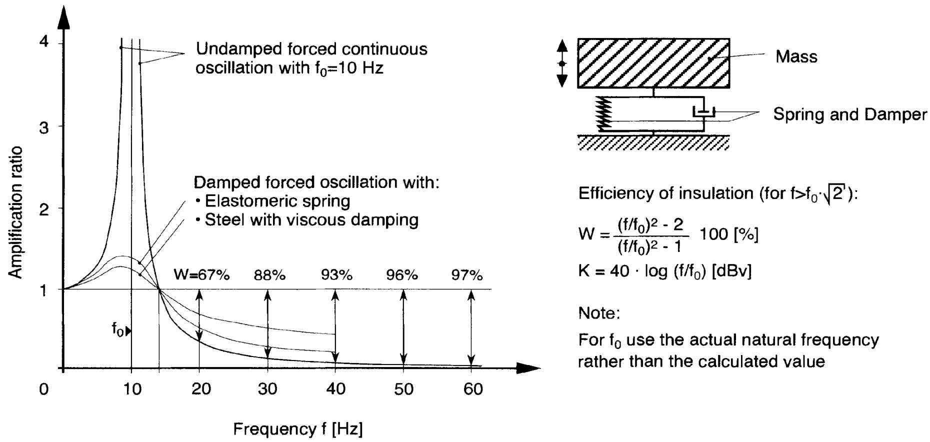


Figure 4.6: Dynamic amplification ratio of a mass-spring-system with a natural frequency of f_0 = 10 Hz

in-place slabs. The total mass ranges between 4'000 and 9'000 kg/m, depending on the axle load of the train (120 to 210 kN); a total height of 0.80-1.40 m is required for this type of construction, measured from the top of the rail to the tunnel base. The shape of the slabs or troughs is usually adapted to the cross section of the tunnel and to the gauge limit of the trains (see Figures 4.7 and 4.8).

Care should be taken to design this slab correctly in order to avoid low frequency resonances of the slab itself. For the dimensioning of the slabs as well as the bearings, the type of bedding is important - a single-poin- bearing-support with spacing up to 4 m is as efficient as a continous supporting medium between slab and tunnel base (see Figures 4.7 and 4.8).

If buildings with timber ceilings having low natural frequencies need to be protected, this will force a reduction of the natural frequency of the mass-spring-system for the slab down to 5 to 7 Hz. While this is technically possible, it increases the costs. The calculated natural frequency of the mass-spring-system should be well below the natural frequency of the timber ceilings (by a factor of 1.5 to 1.7). Similar problems arise with the fundamental frequency of large-span tunnel ceilings.

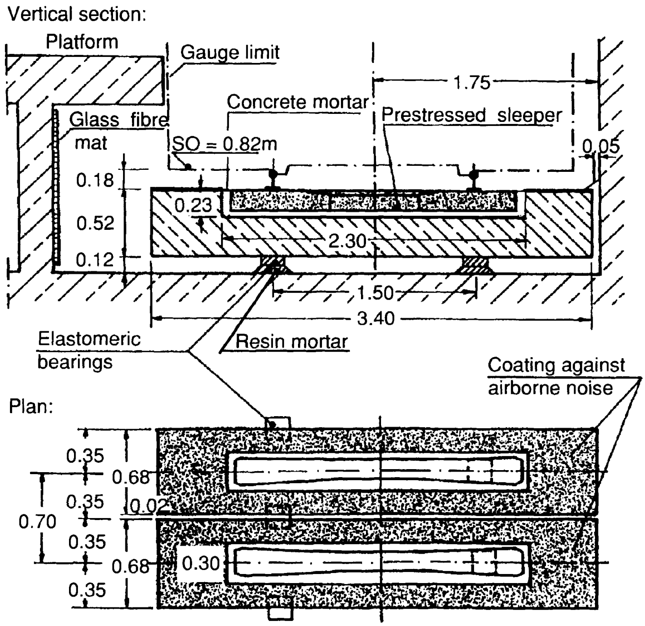


Figure 4.7: Mass-spring-system without ballast; built-up with prefabricated units

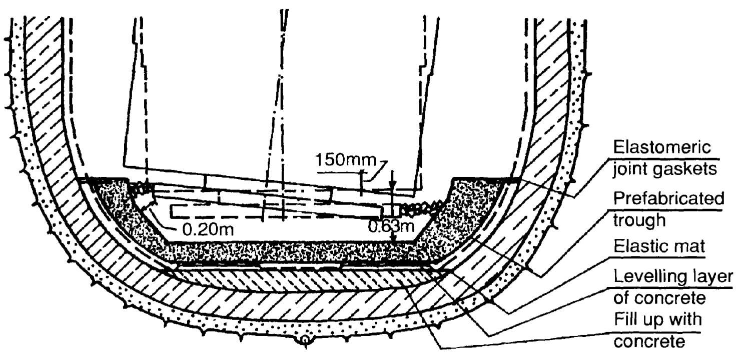


Figure 4.8: Mass-spring system with ballast track

The dimensioning is carried out straightforwardly, modelling an elastically supported continuous beam on a half-space, for example; the force is assumed to have a static component which is multiplied by a dynamic factor in order to take into consideration wheel impacts (1.5 to 1.6) and centrifugal effects (1.1 to 1.2).

The "spring" usually consists of elastomeric elements (bearings). They should have good linearity over the whole loading range. From the load-deformation diagram of the bearings the spring stiffness (c = load/deflection, [kN/mm]) - or the modulus of reaction (C = surface pressure/deflection, [N/mm^3]) - is evaluated at the load-deflection point corresponding to the weight of the construction. If the damping of the spring material is neglected, the natural frequency can be calculated as follows:

$$f_0 = \frac{5}{\sqrt{w}} \text{ [Hz]} \tag{4.1}$$

where w = dead load deflection [cm].

For example, a deflection of 3 mm under the weight of slab and superstructure results in a calculated natural frequency of 9.1 Hz. The actual natural frequency lies about 10 to 20% higher due to nonlinear effects, depending on the elastomeric material used. In order to avoid overstressing of the rails and the elastomeric elements, the additional deflection caused by the axle loads of the train should not exceed double the value that results from the static preload.

The mass-spring-system is the most efficient measure for reducing all noise and vibration problems, but it needs to have its fundamental frequency in the right range. When a mass-spring system is used with a ballasted track, the lateral displacement in curves due to the centrifugal forces should be kept within limits. A value of 2 to 3 mm should not be exceeded.

4.2.8 Remedial measures

For an existing rail traffic system all protective measures mentioned above are feasable, including the mass-spring system. For example, a noisy rapid transit track may be repaired by using a mass-spring system without ballast or with a highly efficient sub-ballast mat. Further developments are also taking place with continuously bedded rails and with elastic rail fastenings. Lastly, a simple "rubber apron" near a tramway track may lead to a sufficient reduction of vibration effects (see Figure 4.9). The efficiency of such rubber aprons has to be carefully examined, however

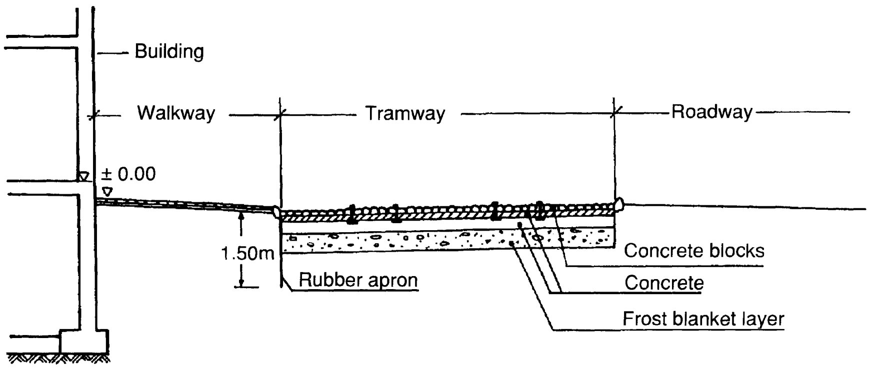


Figure 4.9: Ground-level track; protection with a rubber apron

4.3 Bridges

W. Ammann

4.3.1 Problem description

Besides their static weight, road and railway bridges are strongly affected by dynamic actions. This is due to the vehicle moving over a flexible structure and this gives rise to time dependent variable deflections. In addition all other parameters cited in Section 4.1.1 for road vehicles and in Section 4.2.1 for railway systems remain applicable.

4.3.2 Dynamic actions

For the design of road and railway bridges and other structures supporting moving vehicles it is usual to introduce the design forces as externally applied forces. The dynamic interaction effects between vehicle and substructure are only taken into account by a quasi-static increase of the weight (dynamic increment).

Road bridges have to be designed for the following forces:

- weight increased by a quasi-static factor (dynamic increment)
- deceleration (braking) forces
- accidental impact forces on guidewalls and railings.

Railway bridges also have to be designed for the above-mentioned forces and, in addition, for:

- derailment forces
- centrifugal forces in curved bridges
- acceleration forces
- hunting forces.

These forces act in different directions and at various locations. Codes and standards generally specify the degree of superposition of each of these different forces.

Many codes and standards contain information on the (quasi-static) dynamic increment required for design. For road bridges this increment is highly dependent on the roughness of the pavement. Figure 4.10 shows the relation between the dynamic increment and the fundamental frequency of concrete bridges with ordinary pavement roughness [4.8]. The values given in Figure 4.11 apply for a situation with one specific roughness on the bridge deck, e.g. vehicle driving over a plank. Some peak values of the dynamic increment can be as much as 300%. For design purposes an increase of 80% of the weight of the bridge may be used.

For railway bridges the dependence of dynamic increment on the span length is more pronounced. In addition, the design speed of the train may be taken into account. For design purposes the maximum dynamic increment should not exceed 70%. The centrifugal force component depends mainly on the square of the design speed and the radius of curvature.

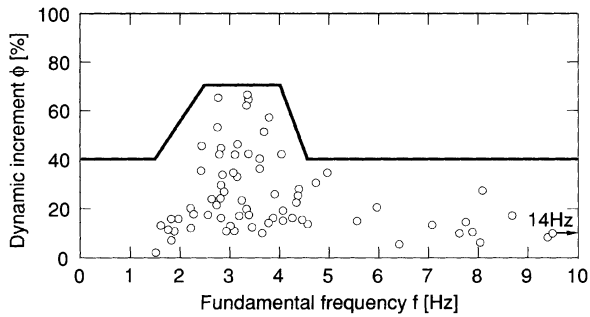


Figure 4.10: Dynamic increments as a function of the fundamental frequency f based on 73 values for concrete bridges with ordinary surface roughness [4.8]

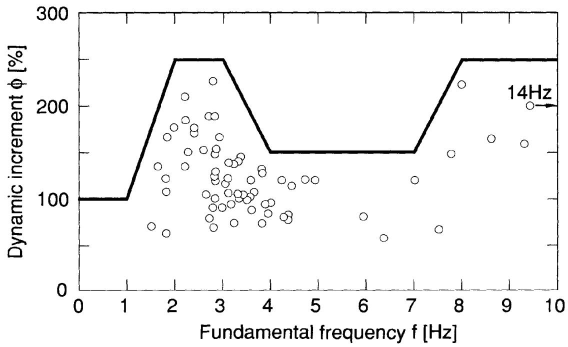


Figure 4.11: Dynamic increments as a function of the fundamental frequency f based on 69 values of concrete bridges provided with an artificial bump (plank) [4.8]

Besides the quasi-dynamic forces that influence the design of a bridge, structural vibrations occur due to vehicles moving on a flexible structure and exciting various natural modes of vibration of the bridge. Bumpy irregularities of the pavement, pronounced rail corrugations, expansion joints, inspection covers, etc. lead to impacts that excite a broad band of natural modes. However, a quantification of this type of dynamic force is not generally possible.

4.3.3 Structural criteria

As shown previously the quasi-static dynamic increment affects the design of a bridge structure. In addition - and especially for slender structures such as cable-stayed bridges - vibrations excited by moving vehicles may lead to fatigue problems in suspension cables, to a decrease in bond between pavement and structure, etc.

Vibrations transmitted from the bridge deck through the column foundations or through abutments to the ground may cause excitations in adjacent structures leading to similar problems as stated in Sub-Chapter 4.1.

The fundamental frequency of bridges is in general rather low and depends of course strongly on the span length and the type of bridge construction. For highway bridges, Figure 4.12 shows that the fundamental frequency may be simply estimated as

$$f = 100/L\,[\mathrm{Hz}] \tag{4.2}$$

where L = span length in [m]

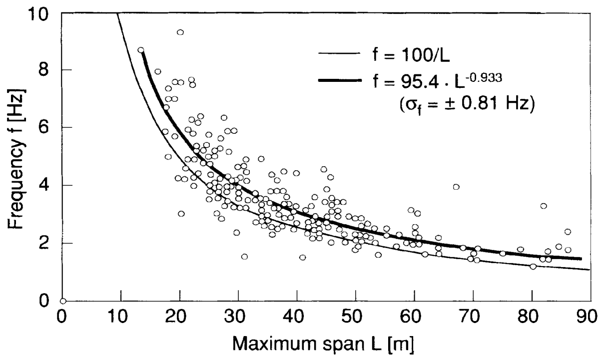


Figure 4.12: Fundamental frequency f as a function of the maximum span L, derived from 224 bridges [4.8]

Many of the bridges lie within the frequency range of 2 to 4 Hz, where according to Figure 4.10, the maximum dynamic increment occurs. The damping of bridges is again very much dominated by the type of bridge construction, but in general should not exceed ζ = 2%.

4.3.4 Effects

The vehicle-bridge interaction may cause the following effects:

- endanger the load-carrying capacity of the bridge if not designed for appropriate forces
- impair the fatigue behaviour of the bridge and its installations if not designed for appropriate forces
- impair the safety of the vehicles
- impair the serviceability of the bridge and its installations
- impair the serviceability of adjacent structures by indirect excitations.

In addition, acoustic phenomena (structure-borne sound) can be induced. Also, as a result of noise generated directly by the vehicles, windows and other objects in adjacent structures can begin to rattle.

4.3.5 Tolerable values

Whereas the dynamic increment has to be taken into account in the whole design procedure, effects which impair the serviceability are examined based on resulting structural vibration velocities. To assess the effects on pedestrians and installations or on the drivers or occupants of vehicles it is advisable to take the measurements at specific affected locations on the bridge deck. It is suggested that the same tolerable values as cited in Sub-Chapter 4.1 or also in Appendices I and J be used.

4.3.6 Simple design rules

Road and railway bridges have to be designed for the forces cited in Section 4.3.1. Simple measures to prevent structural vibration problems in adjacent structures would include the same ones as presented in Section 4.1.6.

4.3.7 More advanced design rules

A number of detailed calculations and design methods are available for the global design of a bridge. In situations where the vibration characteristics of a bridge is of dominant interest - e.g. fatigue problems - a full dynamic analysis is highly recommended.

4.3.8 Remedial measures

Measures to eliminate vibration effects on a bridge deck or on adjacent structures include e.g.:

- frequency tuning by stiffening the bridge deck with external reinforcement or a reduction of the bridge deck mass: track without ballast, minimal pavement, etc.
- tuned-mass dampers installed under the bridge deck.
- damping elements between suspension cables or between cable and bridge deck (especially against wind galloping: see Chapter 3 and Appendix H).

4.4 Construction Work

G. Lande, W. Ammann

4.4.1 Problem description

Many activities in construction work lead to vibrations in the neighbourhood of the construction site or even to far-field vibrations. The following construction activities and machinery are associated with possible vibrations:

- vehicles on construction sites
- piling, sheet piling
- vibratory compaction
- dynamic consolidation with dropping mass
- excavation by heavy equipment
- blasting.

In addition, vibrations are induced by:

- exploration of the subsoil by dynamic testing
- stationary heavy machinery (sorting equipment)
- air-borne vibrations, etc.

Construction work usually initiates very complex vibrations. In general, it is possible to classify vibrations induced by construction activities into three main categories:

- soil or rock related vibrations (with single or multiple sources)
- structure-borne vibrations
- air-borne vibrations (noise).

4.4.2 Dynamic actions

While deterministic vibrations can be described by mathematical expressions and therefore predicted, this is practically impossible with regard to random vibrations and noise which can only be described by statistical means. The motion is said to be random even if it obeys in many cases known natural laws. Generally there are many cooperating sources and/or influences that makes it practically impossible to view the situation in detail. The most common terms used to describe random vibrations and noise are root-mean-square values of vibration quantities, power density spectra, correlation functions, and probability density functions.

Transient vibrations of the deterministic type never exist in the strict sense. Many vibrations that occur in practice can, however, be relatively easily approximated with an exponentially decaying sinusoidal vibration. Examples are structural vibrations when the building has been excited by a shock of short duration from pile driving or by internal shocks.

Machines and tools used for construction work generate vibrations which propagate through the soil, foundations and buildings in the form of elastic waves. Such waves also appear in the soil due to piling and blasting.

In order to predict what may happen with a structure at a certain distance from an excitation source, one must know the existing types of waves, how the motion is transmitted and how the structures are affected. The most important waves in this context are shear waves (S-waves), compression waves (P-waves) and surface waves (Rayleigh- or R-wave, Love- or L-wave). Figure 4.13 shows typical ranges of P- and S-wave velocities for different subsoil conditions. Additional information may be found in Appendix E.

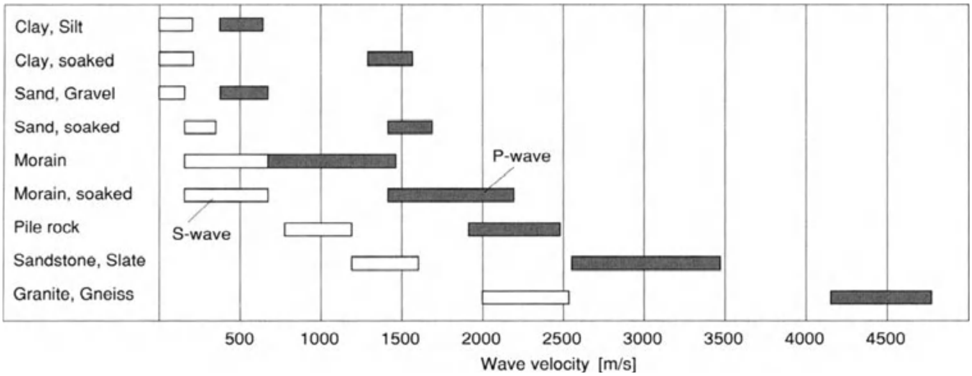


Figure 4.13: Typical ranges of P- and S-wave velocities for different subsoil conditions

4.4.3 Structural criteria

As stated in Appendix E the wavelength is a parameter of great significance for the influence of vibrations induced by construction work on different structures. In many cases wavelengths in the order of 10-30 m are of practical importance. These are relatively disadvantageous ranges because in many cases buildings have dimensions that are comparable to or greater than these wavelengths. However, it is not a random phenomenon that makes this wavelength range dominant. A vibration source with wavelengths greater than the dimensions of a structure does not generally induce vibrations in that structure. Waves with short wavelengths are on the other hand damped out rapidly with increasing distance. This is not the case for surface waves (R- and L-waves), however, which can also have wave-lengths longer than 100 m.

In most cases, construction activities have to be performed in areas occupied by existing structures. Vibrations induced by construction activities mainly influence walls, ceilings and secondary elements of these existing buildings with a main frequency range between 10 and 30 Hz. Thus these structural elements will probably suffer from resonance excitation.

4.4.4 Effects

The effects on people are similar to those stated in previous chapters: disturbance and annoyance of people due to the vibrations transmitted through the foundations into the structures and, further on, to equipment.

The damaging effect of vibrations on structures is mainly expressed in terms of vibration velocity. This is due to the fact that for high frequency vibrations stresses and strains induced in structures are proportional to the particle velocity of the structure or foundation. Amplifications of vibrations from foundation to upper levels within a structure have to be taken into account.

In the frequency range from 10 to 20 Hz, secondary structural elements and equipment can suffer resonance, especially ceilings. Structural damage, implying a risk for the entire building structure, will not occur other than in very exceptional cases. On the other hand fine cracks can occur in ceiling corners, especially if they are painted or plastered. However, such damage can usually be classified as cosmetic.

To systematize the effects on structures, structural damage caused by vibrations can be divided into three categories:

- direct vibration damage
- accelerated ageing
- secondary vibration damage.

Direct vibration damage is damage that is generated directly by vibrations in a structure that was previously undamaged and was not exposed to an abnormal state of stress.

Accelerated ageing is due to the fact that most of the structures that are founded on soil are damaged to some extent during their life by static settlement of the foundations and that dynamic stress from vibrations can accelerate the development of this damage. It should be noted, however, that this type of damage may even arise from normal use of the building, e.g. by people walking, running, dancing, moving furniture, slamming the doors etc. In addition, climatological factors, such as wind and variations in temperature create increased stresses in the structure.

Secondary vibration damage to structures may occur when a heavy vehicle passes near a structure situated in an area of soft soil. The stress from the vehicle is distributed over a wide area around the vehicle because of the low stiffness of the sub-structure or the subsoil. It may cause similar damage patterns as from (static) differential settlement.

In addition, vibrations can in some cases cause compaction of loose subsoil leading to settlement that can result in subsequent structural damage in the buildings. This is also an example of indirect vibration damage of buildings. The influence of the vibration acceleration on the degree of compaction of a soil sample is shown in Figure 4.14. It appears that already for an acceleration of 0.1 g (1 m/s^2) there is a decrease of the void ratio by 7-8%.

Direct damage to structures may also be influenced by the subsoil vibrating in resonance with incoming shearwaves resulting in pronounced low frequency vibrations which may then excite the structures as a whole at their fundamental frequency.

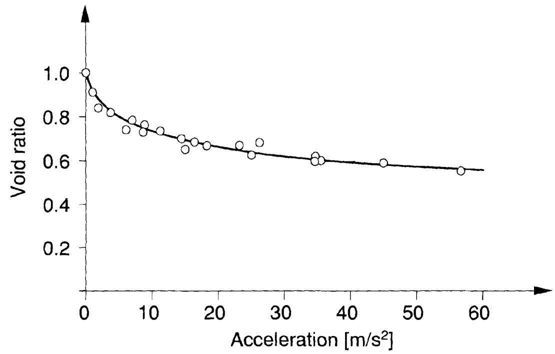


Figure 4.14: Relation between the void ratio e and applied vibration acceleration in laboratory tests of a sand sample

Determination of the dominant frequency of the subsoil may be required. As a first approximation the dominant frequency of the shear-wave can be described by:

$$f = c_s/4H \tag{4.3}$$

where c_s = shear wave velocity
H = thickness of dominant subsoil layer

4.4.5 Tolerable values

For various types of vibration sources there are only a few generally accepted standard values for tolerable vibration velocities since the number of well-documented damage cases is very limited. Vibrations from piling, sheet piling, soil compaction and traffic related to construction work, etc. seldom reach levels where they can cause direct vibration damage. In the majority of such cases it seems that damage can be classified in general as accelerated ageing resulting in cracks or other "cosmetic changes".

Much experience with construction activities and with vibration measurements is necessary in order to recommend suitable and realistic tolerable values for vibration exposure. Severe restrictions, e.g. low values of particle velocity for blasting, can increase the excavation costs considerably. Therefore it is important first to carry out visual inspection of the structures concerned and then to estimate the sensitivity of the buildings and their foundation to vibrations. Many factors influence the threshold values for vibration damage. Some of the most important are:

- vibration resistance of the building materials
- general condition of the building
- duration and character of the vibrations
- presence of equipment sensitive to vibrations
- foundation of the buildings (type, condition)
- propagation characteristics of the waves in rock, soil and building material.

Since blasting has been a major cause of damage in buildings, research work concerning the influence of blasting has been extensive. For that reason more information on blast-related structural damage exists than for other types of construction work. Figure 4.15 shows an overview of proposals and recommendations [4.9], [4.10].

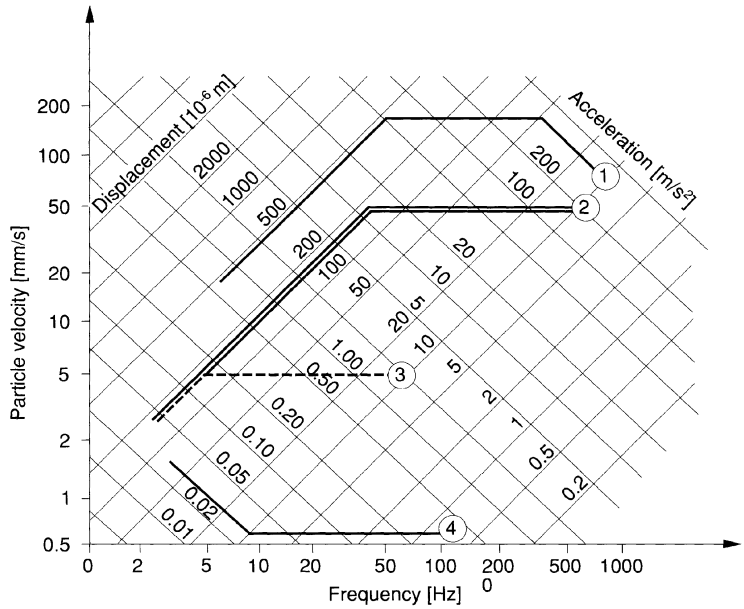


Figure 4.15: Vibration criteria for damage and recommended values:
1) Damage in buildings from vibrations caused by blasting; 2) Recommended upper limit for blasting; 3) Recommended upper limit for piling, sheet piling, vibratory compacters, dynamic deep compaction and construction-site traffic; 4) Disturbing vibrations for people

[4.11] contains a compilation of the results of vibration measurements carried out in Switzerland after 1960. The scope was to determine criteria for damage to buildings caused by vibration. The resulting standards contain criteria for four classes of structures for vibrations caused by piling, vibration compaction, and traffic. These values are given in Table J.1.

Other recommended tolerable vibration values for vibratory compaction, determined from literature and experience were published in [4.12]. These recommended values are compiled in Table J.2.

Information about tolerable vibration values for vehicle traffic is given in [4.1] and compiled in Table J.3. The buildings are assumed to be founded on clay or loosely layered sand. The recommendations refer to the case when the number of vehicles with a total weight of 10 tons exceeds 50 vehicles per day. If the number is lower, the above mentioned tolerable values can be increased by 50%.

Table J.4 presents tolerable values, commonly used in Sweden for assessing the risk of damage caused by ground vibrations in normal residential buildings [4.13]. Normal residential buildings imply houses with foundations and floors of concrete, outer walls of brick and intermediate partitions of plastered, compact light concrete.

In connection with blasting near telecommunication offices, relay stations or buildings containing other sensitive equipment such as computers, electron microscopes, scales, grinding machines, turbines etc. attention should also be paid to the acceleration levels generated. In this respect recommendations are proposed in [4.14] e.g. for telecommunication offices: $v \leq 50$ mm/s, and $a \leq 10$ to 30 m/s^2 , depending on the type of structure or for television stations: $v \leq 35$ mm/s, $a \leq 30$ m/s^{2}and for power plants with respect to relays: $a \leq 5$ to 20 m/s^2.

Owners and manufacturers of computer systems are very restrictive concerning permissible vibrations and therefore considerable cost increases and delays in construction work can occur when blasting must be carried out in the neighbourhood of such vibration sensitive equipment. Limit values given by manufacturers, suppliers and users of computer equipment are defined in various ways (accelerations, displacements, RMS-values, peak values). In many cases it is not possible to determine whether the values given are related to a certain shock profile or to continous vibrations, etc. Often it is also not clear where to measure the incoming vibration. Disk storage units are usually the most sensitive parts of a data processing system. For several years damage threshold values as low as 0.25 g have been used. These very low values sometimes cause severe problems for a contractor.

4.4.6 Simple rules

a) Vehicles on construction sites

Vehicles on construction sites generate structural vibrations due to two main mechanisms. One is the depression around the vehicle because of its weight. The second is the rapid change of pressure on the pavement or ground from every wheel as the vehicle rolls on the roadway. In this manner energy is continuously transmitted from the vehicle to the ground. If the roadway is bumpy, shocks are induced and the energy propagates then to the sur- roundings in the form of elastic waves. Surface waves of the Rayleigh-type dominate, but there are also vertically polarised shear waves (SV-waves).

As heavy vehicles are often long and all the wheels function as vibration sources, the vibrations have a long duration. They also have rather low frequencies and can therefore cause vibration of both structures and secondary elements. That is why even relatively small vibration levels in the framework of the building can be felt and become annoying.

Measures to reduce vibrations from construction vehicles are e.g.

- vehicles: restrictions of type of vehicle and speed
- road : surface smoothness, isolation from the ground
- soil : screening, defocusing, spreading, shielding, damping.
- transmission to and within buildings: foundations on solid stiff ground, isolation and damping.

b) Piling, sheet piling

The driving methods normally used for piling and sheet piling are:

- ramming
- vibration
- drilling
- after excavation in situ-casting in a tube or ground slot.

Piles and sheet piles can be rammed with light or heavy equipment. Heavy means that the weight of the drop hammer is equal to the total weight of the pile or sheet pile. Examples of heavy impact equipment are drop hammers, diesel hammers, heavy single-acting pneumatic hammers, or steam hammers. Light impact equipment is generally a double-acting pneumatic hammer. Heavy hammers usually beat at 30-60 strokes/min while light doubleacting pneumatic hammers operate at 300-1000 strokes/min. Drop hammers with a weight of 30-40 kN are commonly used, but occasionally some weigh up to 80 kN.

Vibrating equipment for pile driving is usually in the heavy category with a total weight normally higher than the weight of the pile or sheet pile. Most equipment consists of two contra-rotating wheels with equal eccentric masses. The speed is generally 1000-2500 r.p.m., but in some Eastern European countries 500 r.p.m are used in loose soils.

When predrilling is used for piles or interpile sheeting, drilling machines are often used with a hammer lighter than mentioned above. For excavation of ground trenches or in steel tubes a heavy excavation machine is often used. One part of such work can be blasting or chiselling of boulders, hard soils, levelling of rock or chiselling in rock slopes for a pile shoe. For such work heavy chisels weighing 20-50 kN are often used.

Piling and sheet piling work on construction sites affect the environment in different ways depending on local geologic conditions. Usually, piling and sheet piling cause noise, vibration, settlement or heaving close to the site. The effects are dominated by the energy introduced into the subsoil, the distance from the source and, of course, the subsoil conditions.

Settlement occurs in non-cohesive soil, i.e. sand and gravel, when the particles in the soil move into a tighter packed array under a vibrational force created by a pile or sheet driver. The volume of the settlement can be 5 to 50% of piled volume. A simple rule of thumb is that settlement or heaving can occur on the ground within a distance from the pile or sheet driver of around half the pile depth. Heaving occurs in cohesive soils, e.g. clay, when the piles are driven into it. The heaving corresponds to the volume of the rammed piles. Special attention has to be given to the possibility of raising pore-water pressure due to piling because this can create local instability of the ground.

Propagation of vibrations to the surroundings depends on the weight of the ramming equipment, impact velocity, impact duration, shape of the pile, surrounding and underlying soil, cross-sectional area of the pile, straightness of the pile and eccentric or oblique strokes.

Vibrations are rapidly damped in non-cohesive soils but propagate far from the construction site in cohesive soils. Usually, vibratory driving in non-cohesive soils gives less disturbance to the surrounding area than striking with drop hammers. In clay, vibratory equipment generally creates a greater disturbance to the surroundings than a drop hammer. The normal range of frequencies for ground vibrations from piling or sheet piling is 2-15 Hz, e.g. well below the "normal" frequency of vibrations from blasting in rock materials.

Guidelines for tolerable vibration levels from blasting should not be used for evaluation of the severity of vibrations from piling.

The following measures may help to limit vibrations from piling:

- Proper choice of cushion between the hammer and the pile-head.
- Proper alignment between the striking equipment and centering along the longitudinal axis of the pile.
- Prevention of initial bending of piles.
- Driving with low drop-heights in loose soils.
- Use of piles with smallest possible cross-sectional area.
- Use of light impact equipment in non-cohesive soil and a heavy one in cohesive soil.
- Control of penetration per blow.
- Good planning and organisation of piling work e.g. starting and advancing from the building.
- Avoiding vibrating equipment in clay.
- During winter, drilling out the frozen crust before piling.
- Use of tubes for clay cores if risk of unacceptable heaving may occur.
- Use of in-situ cast piles when risk of vibration focussing and settlement exists.

c) Vibratory compaction

Vibrators and vibratory rollers are often used in construction for compaction work. The following types are used:

- tow-type vibratory roller with a weight of 40 to 60 kN. Heavier models, e.g. with 150 kN static weight, are used primarily for extensive compaction of rock fill.
- self-propelled vibratory rollers with pneumatic driving wheels of 40 to 100 kN.
- vibratory tandem rollers with one or two vibratory drums of 15 to 100 kN.
- vibratory plate compactors with a static weight of 1 to 8 kN.

Special precautions are normally not necessary for small vibratory compactors with static weight below 20 to 30 kN. A general rule is that rollers with a maximum drum module heavier than 50 kN should not normally be used on city streets with adjacent buildings.

It is observed that very wide variations exist in safety limits. Factors to be considered include:

Vibratory compaction of different soil profiles may lead to large discrepancies between predicted and measured vibration levels. The strongest ground vibrations are found in silt and clay with a water table near the surface. In frozen soils, the ground vibrations may be stronger than during unfrozen conditions. Frequency can change due to difference in stiffness (see [4.15]).

Resonance phenomena in different parts of a building, such as a chimney-stack, may increase the risk of damage. If the stress limit in a material is already approached, as is often the case in plastered walls, very small additional stress may cause damage. Starting and stopping of the vibratory compactor may temporarily increase the ground vibrations.

As a rule of thumb, a minimum safe distance [m] for vibratory rollers from buildings is 0.15 times the drum module weight [kN]. If there is any risk of damage to buildings, only low amplitude or even static compaction should be used.

d) Dynamic consolidation

In construction of dams, embankments and piers, earth or rock-fill is used as a construction material. Compaction of such soil used for fill can be carried out with a dropping mass and is called dynamic consolidation. The weight may reach 200 kN and even more, with a drop height up to 20 m; this leads to a pronounced input of energy into the subsoil.

e) Excavation

Construction works very often include excavation of many different materials and such work is carried out under many different circumstances. For instance, there is a great difference between the vibration levels generated when excavating clay in the summer or in the winter when the surface is frozen.

Vibrations arise from impacts with the bucket and when boulders etc. are to be loosened. Moving the excavator also causes vibrations but they are generally much smaller and comparable to traffic vibrations mentioned earlier. The vibration duration is usually very short and has the character of a shock, but as the work is repeated the duration may be hours or days. At short distances the vibration levels are low and rarely exceed 5 mm/s.

f) Blasting

At most construction sites where blasting is necessary, there are nearby structures whose sensitivity limits the blasting due to permissible vibration levels. Client or contractor must then determine the maximum charge that can be detonated without exceeding permitted vibration levels in the neighbourhood. Since the cost increases considerably if the charge has to be reduced, financial considerations influence the decisions taken to a great extent. Planning of blasting work should include the following steps:

1. Geological examination of working site and risk area.
2. Investigation of the risk when lowering the groundwater level.
3. Investigation of foundation stability of buildings in the area.
4. Investigation of construction and condition of buildings within the risk zone.
5. Sensitivity analysis of equipment like computers, electron microscopes, relays etc. with respect to vibrations.
6. Examination of any underground objects like tunnels, cable trenches, telegraph cables, oil cisterns, district heating culverts etc. that might be damaged by blasting.
7. Investigation of the relation between vibration values, explosive charges used and distances.
8. Investigation and assessment of structural responses, magnification factors, interaction etc.
9. Information to the inhabitants in the neighbourhood about possible vibration, noise and dust nuisance.

Since rock is not an isotropic medium it is difficult to predict the vibration level from a certain charge at a given distance. It appears that the different types of waves generated are damped differently depending on the rock foliation or bedding. The schistosity, the distance between existing planes of weakness and fluctuations of the groundwater can affect the vibrations. In addition, the vibrations are also affected by the blasting geometry, the performance of the explosives, the detonation velocity, the coupling between the explosive and the rock and the distribution of distances between the charges.

The maximum particle velocity [mm/s] is normally used to express the maximum vibration that the actual objects can withstand (see also Chapter 2 and Sub-Chapter 4.1).

4.4.7 More advanced measures

Generally, during the transmission of vibrations from the ground to the building the particle velocity is lower in the foundation of the building than in the ground. Amplification of vibrations may occur higher up in the building mainly due to resonance effects. It is possible to reduce the vibrations by making a trench between the source of vibrations and the affected object.

Extensive half-scale experiments have been carried out, where vibrations were reduced by means of a trench in fine gravelly sand [4.16]. It is shown that:

- For greater distances from the vibration source, a wider trench is necessary.
- Intensification of vertical vibrations takes place in front of and beside the trench.
- Wall barriers are less effective than open trenches.

Results from full-scale experiments (see [4.17]) with trenches in the ground show that they have a similar attenuating effect on vibrations. The measurements were based on traffic with lorries and buses on a smooth and on an uneven roadway. The following findings were made for a trench of depth x and width y:

- The maximum particle velocity was reduced by 60 to 70% at a distance of 10 to 40 m behind the trench compared with the particle velocities measured in front of the trench.
- To reduce transmission of vibrations, the trench should be filled so that the acoustic impedance for the infill is as low as possible in relation to the surrounding ground.
- The positioning of the trench should be as close as possible to the disturbing source. The surface waves at this point have higher frequencies and are not yet completely developed; therefore the trench can be made shallower.

Vibration damage associated with excavation work seldom occurs today due to use of modern excavation techniques and equipment; nevertheless, vibration control can sometimes lead to considerable expense, especially in areas with an accumulation of vibration sensitive installations and equipment.

For blasting excavation it is often preferable to have the vibration sensitive equipment mapped in relation to the blasting site. At the same time a study of the applicable restrictions has to be made with respect to floor response and the tolerable values proposed for certain receivers, e.g. computer installations. Based on these tolerable values, the blasting work is planned. Depending on the extent, character and cost of the work, the quantity of explosives is optimized. The quantity of explosives per interval and the delay-time between successive explosions determine the magnitude and the characteristics of the vibrations propagating through the building and interacting with the dynamic properties of the structure and the equipment. In vibration sensitive areas it is recommended that test blast investigations be made prior to any site activities in order to get more insight into the following aspects:

- predominant frequency ranges of entire sensitive structures and of floors, walls, etc.
- transmission function between the foundation and floors
- vibration duration and damping of the structure and structural elements
- damping for vibration-sensitive installations.

Today a number of sophisticated methods (FE-method, BE-method) are available to calculate the dynamic behaviour of structures resulting from vibrations transmitted through the ground and the foundation into a structure. It is even possible to take the soil-structure interaction into account.

4.4.8 Remedial measures

To improve an existing annoying vibratory situation due to construction work all measures cited in Section 4.4.6 and Section 4.4.7 may be applied. It is necessary to know exactly the reason for the excessive vibrations and to locate the source of the vibrations. This is one of the key points in limiting vibrations successfully.

References to Chapter 4

[4.1] Bonde G., Rundquist G. et al: "Criteria for acceptable traffic induced vibrations". Institute of Technology, Uppsala University, UPTEC 81 42 R, TRAVI - K. (in Swedish), 1981.

[4.2] Stüssi U.W., Rutishauser G., Haldimann W.: "Schwimmende Strassen- und Gleisplatten" ("Floating Road Pavement and Track Slabs"). Bauingenieur 61, p. 107 - 116, 1986.

[4.3] Studer J.A., Ziegler A.: "Bodendynamik" ("Soil Dynamics"). Springer Verlag Berlin, 1985.

[4.4] Eisenmann, J.: "Oberbau bei Stadtbahnen und U-Bahnen unter besonderer Berücksichtigung der Körperschallemission" ("Rail Track for Tramways and Underground Railways with Special Reference to Structure-borne Sound"). Internationales Verkehrswesen 33, Heft 1, Seite 44 - 48, 1981.

[4.5] Hölzl, G.: "Körperschall- bzw. Erschütterungsausbreitung an Schienenverkehrswegen" ("Structure-Borne Sound and Vibration Emission on Railway Track Systems"). Eisenbahntechnische Rundschau 31, Heft 12, Seite 881 - 887, 1982.

[4.6] Eisenmann, J., Deischl, F., Steinbeisser, L.: "Körperschallemission bei Schienenbahnen, Messung - Dämmung - Prüfung" ("Structure-Borne Sound Near Railway Tracks"). Mitteilungen des Prüfamtes für Bau von Landverkehrswegen der Technischen Universität München, Heft 44, 2. Auflage, 1986.

[4.7] Eisenmann, J., Deischl, F.: "Körperschalldämmung bei unterirdischen Bahnanlagen" ("Attenuation of Structure-Borne Sound From Underground Railway Systems"). Der Eisenbahningenieur, Heft 3, Seite 101 - 110, 1986.

[4.8] Cantieni R.: "Dynamic Load Testing of Highway Bridges". International Association for Bridge and Structural Engineering (IABSE), Zürich. Proceedings P-75/84, August 1984.

[4.9] Persson P-A., Holmberg R., Lande G., Larsson, B.: "Underground Blasting in a City". Subsurface Space, Rock Store 80. Proceedings of the International Symposium, Stockholm, June 23-27. M. Bergman, Editor, Pergamon Press, pp 199-206, 1980.

[4.10] Holmberg, R., Ekman, G., Sandström, H.: "Comments on Present Criteria for Vibration Sensitive Electronic Equipment". Swedish Detonic Research Foundation, Report DS 1983:3, Stockholm, 1983.

[4.11] Studer J., Suesstrunk A.: "1981 Swiss Standard for Vibration Damage to Buildings". Proceeding of the Tenth International Conference on Soil Mechanics and Foundation Engineering, Stockholm, 1981.

[4.12] Forsblad I.: "Vibratory Soil and Rock Fill Compaction". Dynapac Maskin AB, Solna, Stockholm, 1981.

[4.13] Langefors U., Kihlström B.: "The Modern Technique of Rock Blasting". Second Edition, Almqvist & Wiksells Boktryckeri AB, Uppsala, 1967.

[4.14] Forsgren P.G.: "Regulations for blasting close to electric installations". Stockholms Elverk, Serie no. 5/73, Stockholm (In Swedish), 1973.

[4.15] Lande G.: "Vibrations in Soil and Frozen Soil Measurements in Orebro". Institute of Technology, Uppsala University. UPTEC 81 34 R, TRAVI-4 (In Swedish), 1981.

[4.16] Woods R.D.: "Screening of Surface Waves in Soils". J. Soil Mech. and Found. Div. (American Society of Civil Engineering) Proceedings, July 1968.

[4.17] Lande G.: "Vibration Damping Effects due to Road Embankment Thickness and Slot Excavation". Institute of Technology, Uppsala University, UPTEC 81 35 R, TRAVI-5 (In Swedish), 1981.

A Basic vibration theory and its application to beams and plates

A.J. Pretlove, H.G. Natke

A.1 Free vibration

A single degree of freedom (abbreviated to SDOF) linear system is described by the following quantities:

x = displacement
m = mass
c = viscous damping coefficient
k = stiffness

Free vibration of such a system is characterised by the homogeneous differential equation of motion:

$$m\ddot{x} + c\dot{x} + kx = 0 \tag{A.1}$$

The solution to this equation is a sinusoidal vibration the frequency of which is strongly dependent on k and m. The value of c affects the decay of the vibration but has a relatively weak influence on the frequency of vibration; negligible in most cases of structural vibration. The damping in real structures is often not strictly viscous (linear) but in most such cases an equivalent viscous damping coefficient can be used with satisfactory results, see Appendix C. If $c = 0$, the *circular natural frequency* is

$$\omega_1 = \sqrt{\frac{k}{m}} \text{ [rad/s]} \tag{A.2}$$

Note that circular frequencies are related to cyclic frequencies (f in Hz) by the relationship

$$\omega = 2\pi f \tag{A.3}$$

If $c > 0$, then the non-dimensional *damping ratio* is defined as

$$\zeta = \frac{c}{2m\omega_1} = \frac{c}{2 \cdot \sqrt{mk}} \ . \tag{A.4}$$

The quantity $2m\omega_1$ is known as the critical damping coefficient. In structural engineering it is extremely rare that $\zeta > 1$ so this case will not be considered further. For $\zeta < 1$ the full solution to the differential equation is:

$$x(t) = e^{-\zeta\omega_1 t} X \cdot \sin(\omega_d t + \beta) \tag{A.5}$$

The phase angle β depends upon the set time origin and the initial conditions for the motion. X is the amplitude constant for the motion (though this value of displacement may not actually be reached). The natural frequency of the damped system is

$$\omega_d = \omega_1\sqrt{1-\zeta^2} \tag{A.6}$$

In very many real cases damping is small. Then we find:

$$\omega_d \approx \omega_1 \text{ for } \zeta < 0.1 \tag{A.7}$$

For a system subjected to an impulse the initial conditions at time $t = 0$ are

$$x(0) = 0 \quad \text{and} \quad \dot{x}(t)_{t=0} = V = \text{impulse/mass} \tag{A.8}$$

and the solution is then

$$x(t) = e^{-\zeta\omega_1 t} X \cdot \sin\omega_d t \quad \text{where} \quad X = V/\omega_d \tag{A.9}$$

This is a useful result for problems in which the system is acted upon by a force of short duration compared with the natural period $T = 2\pi/\omega_d$. "Short" here means one tenth or less.

If the exponentially decaying vibration can be measured and plotted out then the damping ratio is easily determined from the so-called logarithmic decrement (Λ) or logdec, for short. Figure A.1 shows such a graph. The logdec is defined as the natural logarithm of the ratio (>1) of two successive peaks (taken over a whole cycle) and is derived from the decay plot.

$$\Lambda = \ln(X_1/X_2) \quad \text{and} \quad \Lambda \approx 2\pi\zeta \quad \text{(accurate for } \zeta < 0.1\text{)} \tag{A.10}$$

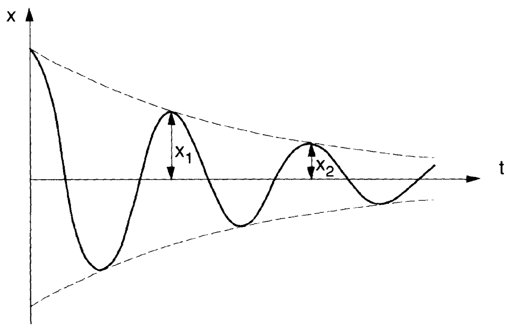


Figure A.1: A typical record of decaying vibration

A.2 Forced vibration

When a time-varying force $F(t)$ is applied to the system the differential equation of motion of the system is now inhomogeneous:

$$m\ddot{x} + c\dot{x} + kx = F(t) \tag{A.11}$$

We have already seen how to solve this problem if $F(t)$ is impulsive. There are four other classes of forcing which may arise:

- Harmonic excitation
- Other periodic excitation
- Transient excitation
- Stochastic (random) excitation

A brief account of the first of these is given in the following section. For other periodic excitations the forcing function can be decomposed into harmonic parts in the form of a Fourier series and treated in the same way (see the following section on harmonic analysis). The other classes of forcing may also frequently occur upon structures. For example, a vehicle crossing a bridge provides a transient excitation; wave or wind action on a structure provides stochastic excitation. The reader is referred to reference [A.1] for details of vibration analysis in these latter cases.

A.3 Harmonic excitation

Harmonic excitation may be caused in a number of ways, for example, as a result of out-of-balance in rotating machinery. It is characterised by the equation

$$F(t) = \hat{F}\cos\Omega t \tag{A.12}$$

where $\hat{F}$ = loading or force amplitude
Ω = circular frequency of excitation

Ignoring the initial transient motion, the continuous steady-state motion which results is

$$x(t) = \frac{\hat{F}}{k}(DMF)\cos(\Omega t - \phi) \tag{A.13}$$

The phase angle ϕ by which the motion sinusoid lags behind the force sinusoid is termed the *phase lag*. The non-dimensional constant (DMF) is the *Dynamic Magnification Factor* which describes how much greater the displacement is dynamically than it would be under a static load of $\hat{F}$. Its value is

$$(DMF) = \frac{1}{[(1-\eta^2)^2 + 4\zeta^2\eta^2]^{1/2}} \quad \text{where} \quad \eta = \frac{\Omega}{\omega_1} \tag{A.14}$$

This is shown graphically in Figure A.2.

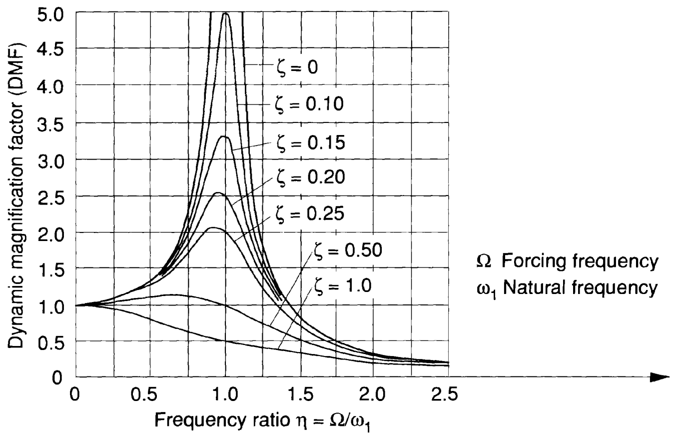


Figure A.2: Forced response of a simple vibrating system with curves shown for different damping ratios

The phase lag ϕ is given by:

$$\tan\phi = \frac{2\zeta\eta}{(1-\eta^2)} \tag{A.15}$$

The maximum value of (DMF) occurs when

$$\Omega = \omega_1 (1 - 2\zeta^2)^{1/2} \tag{A.16}$$

and this is a condition of amplitude *resonance*. If damping is small $(\zeta < 0.1)$ then

$$\Omega \approx \omega_1 \tag{A.17}$$

and the peak value for (DMF) is:

$$(DMF)_{max} \approx 1/(2\zeta) \tag{A.18}$$

Under these conditions the phase lag is:

$$\phi \approx 90° \tag{A.19}$$

$\Omega = \omega_1$ where $\phi = 90°$ is called phase resonance.

A.4 Periodic excitation

A.4.1 Fourier analysis of the forcing function

Any excitation F(t) which is periodic over an interval T can be decomposed into a constant part and a (infinite) series of harmonic force contributions which, when superimposed, result in the total force-time function given. This harmonic decomposition results in a *Fourier series* for the excitation, as follows:

$$F(t) = F_o + \sum_{i=1}^{\infty} [a_i \cos(i\Omega t) + b_i \sin(i\Omega t)] \tag{A.20}$$

Theory shows:

$$F_o = \frac{1}{T}\int_o^T F(t)\,dt \tag{A.21}$$

$$a_i = \frac{2}{T}\int_o^T F(t)\cos(i\Omega t)\,dt \tag{A.22}$$

$$b_i = \frac{2}{T}\int_o^T F(t)\sin(i\Omega t)\,dt \tag{A.23}$$

in which Ω $(=2\pi f)$ is the fixed repetition frequency of the excitation corresponding to the period T. The integer i is the index number of the harmonic components (not to be confused with the square root of minus one). The frequencies of the harmonic components are multiples of the frequency Ω. An alternative way of writing the ith component of the force is:

$$F_i(t) = a_i \cos(i\Omega t) + b_i \sin(i\Omega t) = A_i \sin(i\Omega t - \phi_i) \tag{A.24}$$

with a force magnitude A_i and phase angle ϕ_i. These quantities are given by:

$$A_i = \sqrt{a_i^2 + b_i^2} \quad \text{and} \quad \phi_i = -\arctan(a_i/b_i) \tag{A.25}$$

For many practical purposes the Fourier series is expressed as the Fourier sum:

$$F(t) = F_o + \sum_{i=1}^{n} F_o \alpha_i \sin(i\Omega t - \phi_i) \tag{A.26}$$

where F_o is the mean value of the force waveform. The Fourier coefficients α_i then indicate the relative magnitude of the i-th component of the force waveform.

The i-th component of the force waveform is then often given as:

$$A_i = F_o \alpha_i \tag{A.27}$$

For an example of this, in the case of a forcing function from jumping, see Figures G.2 and [G.3]. In real life the series also has to be truncated to n terms, as seen above. The result of the harmonic analysis of the force is commonly shown as a graph of the Fourier component amplitude A_i vs. i. This kind of plot is known as a *discrete Fourier amplitude spectrum.*

All harmonic components of the periodic excitation can be handled as described in the previous section. All responses superimposed result in the total response of the periodic excitation.

A.4.2 How the Fourier decomposition works

Figure A.3 shows the quality of representation of a periodic triangular excitation curve using a limited number of terms from the infinite series. The Fourier coefficients for the triangular waveform are first found using Equations (A.22) and [A.25] above and these are shown in Figure A.4 for $n = 20$. In fact, for this waveform, $b_i = 0$ and when i is an even number A_i is zero. The waveforms shown in Figure A.3 are then assembled, using Equation (A.20), as *approximations* to the triangular waveform. The results are approximate because the series has been limited to a finite number of terms. It is evident that when more terms are included the result is more accurate. This process is further underlined in Figures A.5 and [A.6].

With reference to Equation (A.26) above, Figure A.5 shows the five components which add together to make the total waveform $F(t)$ shown at the bottom. Note in particular the different magnitudes of the four sinusoids indicated by the α-coefficients and the different phase values ϕ ($\phi = 0$ for $F_1(t)$).

Figure A.6 again shows the corresponding discrete Fourier amplitude spectrum with coefficients at frequencies which are i-multiples of the fundamental frequency interval $f = 1/T$. The lines are, of course, separated by an interval of f.

A.4.3 The Fourier Transform

The concept of Fourier series can be extended by letting T tend to infinity. In the limit, Equation (A.20) is then an integral rather than a summation. The lines on the discrete Fourier amplitude spectrum become infinitesimally close together (f becomes an infinitesimal interval) and this then becomes a *continuous Fourier amplitude spectrum.* The theory for this is called the Fourier Transform theory and is beyond the scope of this appendix. However, it is widely applied in approximate form by modern instrumentation using the Discrete Fourier Transform. The result of one such analysis on a time wave-form of limited duration is shown in Figure G.4 for hand-clapping.

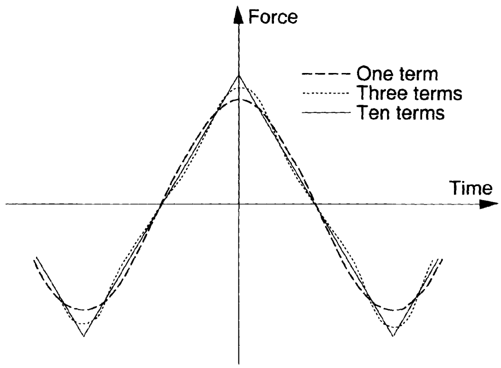


Figure A.3: Fourier series representation of a triangular forcing function using one, three and ten terms in the series

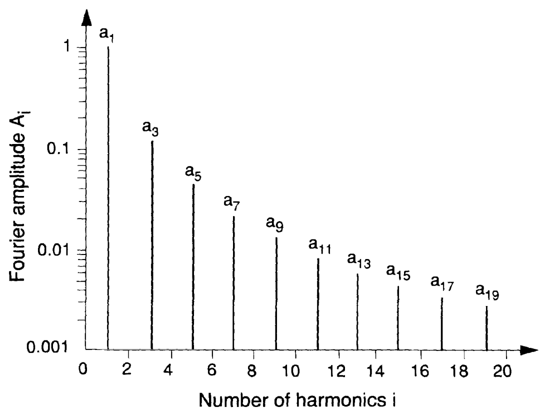


Figure A.4: Discrete fourier amplitude spectrum for the triangular wave of Figure A.3 up the twentieth harmonic

A.5 Tuning of a structure

The aim of frequency tuning of a structure is to avoid the possibility of resonant excitation. If a structure is designed so that the resonance frequency is greater than the forcing frequency ($\eta < 1$) the system is said to be "high-tuned". Conversely, if the resonance frequency is less than the forcing frequency ($\eta > 1$) the structure is said to be "low-tuned" and is often called a "compliant" structure.

Occasionally, as for example in the case of a weaving machine or in the case of the footfall waveform considered in,Appendix G, the forcing waveform is such that one of the higher harmonics (obtained by Fourier analysis, as in the preceding section) is of significant amplitude. When this occurs high- or low-tuning has to be considered *in relation to the frequency of this higher harmonic.*

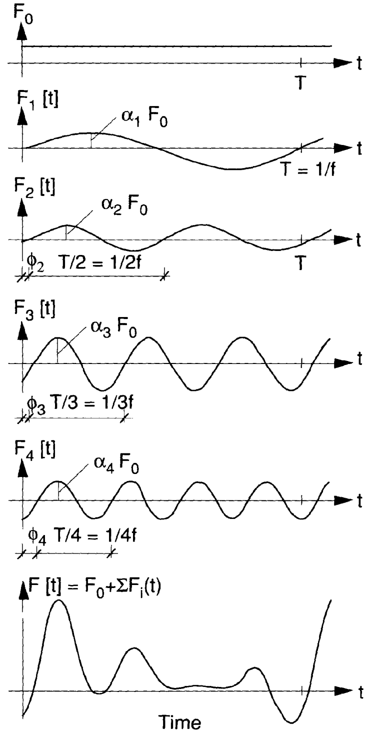


Figure A.5: Fourier decomposition of a periodic function

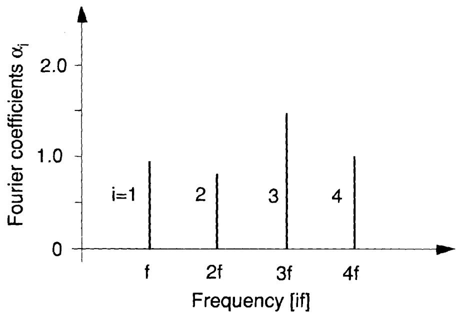


Figure A.6: Discrete Fourier amplitude spectrum (coefficients) of the function of Figure A.5

A.6 Impedance

Many real systems, particularly when they have many degrees of freedom, are treated by means of electrical circuit analogies. At a point in a mechanical system the impedance is defined by

$$Z = \frac{\text{Force}}{\text{Velocity}} \tag{A.28}$$

Using the harmonic analysis given above, the complex impedance may be derived for the mass-spring-damper:

$$Z = c + i(m\omega - k/\omega) \tag{A.29}$$

A.7 Vibration Isolation (Transmissibility)

From the analysis of the preceding section it is possible to calculate the magnitude of the force transmitted to the ground $\hat{F}_g$, where

$$\hat{F}_g = c\dot{x} + kx \tag{A.30}$$

The transmissibility T, is defined as

$$T = \frac{\hat{F}_g}{\hat{F}} = \left[\frac{1 + 4\zeta^2\eta^2}{(1-\eta^2)^2 + 4\zeta^2\eta^2}\right]^{1/2} \tag{A.31}$$

This function gives a measure of how well a vibrating system can isolate the ground from oscillating forces. Note that when $\eta = \sqrt{2}$, $T = 1$. The most effective isolation occurs when $\eta \gg \sqrt{2}$ with $T < 1$, and this generally implies the use of soft support springs. Figure A.7 shows the graphical form of the transmissibility.

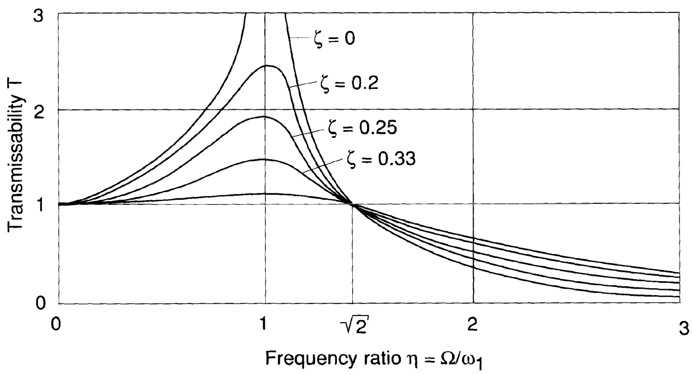


Figure A.7: Transmissibility of an isolator as a function of frequency ratio

The result is usually expressed in dB's of isolation according to the formula:

$$dB = 20 \cdot \log(T) \tag{A.32}$$

and this is shown in Figure A.8. It is apparent that damping does not have a strong influence for harmonic excitation though less damping is better. However, it can be shown that for transient excitation (for example, machine start-up sequences) some damping is essential to prevent large motion as the system passes through resonance.:

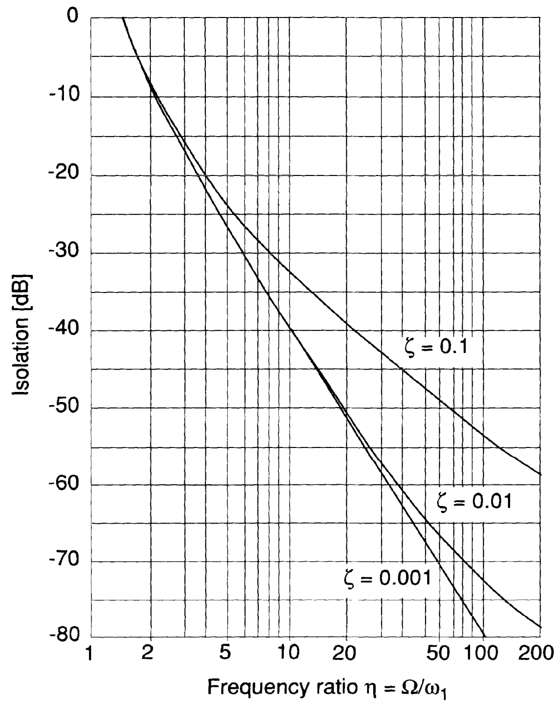


Figure A.8: Attenuation in dB of the force transmitted to ground as a result of isolation

A.8 Continuous systems and their equivalent SDOF systems

In this final section we shall briefly consider the vibrations of beams and plates and how their fundamental vibration may be characterised as those of an equivalent single-degree-of-freedom (SDOF) system. The basis of the analysis of real continuous systems is one of the following:

(1) a continuum differential equation of motion for the system

(2) a discrete finite element approximation, which can be more or less complex.

In both cases the analysis can be reduced, by suitable coordinate transformations, to a problem involving a set of simple oscillators each of which describes one of the characteristic vibrations of the system. This is the basis of the normal mode method and the details of it can be found in good standard textbooks such as [A.1] and [A.2]. In certain circumstances only the fundamental mode of vibration is important and so the continuous system can be approximated by an equivalent SDOF model. The circumstances in which this approximation works well are

(1) when the spatial distribution of forces is reasonably uniform
(2) when the maximum frequency of the (Fourier transformed) force with non-negligible force amplitude is less than or equal to the fundamental natural frequency, and
(3) when the two lowest natural frequencies are not close in value.

In other circumstances consideration must be given to a more rigorous analysis involving the use of more modes of vibration, but this is beyond the scope of this appendix.

Equivalent SDOF systems can easily be found for beams and plates. The procedure for beams is shown in the following. For plates, the procedure is analogous.

To determine the properties of the equivalent SDOF we consider a beam of length L, stiffness EI and distributed mass μ. It is excited by a distributed load $p \cdot \cos(\Omega t)$.

Because we only consider the first eigenmode of the beam, the displacements can be expressed simply as

$$w(x,t) = \xi(t) \cdot f(x) \tag{A.33}$$

where $w(x,t)$ = displacement of the beam
$\xi(t)$ = displacement in a reference point at $x = \xi$
$f(x)$ = shape of the first eigenform of the beam with $f(x = \xi) = 1$

Using the Laplace equation

$$\frac{d}{dt}\left(\frac{\delta E_k}{\delta \dot{\xi}}\right) - \frac{\delta E_k}{\delta \xi} = 0 \tag{A.34}$$

We find:

$$\ddot{\xi} \cdot \mu \int_0^L f^2(x)\,dx \quad + \quad \xi \cdot EI \int_0^L (f''(x))^2 dx \quad = \quad p \cdot \cos(\Omega t) \int_0^L f(x)\,dx \tag{A.35}$$

Equation (A.35) can be written in the form of the governing differential equation of an SDOF system:

$$\tilde{m}\ddot{\xi} + \tilde{k}\xi = \tilde{p}\cos\Omega t \tag{A.36}$$

where $\tilde{m}$ = generalised mass
$\tilde{k}$ = generalised stiffness
$\tilde{p}$ = generalised load

This equivalent SDOF has the same *natural frequency* as the original system, i.e. the beam, and the same *reference displacement amplitude*. The generalised properties of the equivalent SDOF can be found as follows. The generalised mass is given by:

$$\tilde{m} = \mu \int_0^L f^2(x)\, dx = \phi_M \cdot \mu L \qquad \text{and} \qquad \phi_M = \frac{1}{L} \int_0^L f^2(x)\, dx \tag{A.37}$$

where μ = distributed mass
L = length of the beam
f = shape of the first eigenform
ϕ_M = mass factor

Obviously, the generalised mass is independent of the load. For a lumped mass the mass factor becomes $f^2(x = \xi_M)$, i.e. the square of the displacement of the first eigenform at the location of the lumped mass.

The generalised stiffness of the equivalent SDOF is given as

$$\tilde{k} = EI \int_0^L (f''(x))^2 dx = \vartheta \cdot \frac{EI}{L^3} \qquad \text{and} \qquad \vartheta = \frac{1}{L^3} \int_0^L (f''(x))^2 dx \tag{A.38}$$

where EI = flexural stiffness of the beam
For certain materials the difference between the dynamic and static E-modules must be considered (see Appendix F for details).
ϑ = stiffness factor

For practical cases the generalised stiffness can be approximated as follows:

$$\tilde{k} \approx \phi_L \cdot k \tag{A.39}$$

where ϕ_L = load factor (see below)
k = beam stiffness, i.e. stiffness at the reference point for the given load.

The generalised load is defined by the following equation

$$\tilde{p} = p \int_0^L f(x)\, dx = \phi_L \cdot pL \qquad \text{and} \qquad \phi_L = \frac{1}{L} \int_0^L f(x)\, dx \tag{A.40}$$

where ϕ_L = load factor
p = distributed load

The load factor for a single load acting at the reference point is always 1.0.

For simple cases of beams and plates the values of the load, stiffness and mass factor are given in Figures A.9 and A.10. For systems or loads that are not shown there, the above described factors can easily be approximated using the static deflection curve instead of the first eigenform of the system.

The calculation of ϑ (or ϕ_L), ϕ_M, $\tilde{k}$ and $\tilde{m}$, leads to a rough estimation of ω by

$$\omega^2 = \tilde{k}/\tilde{m} \tag{A.41}$$

For plates (see Figure A.10) the flexural stiffness EI must be calculated for unit width considering plain stress conditions:

$$E_{plate} = E/(1-\nu^2) \tag{A.42}$$

$$I = t^3/12 \tag{A.43}$$

where ν = Poisson's ratio of the material
E = E-modulus of the material
t = thickness of the plate

If damping is to be considered, the damping ratio of the original system may be used unaltered.

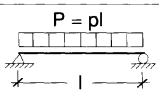
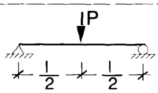
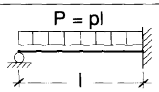
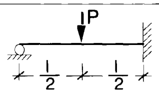
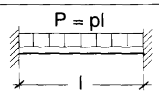
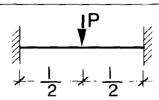
Loading and support conditions Reference point at l/2	Load factor ϕ_L	Mass factor ϕ_M		Effective beam stiffness k	Stiffness factor ϑ
		Lumped mass	Distributed mass		
	0.637	—	0.5	$\frac{384\ EI}{5 \cdot l^3}$	48.7
	1.0	1.0	0.5	$\frac{48\ EI}{l^3}$	48.7
	0.595	—	0.479	$\frac{185\ EI}{l^3}$	113.9
	1.0	1.0	0.479	$\frac{107\ EI}{l^3}$	113.9
	0.523	—	0.396	$\frac{384\ EI}{l^3}$	198.5
	1.0	1.0	0.396	$\frac{192\ EI}{l^3}$	198.5

Figure A.9: Equivalent substitute SDOF parameters for single span beams with various support and load conditions (see explanation in the text)

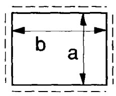
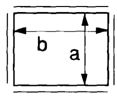
Support conditions	$\frac{a}{b}$	Load factor ϕ_L	Mass factor ϕ_M	Effective plate stiffness k
 $\frac{b}{2}$ a b	1.0	0.45	0.31	$\frac{271EI_0}{a^2}$
	0.9	0.47	0.33	$\frac{248EI_0}{a^2}$
	0.8	0.49	0.35	$\frac{228EI_0}{a^2}$
	0.7	0.51	0.37	$\frac{216EI_0}{a^2}$
	0.6	0.53	0.39	$\frac{212EI_0}{a^2}$
	0.5	0.55	0.41	$\frac{216EI_0}{a^2}$
 $\frac{b}{2}$ a b	1.0	0.33	0.21	$\frac{870EI_0}{a^2}$
	0.9	0.34	0.23	$\frac{798EI_0}{a^2}$
	0.8	0.36	0.25	$\frac{757EI_0}{a^2}$
	0.7	0.38	0.27	$\frac{744EI_0}{a^2}$
	0.6	0.41	0.29	$\frac{778EI_0}{a^2}$
	0.5	0.43	0.31	$\frac{866EI_0}{a^2}$

Figure A.10: Approximate equivalent substitute SDOF parameters for rectangular plates with various support conditions (uniformly distributed mass and load only); the factors have been derived for simultaneous excitation of more than one mode of vibration as a result of impact loading and are not true SDOF equivalents

B Decibel Scales

H.G. Natke, G. Klein

B.1 Sound pressure level

In acoustics, large numerical ranges of sound pressure are possible between minimum and maximum values. This makes their mathematical manipulation very cumbersome. Consequently, for acoustical purposes the unit of sound pressure was standardized to be the decibel [dB], which is defined as 20 times the logarithm of the ratio of the pressure to a suitable corresponding reference pressure [ISO 131], [ISO/R 357], [DIN 45630/1].

If p is the sound pressure and p_o the reference pressure, then the pressure level is given by

$$L_p = 20 \cdot \log \frac{p}{p_o} = 10 \cdot \log\left(\frac{p}{p_o}\right)^2 = 10 \cdot \log\left(\frac{I}{I_o}\right) \text{ [dB]} \tag{B.1}$$

where I and I_o are intensities corresponding to p and p_o. The reference sound pressure is defined by $p_o = 20\ \mu\text{Pa} = 2 \cdot 10^{-5}\ \text{N/m}^2$ and is the r.m.s-value of the average human hearing threshold at 1000 Hz.

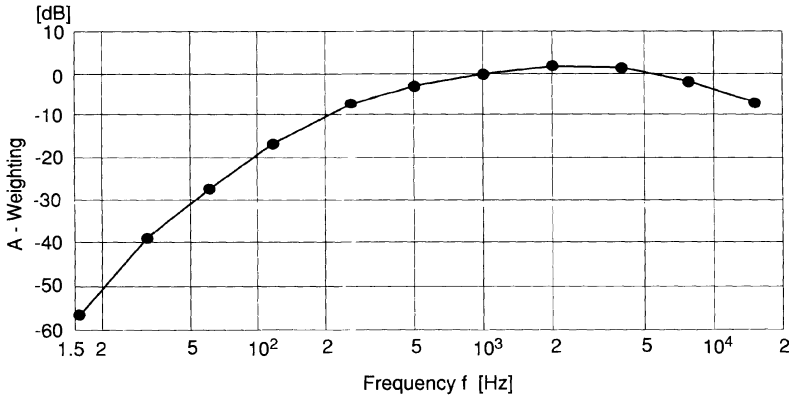


Figure B.1: A-weighting

B.2 Weighting of the sound pressure level

In common use is also a weighting of the sound pressure level due to the perception characteristics of the ear, resulting in dB(A) values instead of dB values [DIN IEC 651]. The weighting is done by subtracting from the L_p [dB] value a value depending on the frequency and shown in Figure B.1. For example for 50 Hz a value of 30.2 dB must be subtracted from L_p [dB] in order to obtain the value in dB(A).

In the same way definitions can be used for other quantities such as intensities, and the r.m.s-values of accelerations, velocities and displacements. For example the displacement level is given by

$$L_x = 20 \cdot \log \frac{x_{rms}}{x_o} \text{ [dB]} \tag{B.2}$$

with a reference value conveniently chosen as

$$x_o = 0.8 \cdot 10^{-11} \text{ m} \tag{B.3}$$

Concerning the velocity and acceleration levels the reference values are

$$v_o = 5 \cdot 10^{-8} \text{ m/s and } a_o = 5 \cdot 10^{-4} \text{ m/s}^2 \tag{B.4}$$

The reference value v_o is given e.g. in [DIN 45630/1], the other reference values follow by conversion with 1000 Hz.

C Damping

O. Mahrenholtz, H. Bachmann

C.1 Introduction

Damping in a vibrating structure is associated with a dissipation of mechanical energy, usually by conversion into thermal energy. The energy dissipation equals the work done by the damping force. In the case of a free vibration the presence of damping results in a continuous decay of the amplitude. In order to maintain a constant amplitude in the case of a forced vibration, the energy dissipation must be continuously replenished by an external mechanical energy source.

In Figure C.1 the more important types of damping are shown.

C.2 Damping Quantities (Definitions, Interpretations)

In the case of oscillatory forcing functions, the stress (or the internal forces) are found to lead the strain (or the deformation). Thus a hysteresis loop is formed and completed during each cycle (Figure C.2)

The area

$$W_D = \oint \sigma \, d\varepsilon \tag{C.1}$$

inside the hysteresis loop represents the mechanical energy dissipated, or the work done by the damping force, in a unit volume of the material.

The *damping factor* ψ of the *material* is proportional to the ratio of energy dissipation W_D to maximum strain potential energy E_{pot} (see Figure C.2b):

$$\psi = \frac{1}{2\pi} \cdot \frac{W_D}{E_{pot}} \tag{C.2}$$

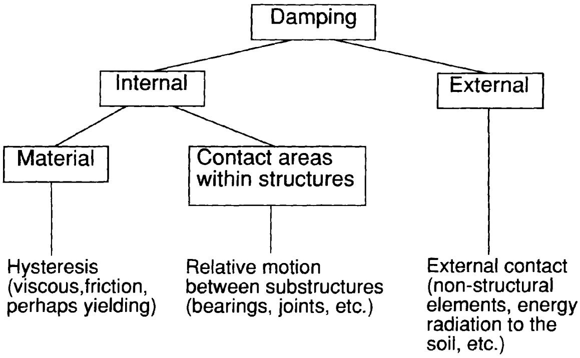


Figure C.1: Different types of damping

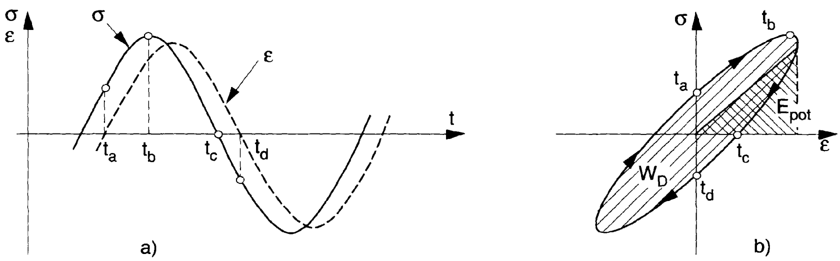


Figure C.2: Phase displacement and hysteresis loop

The damping factor ψ_S of the *structure* is obtained by integrating over the volume of the structure and averaging:

$$\psi_S = \frac{\int \psi \ dV}{V} = \frac{1}{2\pi} \cdot \frac{W_{DS}}{E_{potS}} \tag{C.3}$$

For a homogeneous structure is $\psi_S = \psi$.

If the structure is modelled as a simple linear oscillator with a harmonic forcing function (Figure C.3), the equation of motion is

$$m\ddot{x} + c\dot{x} + kx = F(t) = \hat{F}\cos\Omega t \tag{C.4}$$

where $\hat{F}$ is the amplitude and Ω the circular frequency of the forcing function. A steady-state solution of Equation (C.4) is

$$x = \hat{x}\cos(\Omega t - \phi) \tag{C.5}$$

The *damping coefficient* c represents viscous or linear damping.

For this case of linear viscous damping the energy dissipation in the structure, per cycle (of duration (period) $T = 2\pi/\Omega$), is then obtained as

$$W_{DS} = \int_o^T (c\dot{x})\dot{x}dt = \pi c \Omega \hat{x}^2 \tag{C.6}$$

The maximum strain energy in the structure is

$$E_{potS} = \frac{1}{2}k\hat{x}^2 \tag{C.7}$$

The damping factor Equation (C.3) becomes now

$$\psi_S = \frac{1}{2\pi} \cdot \frac{W_{DS}}{E_{potS}} = \frac{c\Omega}{k} \tag{C.8}$$

Frequently the damping is quite small and only affects the vibration behaviour significantly near resonance. For $\Omega = \omega_1 = \sqrt{k/m}$ (see Appendix A), the damping factor at resonance becomes

$$\psi_{S1} = \frac{c\omega_1}{k} = \frac{c}{\sqrt{km}} \tag{C.9}$$

The *damping ratio* ζ is also often used:

$$\zeta = \frac{c}{2\sqrt{km}} = \frac{c}{2m\omega_1} \tag{C.10}$$

and it follows that

$$\psi_{S1} = 2\zeta \tag{C.11}$$

The quantity $2m\omega_1$ is known as the *critical damping coefficient* c_{crit}. Hence

$$\zeta = \frac{c}{c_{crit}} \tag{C.12}$$

For $c = c_{crit}$ (or $\zeta = 1$) the vibration changes its time dependent character from an oscillating function to a function steadily converging to a zero displacement.

From Equations (C.8) and (C.11) it follows that:

$$\zeta = \frac{1}{4\pi} \cdot \frac{W_{DS}}{E_{potS}} \tag{C.13}$$

A free vibration of the oscillator of Figure C.3 has a frequency of $\omega_d = \omega_1\sqrt{1-\zeta^2}$ (see Appendix A). The natural logarithm of the ratio of two successive amplitudes one period apart is called the *logarithmic decrement.*

$$\Lambda = \ln\frac{x(t)}{x\left(t+\frac{2\pi}{\omega_d}\right)} = \frac{2\pi\zeta}{\sqrt{1-\zeta^2}} \tag{C.14}$$

For low damping is $\zeta^2 \ll 1$ and the damping ratio can then be found from

$$\zeta \approx \frac{\Lambda}{2\pi} \tag{C.15}$$

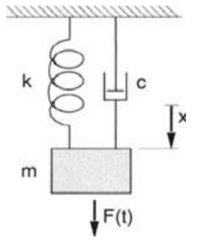


Figure C.3: Model of a single degree of freedom oscillator (SDOF model)

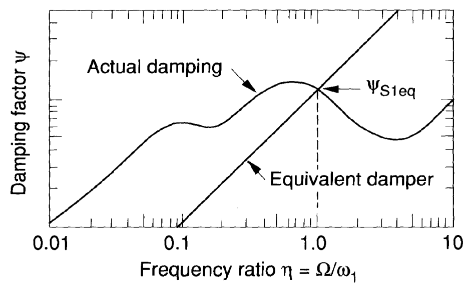


Figure C.4: Actual damping and equivalent damper

For a viscous damper the damping coefficient c of Equation (C.4) is a constant, and the hysteresis loop of Figure C.2 is an ellipse. Real structures do not often exhibit purely viscous damping. They are also subject to other kinds of energy dissipation such as friction (Coulomb) damping and energy radiation (Figure C.1). Since the main effect of damping is found in the region of resonance, the actual behaviour can usually be represented satisfactorily by an *equivalent damper* with an *equivalent viscous damping ratio* ζ_{eq}, incorporating all kinds of energy dissipation:

$$\zeta_{eq} = \frac{1}{4\pi}\cdot\frac{\text{Energie dissipation per cycle}}{\text{Maximum potential energy}} \tag{C.16}$$

ζ_{eq} corresponds to an equivalent viscous damping factor ($\psi_{S1,\,eq}$) which equals the actual damping factor for $\Omega/\omega_1 = 1.0$ (see the dashed line in Figure C.4).

This approach can also be used for systems with more degrees of freedom. An equivalent damper is then associated to each eigenmode. Any coupling between modes is neglected, and an a-priori knowledge of the actual damping is assumed for those frequency bands in which damping is significant.

If the limits of such a band are not known, then the damping model must be effective for a large frequency region. In such a case a frequency-dependent *equivalent viscous damping coefficient* is introduced:

$$c(\Omega) = \frac{k\psi_S(\Omega)}{\Omega} \tag{C.17}$$

If the frequency-dependent equivalent damping coefficient is introduced into Equation (C.4), one obtains

$$m\ddot{x}(t) + c(\Omega)\dot{x}(t) + kx(t) = F(t) \tag{C.18}$$

In engineering applications, it is often found that the energy loss W_{DS} is dependent upon $\hat{x}^2$, but for all intents and purposes independent of the frequency, such that

$$\psi_S = \text{constant} \tag{C.19}$$

instead of the result of Equation (C.8). The result is a *frequency-dependent equivalent damping coefficient*

$$c(\Omega) = k\frac{\psi_S}{\Omega} \tag{C.20}$$

For dry contact areas, *Coulomb friction* represents the most significant dissipation mechanism. A simple model of such a mechanism is shown in Figure C.5.

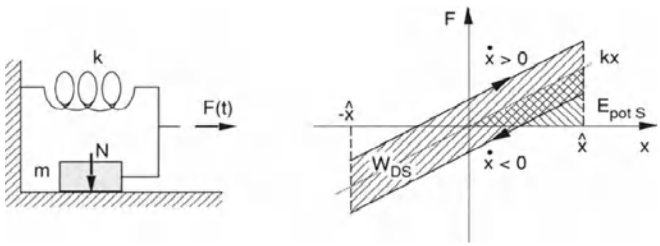


Figure C.5: Simple Coulomb model and its hysteresis loop

The differential equation describing such a system is

$$m\ddot{x} + \mu N \operatorname{sgn}(\dot{x}) + kx = F(t) \tag{C.21}$$

which can be linearized, provided $\mu N \ll k\hat{x}$. The energy dissipation per cycle is

$$W_{DS} = 4\mu N\hat{x} \tag{C.22}$$

If the energy loss is compared with that of Equation (C.6) an equivalent damping coefficient dependent upon frequency and amplitude results:

$$c(\Omega, \hat{x}) = \frac{4\mu N}{\pi\Omega\hat{x}} \tag{C.23}$$

For a harmonic forcing function $F(t) = \hat{F}\cos\Omega t$, the *amplitude ratio* becomes

$$\frac{\hat{x}}{\hat{F}/K} = \frac{1}{1-\left(\frac{\Omega}{\omega_1}\right)^2} \cdot \sqrt{1-\left(\frac{4\mu N}{\hat{F}}\right)} \tag{C.24}$$

Since the energy loss of the model is a linear function of the amplitude (Equation (C.22)), the amplitude at resonance grows without limit if

$$\hat{F}\pi > 4\mu N \tag{C.25}$$

C.3 Measurement of damping properties of structures

C.3.1 Decay curve method

By measurement of a decay curve (e.g. of a floor, a bridge, etc. responding as a SDOF model, (see Figure C.6) the equivalent viscous damping properties using Equations (C.14) and (C.15)

$$\Lambda = \frac{1}{m} \cdot \ln\left(\frac{x_n}{x_{n+m}}\right) \quad \text{and} \quad \zeta \approx \frac{\Lambda}{2\pi} \tag{C.26}$$

Λ or ζ are often amplitude-dependent. Thus from different parts of the decay curve different damping quantities result. (Also the natural frequency may be amplitude-dependent due to change of stiffness and coupling with other eigenmodes).

For purely viscous damping, the dotted envelope line in Figure C.6 is an exponential decay, and for pure friction damping (Coulomb damping) it is a straight line decay. For real structures the envelope line generally lies in between these two cases.

C.3.2 Bandwidth method

For an ideal damper and small damping ($\zeta \ll 1$ or $\zeta < 0.1$, respectively), the damping ratio can be obtained from the *half power bandwidth* ($\Omega_2 - \Omega_1$) (Figure C.7) from the resonance curve due to a harmonic forcing function [C.1]:

$$\zeta = \frac{\Omega_2 - \Omega_1}{\Omega_2 + \Omega_1} \tag{C.27}$$

Instead of the displacement amplitude curve the velocity or the acceleration amplitude curves may be used but only, of course, in cases where damping is low.

However, for measurement of damping in most cases the bandwidth method cannot be recommended. Non-linear behaviour of the structure may lead to a turning of the peak of the res-

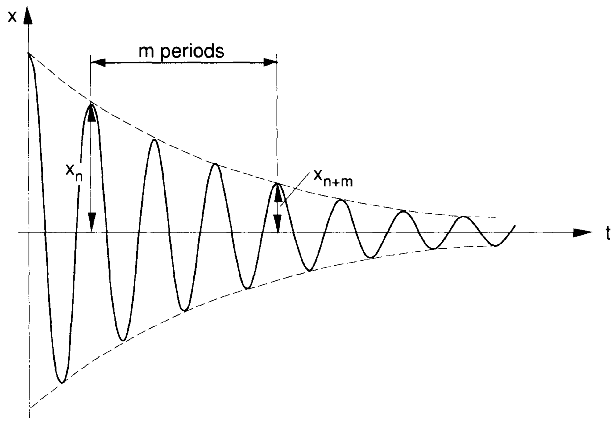


Figure C.6: Measured decay curve

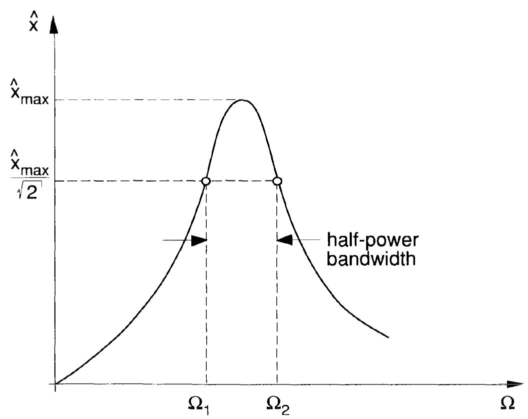


Figure C.7: Resonance curve with half power bandwidth onance curve to the left (increasing stiffness with displacement) or to the right (decreasing stiffness with displacement), with the result that for a certain value of Ω/ω_1 two (or three) different amplitudes may occur. This makes it very difficult to evaluate the resonance curve. Also measuring errors will affect to a great extent the determination of the damping quantity. In addition, the effort expended in measuring a resonance curve is much higher than that for the measurement of a decay curve.

C.3.3 Conclusions

In most practical cases damping is evaluated by the decay curve method rather than by the bandwidth method. In practice a more or less free decay can be produced by one of several methods, for example: by people standing still after jumping at the resonance frequency, or by abruptly stopping a rotating eccentric type of machine, or by imparting an impact such as a dropped sandbag (provided that one natural frequency dominates the vibrational behaviour). For horizontally vibrating structures (and in some cases for vertically vibrating structures also) an initial displacement may be produced by the sudden release of a steel cable or using the pulse of a rocket.

C.4 Damping mechanisms in reinforced concrete

Material damping in reinforced concrete elements (and similarly in partially prestressed concrete elements) in the quasi-elastic range (no yielding of reinforcement) shows some special features mainly due to cracking. The damping depends strongly on the stress intensity. Figure C.8 shows the equivalent damping ratio ζ of a bending element or a beam mainly subjected to bending moments. The stress intensity may be characterised by the stress amplitude in the bending reinforcement or by the displacement amplitude of a beam, both defined at the point of maximum stress or displacement respectively.

For low stress intensity, corresponding to the uncracked state, relatively low damping ratio ($\zeta < 1$ %) exists. With formation of cracks the damping ratio increases. In the final cracked state but still with relatively low stress intensity the damping ratio is relatively high, perhaps twice or three times the value of the initial uncracked state. With further increase in the stress intensity the damping ratio decreases rapidly and may reach a value smaller than that of the initial uncracked state.

This damping behaviour can be explained as follows [C.2]:

- In the *uncracked state* nearly pure viscous damping occurs in the concrete.
- In the *cracked state* two kinds of damping occur:
  - nearly pure viscous damping in the concrete in the uncracked compression zone
  - nearly pure friction damping due to friction between the concrete and the reinforcing steel in the cracked tension zone.

Figure C.9 shows a cracked bending element and a corresponding relevant model. The damping in the compression zone is modelled by a viscous damper, and the damping in the tension zone by a friction damper. The spring represents the bending stiffness of the bending element, and m represents the relevant mass.

Following the definition of the equivalent damping ratio according to Equation (C.16) one can see: E_{potS} is proportional to the square of the stress intensity, in Equation (C.7) represented by $\hat{x}$. The energy dissipation per cycle due to viscous damping, W_{DS}, is also proportional to the square of the stress intensity, see Equation (C.6). Hence for constant depth of the bending compression zone a viscous damping component of ζ results which is independent of the stress intensity. On the other hand, the energy dissipation per cycle due to friction damping, W_{DS}, is linearly proportional to the stress intensity, see Equation (C.22). Hence a friction damping component of ζ results which decreases hyperbolically with increasing stress intensity.

Note that with increasing stress intensity the energy dissipation per cycle, in spite of the decreasing equivalent damping ratio ζ, is increasing due to the fact that the maximum strain energy E_{potS} is increasing.

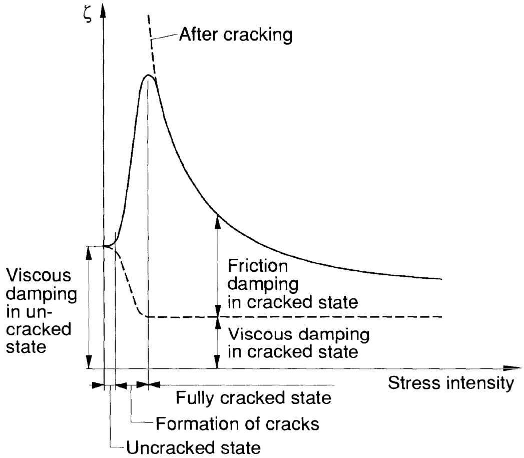


Figure C.8: Equivalent damping ratio of a reinforced concrete element in different states [C.2]

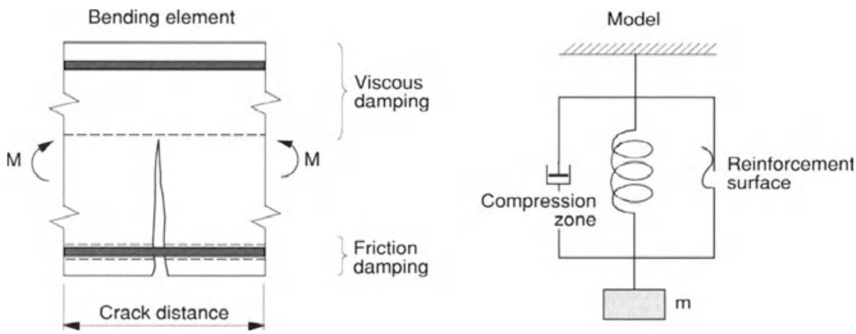


Figure C.9: Cracked bending element and relevant model for material damping of reinforced concrete [C.2]

C.5 Overall damping of a structure

Depending on the location of energy dissipation the total overall damping of a structure (e.g. a bridge, a gymnasium, a tower, etc.) is a sum of the following contributions:

- damping of the bare structure
- damping by non-structural elements
- "damping" by energy radiation to the soil

While the first contribution always exists, the second and/or the third contribution may be great or small or not present depending on the type and purpose of the structure.

C.5.1 Damping of the bare structure

Energy dissipation in the bare structure occurs by

- material damping
- damping at bearings and joints

In most cases the first contribution is predominant. Table C.1 shows material damping quantities (equivalent viscous damping ratio) for different materials of construction.

Material	ζ
Reinforced concrete	
• small stress intensity (~ uncracked)	0.007 - 0.010
• medium stress intensity (fully cracked)	0.010 - 0.040
• high stress intensity (fully cracked), but no yielding of reinforcement	0.005 - 0.008
Prestressed concrete (uncracked)	0.004 - 0.007
Partially prestressed concrete (slightly cracked)	0.008 - 0.012
Composite	0.002 - 0.003
Steel	0.001 - 0.002

Table C.1: Material damping of different materials

C.5.2 Damping by non-structural elements

Depending on number, type and relative dimensions non-structural elements may contribute to a smaller or greater extent to the overall equivalent damping ratio of a structure. This contribution may be greater than the equivalent damping ratio of the bare structure alone. This explains why different structures or structural types made of the same materials may have very different values for ζ. By way of illustration the influence of non-structural elements for some structural types is as given in Table C.2.

Type of structure	Non-structural elements	Influence of non-structural elements
Footbridges (vertical excitation)	• pavement • railings	in general relatively small
Gymnasia (vertical excitation)	• flooring • façades • equipment	in general moderate
Building floors (vertical excitation)	• partition walls • flooring • furniture • ceilings • mechanical services	in general relatively great
Towers (horizontal wind excitation)	• equipment • mechanical services	in general small

Table C.2: Influence of non-structural elements on overall damping

C.5.3 Damping by energy radiation to the soil

Energy radiation to the soil by travelling waves may also contribute significantly to the overall equivalent damping ratio. By way of illustration the influence of energy radiation for some structural types is as given in Table C.3

Type of structure	Bearings/Supports/Soil	Influence of energy radiation
Footbridges (vertical excitation)	• steel bearings supporting a beam structure • vibrating support structures (columns or walls) have direct contact with the earth	small medium to great
Towers (horizontal wind excitation)	• medium stiff or soft soil • stiff soil or rock	medium to high low

Table C.3: Influence of energy radiation on overall damping

C.5.4 Overall damping

The great differences which are possible for the damping of the bare structure (predominantly material damping), for the damping by non-structural elements and for the damping by energy radiation to the soil, explain why *very different damping quantities may result*

- for different materials of construction
- for different structure types, although they are of the same material of construction
- for different structures of the same material of construction and the same structure type.

Consequently the damping quantitites given in the different sub-chapters of this book cover a relative wide range.

In the case of a real vibration problem, where ζ cannot be measured, it is the task of the structural engineer to take into account the influences described above in making a cautious assessment or estimate of the overall equivalent viscous damping ratio.

For reinforced concrete structures subjected to wind, damping quantities are given in Appendix H. Note that the subdivision of the overall damping quantity is different from that described above at the beginning of Appendix C.5.

D Tuned vibration absorbers

H. Bachmann, H.G. Natke

D.1 Definition

A vibration absorber is a vibratory subsystem attached to a larger primary vibration system. The vibration absorber consists in general of a mass, a spring and a damper (or some parallel springs and dampers). Accurate tuning of the frequency of the absorber results in induced inertia forces of the absorber mass which counteract the forces applied to the primary system, and less work is done on this system. Hence, the normal practical function of the absorber is to reduce resonant oscillations of the primary system (even though in theory it could be used as an inertia balancer at any frequency, provided it is tuned with respect to the forcing frequency). While the vibration amplitudes of the primary system can thus be suppressed to a large extent, large displacement amplitudes must be accepted in the absorber system [D.1], [D.6], [D.7], [D.8].

D.2 Modelling and differential equations of motion

In most cases vibration at one natural frequency of the primary system is troublesome and requires attenuation. This system can then be modelled as an equivalent single degree of freedom system (SDOF-system, see also Appendix A). Together with the absorber system a two degree of freedom system results (see Figure D.1). The relevant equations of motion are:

$$m_s \cdot \ddot{x}_s + c_s \cdot \dot{x}_s - c_t \cdot (\dot{x}_t - \dot{x}_s) + k_s \cdot x_s - k_t \cdot (x_t - x_s) = \hat{F}\cos\Omega t \qquad \text{(D.1)}$$

$$m_t \cdot \ddot{x}_t + c_t \cdot (\dot{x}_t - \dot{x}_s) + k_t \cdot (x_t - x_s) = 0 \qquad \text{(D.2)}$$

where

s	=	parameter subscript for the primary system
t	=	parameter subscript for the tuned vibration absorber
x	=	total displacement
m	=	mass
c	=	damping coefficient (see Appendix C)
k	=	spring constant
$\hat{F}\cos(\Omega t)$	=	harmonic excitation (see Appendix A) of the primary system

D.3 Optimum tuning and optimum damping of the absorber

The solution of the equations above leads to the *optimum frequency* of the absorber :

$$f_t = \frac{f_s}{1 + m_t/m_s} \tag{D.3}$$

$$\text{where } f_t = \frac{1}{2\pi}\sqrt{k_t/m_t} \text{ and } f_s = \frac{1}{2\pi}\sqrt{k_s/m_s} \tag{D.4}$$

f_t is seen to be somewhat smaller than the frequency f_s of the primary system (~ 95% to 99%) and to depend on the mass ratio m_t/m_s.

Calculations of frequency response curves show that the maximum displacement amplitude of primary systems can be substantially reduced by an absorber with the optimum frequency, but the reduction is sensitive to minor changes in the absorber frequency [D.2]. The absorber hardly affects the response of the primary system away from the optimum absorber frequency.

In general the use of a damping element in the absorber is recommended, i.e. a dashpot. The *optimum damping ratio* (equivalent viscous damping ratio, see Appendix C) of the vibration absorber corresponds to

$$\zeta_{opt} = \sqrt{\frac{3(m_t/m_s)}{8 \cdot (1 + m_t/m_s)^3}} \tag{D.5}$$

This formula is, strictly speaking, only valid for an undamped primary system [D.1], but it can also be used for a damped system with good approximation. ζ_{opt} can also be determined by stepwise variation of the numerical value of ζ in calculating the frequency response curve. Figure D.2 gives an example of the effect of damping ratio of the vibration absorber on the displacement of the primary system. In special cases, damping need not be provided at all, particularly if one is sure that the excitation frequency remains constant.

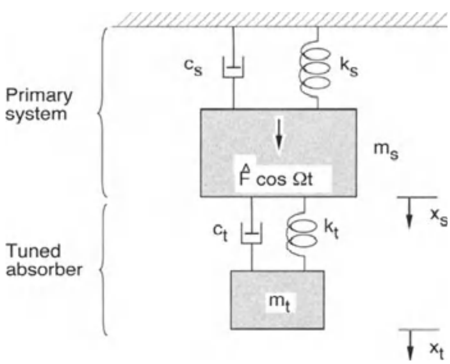


Figure D.1: Dynamic model of primary system with an absorber system

The effectiveness of the absorber is less affected by a difference between the actual damping and the optimum damping of the absorber than by a difference between the optimum frequency and the actual frequency of it. Hence poor tuning of the absorber after installation or detuning over time caused by changes in the stiffness of the primary system or of the absorber springs or by changes in masses may strongly reduce the effectiveness of the absorber.

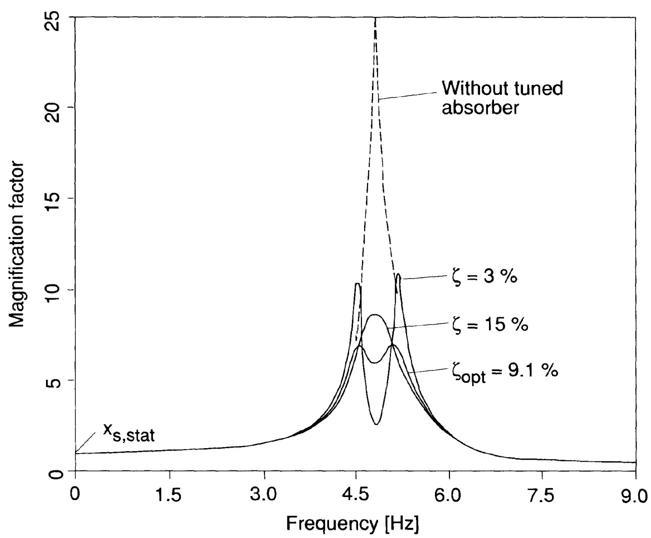


Figure D.2: Example of the effect of the damping ratio ζ of the vibration absorber on the frequency response of a primary system

D.4 Practical hints

To examine a possible application of a tuned vibration absorber and to design such a subsystem, the following factors must be considered:

- The application of vibration absorbers can be considered whenever critical dynamic forcing cannot be avoided, especially in cases where existing or planned structures are difficult to stiffen economically.
- Experience shows that the application of vibration absorbers can be a good solution in the case of one-dimensional beamlike structures such as footbridges, chimneys and pylons of cable stayed bridges, whereas in cases of two-dimensional plate-like structures, such as gymnasium floors, dance floors and concert hall floors, an array of absorbers is needed which is not always an economical solution.
- A vibration absorber of the type described above is only effective over a narrow frequency band and when tuned to a particular structural frequency. It does not work satisfactorily for primary system structures with several closely spaced natural frequencies which are all more or less excited by the dynamic action in question.

- A vibration absorber is more effective
  - · the larger the mass of the absorber compared with the (modal) mass of the primary system (due to the limitation of the displacement amplitude of the absorber mass), and
  - · the smaller the damping of the primary system is.

  Absorbers are particularly well suited for reducing resonant vibrations of structures of not too large a mass and small inherent damping; they will prove inadequate for structures with a large mass or strong vibration despite substantial inherent damping.
- Depending on the mass and the damping of the primary system and on the required reduction of the amplitudes of this system, for a first design approach the mass ratio m_t/m_s may be chosen in the range of 1/15 to 1/50. The influence of this parameter should be studied. Beyond a certain absorber mass any further increase in mass results only in negligible further reduction of the vibration amplitudes of the primary system.
- The dynamic displacement amplitudes of the absorber mass has to be checked by calculation.
- The fatigue behaviour of the absorber springs must be considered.
- Design and installation of an absorber have to take into account
  - · a possible need to replace springs and dampers at some later date
  - · the prevention of the absorber mass from falling down in the case of a spring failure.
- In many cases compression springs are more suitable than tension springs.
- Theoretical calculation for the primary system and the absorber is not sufficient on its own for determining the final absorber parameters. Rather, the accurate frequency of the primary structure has to be measured in situ, and the absorber design should allow for refined tuning after installation. This is more easily done by altering the absorber mass than by varying the spring properties.

Examples of the application of tuned vibration absorbers are given in [D.3], [D.4], [D.5].

E Wave Propagation

G. Klein, J.H. Rainer

E.1 Introduction

The propagation of waves plays an important part in vibration control, particularly when housing and other sensitive installations are to be planned for minimal vibration effects. Such an evaluation will inevitably entail measurements since the ground is a very irregular transmission medium and consequently difficult to treat analytically. With the following theoretical treatment of wave propagation in idealized media, however, an approximate evaluation can be carried out. Further details on this topic can be found, for example, in [E.1] and [E.2].

E.2 Wave types and propagation velocities

An elastic, homogeneous and isotropic medium with density ρ is characterized by the elastic constants of modulus of elasticity E and Poisson's ratio ν, from which the shear modulus $G = E/2(1+\nu)$ can be derived.

Alternatively, Lamé's constants $\lambda = \nu E/(1+\nu)(1-2\nu)$ and $\mu = G$ can be used.

In an infinite continuum, two types of waves exist whose propagation velocities are

$$v_p = \sqrt{\frac{(\lambda + 2\mu)}{\rho}} = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \tag{E.1}$$

$$v_S = \sqrt{\mu/\rho} = \sqrt{\frac{E}{2\rho(1+\nu)}} = \sqrt{G/\rho} \tag{E.2}$$

The first is called a *P-wave* (or compression wave, primary wave, longitudinal wave), for which the particles of the ground vibrate parallel to the direction of wave propagation. The second wave is called *S-wave* (or shear wave, torsional wave, secondary wave, transverse wave), for which the particles vibrate perpendicular to the direction of wave propagation. From the ratio $v_p/v_S = \sqrt{2(1-\nu)/(1-2\nu)} > 1$ it can be seen that v_p is always greater than v_S, and thus at a measurement point, the longitudinal wave will always arrive before the transverse wave.

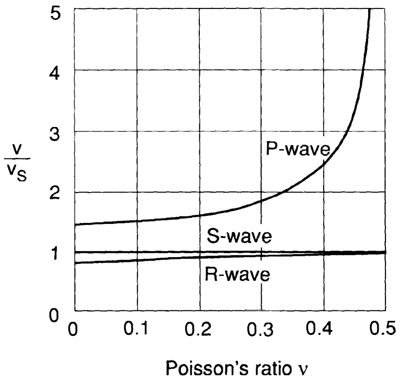


Figure E.1 : Relation between Poisson's ratio and velocities of propagation

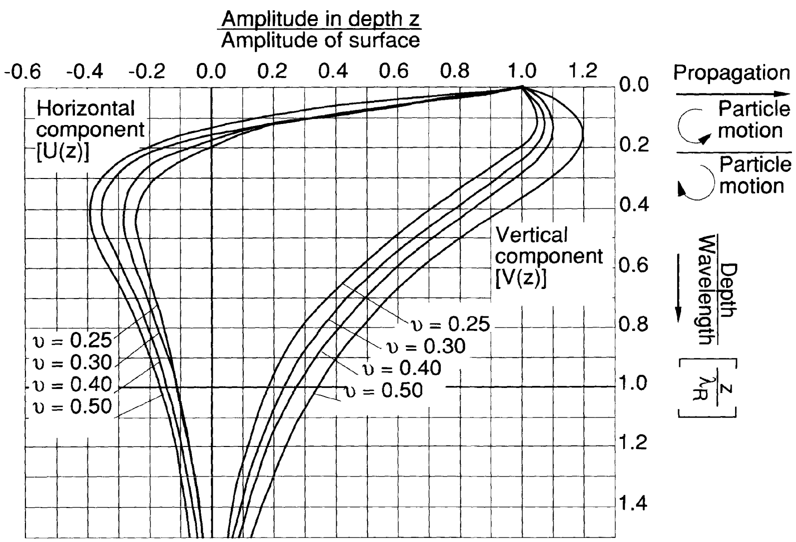


Figure E.2: Relation between amplitude ratio and dimensionless depth for Rayleigh waves with ν as parameter

In a perfectly elastic halfspace, a third type of wave is formed, the *R-wave* (Rayleigh wave), whose propagation velocity v_R can be obtained from Figure E.1. As an approximation:

$$v_R \approx \frac{v_S(0.86 + 1.14\nu)}{1 + \nu} \tag{E.3}$$

Figure E.2 shows the variation of the vertical and horizontal component of the Rayleigh wave as a function of depth z and the Poisson's ratio ν. Both components decrease sharply with depth. At a depth equal to one wave-length λ, i.e. $z = \lambda$, the vertical component is reduced by about 70%, the horizontal component by about 85%. This is therefore a surface wave. The

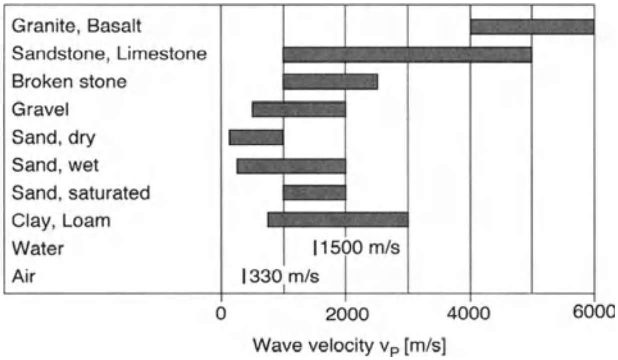


Figure E.3: Typical velocities of compression waves (P-waves) for rocks and soils

vertical component vibrates always in phase, whereas the horizontal component has a phase reversal at about $z = 0.2\lambda$ as a result of which the direction of the elliptical particle motion is reversed. Waves with low frequency components, as is the case with earthquakes, have relatively long wave-lengths. They therefore affect the earth's surface to a greater depth than high frequency waves with relatively smaller wave-lengths. The latter would be likely to originate from machine foundations. Typical recommended values for propagation velocities are presented in Figure E.3. Poisson's ratio for cohesionless loose soils can be taken as 0.3.

The propagation of vibrations in the ground is characterized by a decrease in vibration amplitudes with distance. Particle velocity is generally used as the appropriate measurement quantity. The attenuation depends on the type of the vibration source as well as the type of wave generated. The intensity of particle velocity is further reduced on account of geometric dispersion; another reduction is due to material damping of the soil.

E.3 Attenuation laws

An attenuation law is given by

$$\frac{V}{V_i} = \left(\frac{R_i}{R}\right)^n \cdot D \tag{E.4}$$

where V = vibration amplitude at distance R
V_i = vibration amplitude at reference distance R_i
n = exponent of the amplitud reduction law
D = factor taking account of the transmitting medium

The exponent n for amplitude reduction depends on

- the geometry of the vibration source (point source, line source)
- the type of excitation (stationary, impulsive)
- the predominant type of wave (Rayleigh waves on the surface, body waves at some depth).

For point sources (e.g. machine foundations) the value of n is $n = 0.5$ for surface waves from stationary excitations and $n = 1.0$ for impulsive sources. For body waves the corresponding values are $n = 1.0$ and 1.5.

For line sources (e.g. traffic) the values are $n = 0.0$ and 0.5 for surface waves and $n = 0.5$ and 1.0 for body waves.

The material term D is given by $D = \exp([-\alpha(R - R_i)])$, with the attenuation coefficient $\alpha \approx 2\pi\zeta/\lambda$: λ is the wave length, ζ is the damping ratio of the transmitting medium. For loose soils $\zeta \approx 0.01$ can be used.

Since the attenuation in amplitude can extend over many powers of 10, the measure of attenuation is often given in decibels [dB], which is defined as $\mathrm{dB} = 20 \cdot \log(Q/Q_1)$. Q_i is a fixed reference amplitude, specifically a particle velocity of 5×10^{-8} M/s. The measured amplitudes (at the source or at any point in the medium) are then referred to this reference quantity. Thus one obtains two dB-values whose difference is a measure of the attenuation of the vibration amplitudes between the two particular measurement points (see also Appendix B).

F Behaviour of concrete and steel under dynamic actions

W. Ammann, H. Nussbaumer

F.1 Introduction

Under dynamic actions most material properties - such as modulus of elasticity, strength and strain limits - change to a greater or lesser extent, when compared with the corresponding values for slow, quasi-static loading. The change is usually expressed as a function of the strain rate and in some cases also as a function of the stress rate or the rate of loading. The strain rate is defined as

$$\dot{\varepsilon} = \frac{d\varepsilon}{dt} \tag{F.1}$$

and expresses the variation of strain with time. The stress rate is defined as

$$\dot{\sigma} = \frac{d\sigma}{dt} \tag{F.2}$$

In most vibration problems the average strain rate seldom exceeds $\dot{\varepsilon} = 0.1\ s^{-1}$ so that expected changes in material properties are only moderate. A much larger effect occurs for strain rates $\dot{\varepsilon} = 1$ to $10\ s^{-1}$ as is typical of high impact forces. Dynamic actions may also influence the fatigue resistance of the structural material, even at a low number of cycles if the forces are large enough (low-cycle fatigue).

The following survey is mainly restricted to plain concrete with normal weight natural aggregates and to reinforcing steel. This survey is in accordance with a relevant chapter in [F.1]. It is not the aim of this survey to reproduce all literature data nor to give physical explanations for the relations established. Therefore this survey is not directed to materials specialists, but rather intended for structural engineers who are faced with impact problems. This appendix presents todays knowledge of materials in a condensed practical form. It allows stress or strain rates to be taken into account by means of graphs or simple empirical formulae. The changes of material property are usually described by a straight line when plotted against the strain rate on a semi-logarithmic scale and are therefore defined by an empirical factor with respect to the quasi-static value (i.e. $\dot{\varepsilon} = 5 \cdot 10^{-5}\ s^{-1}$).

The changes of material properties under the influence of dynamic actions must be considered in relation to the reliability of other values used for the calculation, such as soil properties, etc. The aging of concrete, i.e. the increase of the material properties with time is generally neglected and design values are defined at an age of 28 days.

For practical calculations (e.g. of the natural frequencies of R.C. towers or pedestals of machines) the increase in the Young's Modulus under dynamic loads may be neglected and the corresponding value at quasi-static loading to be used.

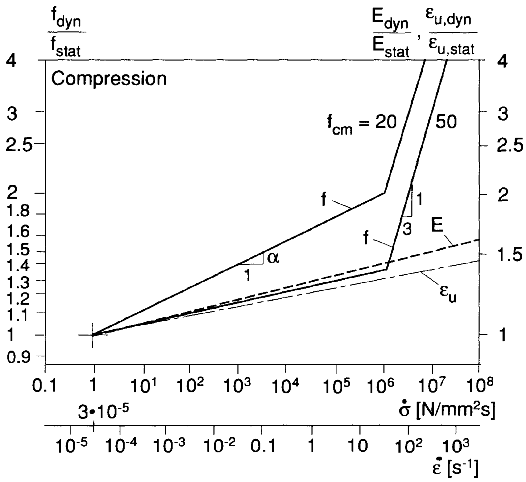


Figure F.1: Influence of stress and strain rate on concrete properties in compression [F.1]

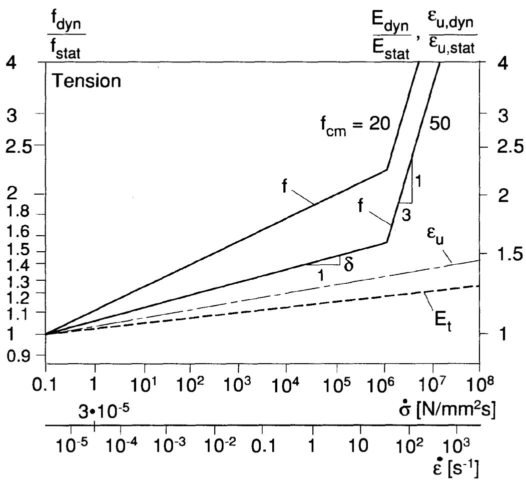


Figure F.2: Influence of stress and strain rate on concrete properties in tension [F.1]

F.2 Behaviour of concrete

F.2.1 Modulus of elasticity

The modulus of elasticity (Young's Modulus) of concrete in *compression* increases with stress and strain rate as

$$E_{dyn}/E_{stat} = (\dot{\sigma}/\dot{\sigma}_o)^{0.025} \text{ with } \dot{\sigma}_o = 1 \text{ N/mm}^2\text{s or} \tag{F.3}$$

$$E_{dyn}/E_{stat} = (\dot{\varepsilon}/\dot{\varepsilon}_o)^{0.026} \text{ with } \dot{\varepsilon}_o = 30 \cdot 10^{-6} \text{ s}^{-1} \tag{F.4}$$

These relations are supposed to be valid for all grades of concrete. Figure F.1 shows the ratio between dynamic and static modulus of elasticity as a function of stress and strain rate.

The influence of stress rate on the modulus of elasticity in *tension* is smaller than for compression. The graph of Figure F.2 can be formulated as

$$E_{dyn}/E_{stat} = (\sigma/\sigma_o)^{0.016} \text{ with } \dot{\sigma}_o = 0.1 \text{ N/mm}^2\text{s or} \tag{F.5}$$

$$E_{dyn}/E_{stat} = (\dot{\varepsilon}/\dot{\varepsilon}_o)^{0.016} \text{ with } \dot{\varepsilon}_o = 3 \cdot 10^{-6} \text{ s}^{-1} \tag{F.6}$$

These relations are valid for all stress and strain rates and all concrete grades. In most cases of dynamic action (but not impact) the increase in modulus of elasticity due to the strain rate effect does not exceed 20%.

F.2.2 Compressive strength

The compressive strength of concrete can be written in terms of strain rate as

$$f_{dyn}/f_{stat} = (\dot{\varepsilon}/\dot{\varepsilon}_o)^{1.026\alpha} \text{ with } \alpha = \frac{1}{5 + 3f_{cm}/4} \text{ for } \dot{\varepsilon} \leq 30 \text{ s}^{-1} \tag{F.7}$$

where f_{cm} = mean static cube strength of concrete [N/mm²].

This relation reveals the fact that the influence of loading rate decreases as the grade of concrete increases. The formula accounts also for the influence of the strain rate on the modulus of elasticity. If the influence of the strain rate on the modulus is not considered the power of the equation above would be 1.0 α. Figure F.1 shows the influence of stress and strain rate on the ratio of dynamic to static compressive strength. Beyond a strain rate of 30 s^{-1} the increase is very pronounced.

F.2.3 Ultimate strain in compression

The ultimate strain ε_u is the strain which occurs at maximum stress. The ultimate strain as a function of strain rate is (see Figure F.1):

$$\varepsilon_{u,dyn}/\varepsilon_{u,stat} = (\dot{\varepsilon}/\dot{\varepsilon}_o)^{0.020} \text{ with } \dot{\varepsilon}_o = 30 \cdot 10^{-6} \text{ s}^{-1} \text{ (see Figure F.1).} \tag{F.8}$$

F.2.4 Tensile strength

In contrast to compressive failure, tensile failure is always a discrete phenomenon. Usually one crack occurs which divides a specimen into two parts. The two separating parts are unloading while the crack width increases. Energy consumption occurs in the cracking zone. The formulation is similar to compressive strength except for the value of the coefficient.

Taking account again of the influence of strain rate on Young's modulus a relation can be defined between strain rate and tensile strength as follows (see Figure F.2):

$$f_{dyn}/f_{stat} = (\dot{\varepsilon}/\dot{\varepsilon}_o)^{1.016\delta} \text{ with } \delta = \frac{1}{10 + f_{cm}/2} \text{ for } \dot{\varepsilon} \leq 30 \text{ s}^{-1} \qquad \text{(F.9)}$$

where f_{cm} = mean static cube strength of concrete.

Tensile strength is more sensitive to strain or stress rate if the concrete has a low grade and is more sensitive to strain rate than compressive strength.

Usually compressive strength is the reference value for the concrete grade and is therefore known. The tensile strength can be estimated from

$$f_{tm} = 0.20 \cdot f_{cm}^{2/3} \text{ [N/mm}^2\text{]} \qquad \text{(F.10)}$$

which is a CEB-FIP recommendation for mean values of concrete strength.

F.2.5 Ultimate strain in tension

Ultimate strain ε_u is the strain at maximum stress and can be expressed as a function of strain rate with (see Figure F.2):

$$\varepsilon_{u,\,dyn}/\varepsilon_{u,\,stat} = (\dot{\varepsilon}/\dot{\varepsilon}_o)^{0.020} \text{ with } \dot{\varepsilon}_o = 3 \cdot 10^{-6} \text{ s}^{-1}. \qquad \text{(F.11)}$$

This relation is recommended for all stress and strain rates as well as all concrete grades.

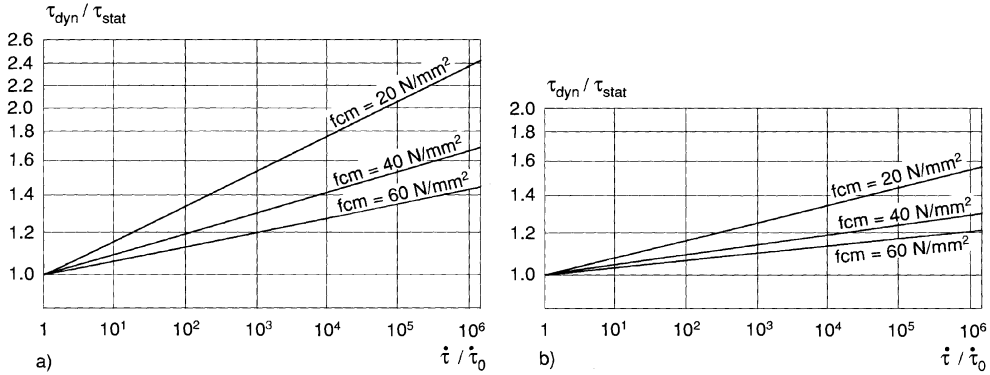


Figure F.3: Influence of stress rate on bond properties [F.2]
a) at a relative displacement of δ = 0.01 mm b) at a relative displacement of δ = 0.2 mm

F.2.6 Bond between reinforcing steel and concrete

The influence of stress rate on the bond of smooth bars and strands is negligible whereas the influence on the bond of deformed bars is considerable. It depends on concrete quality and relative displacement (slip) between steel and concrete. For the bond stress the following formulation is valid:

$$\tau_{dyn}/\tau_{stat} = (\dot{\tau}_{dyn}/\dot{\tau}_o)^{\eta} \tag{F.12}$$

where η = $0.7 \cdot (1 - 2.5\delta) / f_{cm}^{0.8}$
δ = relative displacement [mm]
f_{cm} = mean cube strength [N/mm²]
$\dot{\tau}_o$ = 0.1 N/mm²

This relation is valid under the following conditions:

$$0 < \delta < 0.2 \text{ mm (see Figure F.3) and } 0.065 < A_R/A_o < 0.1 \tag{F.13}$$

where A_R = rib area
A_o = cross section area of rebar
A_R/A_o = relative rib area

F.3 Behaviour of reinforcing steel

The influence of strain rate on steel properties can be expressed by the following generalized relationship:

$$s_{dyn}/s_{stat} = 1 + c/s_{stat} \cdot \ln(\dot{\varepsilon}/\dot{\varepsilon}_o) \tag{F.14}$$

where s_{dyn} = strength or strain value at elevated strain rate
s_{stat} = strength or strain value at quasi-static condition (s_o)
$\dot{\varepsilon}$ = strain rate
$\dot{\varepsilon}_o$ = strain rate at quasi-static condition ($\dot{\varepsilon}_o = 5 \cdot 10^{-5}$ s^{-1})
c/s_{stat} = regression coefficient

This linear approach is used for the sake of simplicity although the standard deviation of the test results is often rather large and regression lines of second or third order would provide a better fit with data.

F.3.1 Modulus of elasticity

The modulus of elasticity (Young's Modulus) remains unchanged with

$$E = 2.06 \cdot 10^5 \text{ N/mm}^2 \tag{F.15}$$

F.3.2 Strength in tension

The formulas (valid for for $\dot{\varepsilon} \leq 10 \text{ s}^{-1}$) for strength in tension are all of the form:

$$f_s/f_{so} = 1 + \kappa \cdot \ln(\dot{\varepsilon}/\dot{\varepsilon}_o) \tag{F.16}$$

where f_s = dynamic material properties with strain rate $\dot{\varepsilon}$ [N/mm²]
f_{so} = quasi-static material properties with strain rate $\dot{\varepsilon}_o$ [N/mm²]
κ = coefficient taking account of steel and strength type (see Table F.1)

A set of measured data for hot rolled steel is given in Figure F.4

	κ			
Steel type	yield stress f_{sy}/f_{syo}	proportional limit $f_{s0.2}/f_{s0.2o}$	tensile strength f_{st}/f_{sto}	ultimate strength f_{sr}/f_{sro}
Hot rolled steel	$6.0/f_{syo}$	-	$7.0/f_{sto}$	$1.5/f_{sro}$
Tempcore steel	$5.1/f_{syo}$	-	$6.4/f_{sto}$	-
Cold worked steel	-	$4.3/f_{s0.2o}$	$6.5/f_{sto}$	$1.9/f_{sro}$
Mild steel *)	$12.0/f_{syo}$	-	-	-
High quality steel**)	-	-	-	-
Prestressing wires**)	-	-	-	-
*) Only very limited data available. Strength increase is more pronounced than for hot rolled steel **) Practically no influence of strain-rate				

Table F.1: Coefficients for the calculation of strain-rate dependent strength in tension of steel

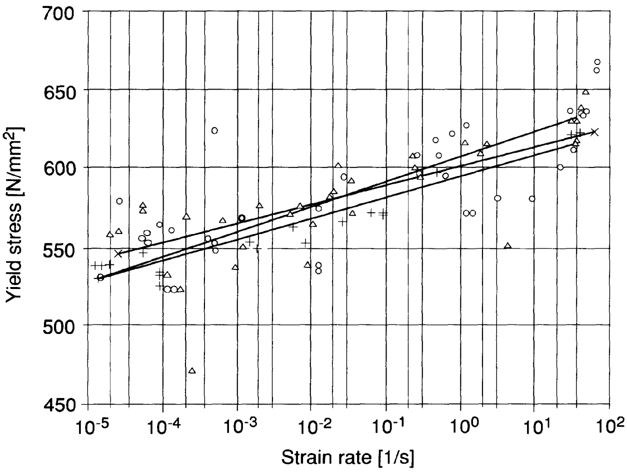


Figure F.4: Influence of strain rate on yield stress of hot rolled reinforcing steel [F.3], the regression lines indicate some minor influence of the different bar diameters

F.3.3 Strain in tension

The formulas (valid for for $\dot{\varepsilon} \le 10\ s^{-1}$) for strain in tension are all of the form:

$$\delta/\delta_o = 1 + \chi \cdot \ln(\dot{\varepsilon}/\dot{\varepsilon}_o) \text{ (strains)} \quad \text{or } \varphi/\varphi_o = \chi \text{ (necking)} \qquad \text{(F.17)}$$

where δ = dynamic material properties with strain rate $\dot{\varepsilon}$
δ_o = quasi-static material properties with strain rate $\dot{\varepsilon}_o$
χ = coefficient taking account of steel and strain type (see Table F.1)

A set of measured data for hot rolled steel is given in Figure F.5

	χ			
Steel type	uniform elongation δ_{g1}/δ_{g1o}	ultimate strain $5d$ δ_5/δ_{5o}	ultimate strain $10d$ δ_{10}/δ_{10o}	necking φ_s/φ_{so}
Hot rolled steel	$0.3/\delta_{g1o}$	$0.2/\delta_{5o}$	$0.1/\delta_{10o}$	1.0
Tempcore steel	$0.3/\delta_{g1o}$	$0.31/\delta_{5o}$	$0.27/\delta_{10o}$	-
Cold worked steel	$0.6/\delta_{g1o}$	$0.5/\delta_{5o}$	$0.6/\delta_{10o}$	1.0
Mild steel *)	-	-	-	-
High quality steel	$0.2/\delta_{g1o}$	$0.2/\delta_{5o}$	$0.2/\delta_{10o}$	-
Prestressing wires	$0.035/\delta_{g1o}$	$0.1/\delta_{5o}$	1.0 (no change)	-
*) Practically no influence of strain-rate, negligible increase				

Table F.2: Coefficients for the calculation of strain-rate dependent strength in tension of steel

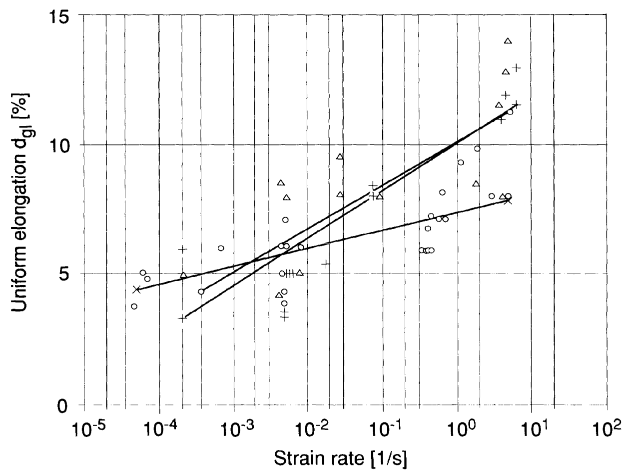


Figure F.5: Influence of strain rate on uniform elongation of cold worked reinforcing steel [F.3], the regression lines indicate some influence of the bar diameters

G Dynamic forces from rhythmical human body motions

H. Bachmann, A.J. Pretlove, H. Rainer

G.1 Rhythmical human body motions

Rhythmical human body motions lasting up to 20 seconds and more lead to almost periodic dynamic forces. These can cause more or less steady-state vibrations of structures. Such activities are often performed to rhythmical music, and if several people are involved, this will practically synchronise their motion. In such cases the dynamic forces increase almost linearly with the number of participants.

Examples of forcing functions from walking, jumping and hand clapping are given in Figures G.1 to G.4. Figure G.4 also gives the relevant continuous Fourier-amplitude spectrum (see Appendix A). These examples show that not only the frequency of the first harmonic of the Fouriertransformation of the forcing function, which is equal to the activity rate (pacing rate, jumping or clapping rate defined as steps per second, jumps per second, claps per second, etc. in Hz) but also the frequencies of upper harmonics may be of importance.

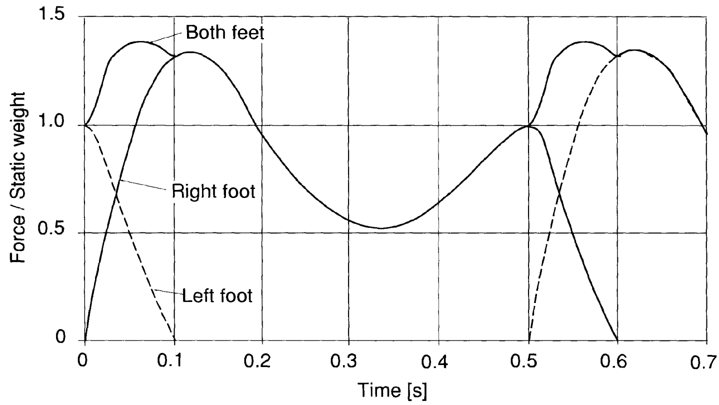


Figure G.1: Forcing function resulting from footfall overlap during walking with a pacing rate of 2 Hz [G.4]

G.2 Representative types of activity

The manifold different types of rhythmical human body motion constitute a large variety of possible dynamic forces. For design purposes, however, the following *representative types of activity* will be chosen:

- walking
- running
- jumping
- dancing
- hand clapping with body bouncing while standing
- hand clapping
- lateral body swaying

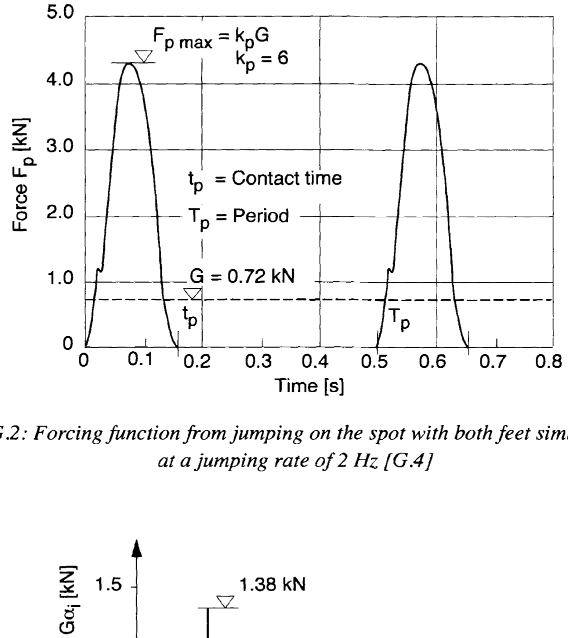


Figure G.2: Forcing function from jumping on the spot with both feet simultaneously at a jumping rate of 2 Hz [G.4]

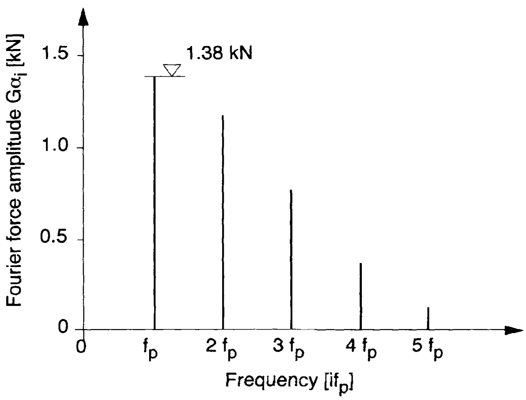


Figure G.3: Discrete Fourier amplitude spectrum for the forcing function from jumping of Figure G.2 up to the fifth harmonic

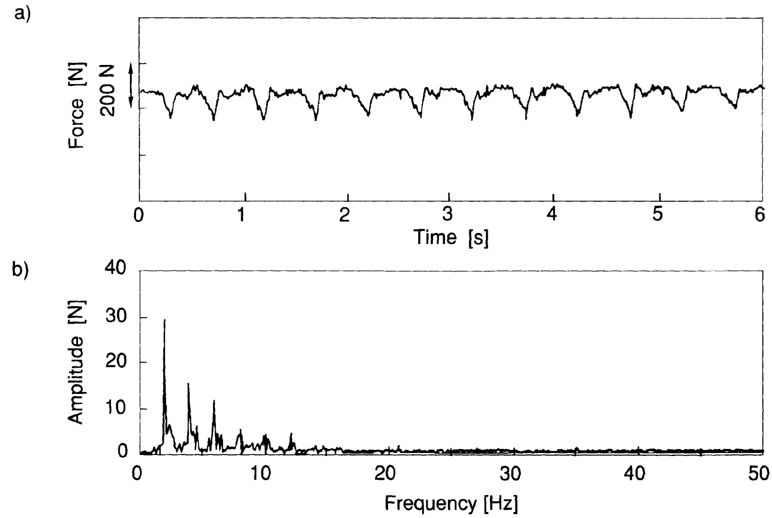


Figure G.4: Forcing function (a) and continuous Fourier-amplitude spectrum (b) from normal rhythmical hand clapping by a seated person with a clapping rate of 2 Hz [G.5]

Table G.1 defines the representative types of activity and gives the relevant ranges of the design activity rates (e.g. pacing, jumping, dancing, hand clapping rate). The columns to the right give the corresponding actual activities and types of structures for which the representative types of activity apply.

G.3 Normalised dynamic forces

To every representative type of activity a normalised dynamic force can be assigned.

The forcing function due to a person's rhythmical body motion can be mathematically described by a Fourier series of the form (Appendix A, Equation (A.27) and [G.1], [G.3], [G.4]):

$$F_p(t) = G + \sum_{i=1}^{n} G \cdot \alpha_i \cdot \sin(2\pi i f_p t - \phi_i) \tag{G.1}$$

where G = weight of the person ("notional pedestrian" of G = 800 N)

α_i = Fourier coefficient of the i-th harmonic

$G \cdot \alpha_i$ = force amplitude of the i-th harmonic equivalent to A_i used in Appendix A

f_p = activity rate (Hz)

ϕ_i = phase lag of the i-th harmonic relative to the 1st harmonic

i = number of the i-th harmonic

n = total number of contributing harmonics

The graphical representation of the force amplitudes of the harmonics is also called the discrete Fourier amplitude spectrum (see Appendix A). An example of the force amplitudes $G\alpha_i$ up to the 5th harmonic of the forcing function of Figure G.2 is given in Figure G.3.

The Fourier coefficients α_i and partly also the phase angles ϕ_i for various human activities have been ascertained in experimental research (e.g. [G.1], [G.2], [G.3], [G.4], [G.5]). In Table G.2 pertinent values for the normalised dynamic forces assigned to the representative types of activity of Table G.1 are given together with an indication of commonly attained design density of persons.

Representative types of activity			Range of applicability		
Designation	Definition	Design activity rate [Hz]	Actual activities	Activity rate [Hz]	Structure type
"walking"	walking with continuous ground contact	1.6 to 2.4	• slow walking (ambling) • normal walking • fast, brisk walking	~ 1.7 ~ 2.0 ~ 2.3	• pedestrian structures (pedestrian bridges, stairs, piers, etc.) • office buildings, etc.
"running"	running with discontinuous ground contact	2.0 to 3.5	• slow running (jog) • normal running • fast running (sprint)	~ 2.1 ~ 2.5 > 3.0	• pedestrian bridges on jogging tracks, etc.
"jumping"	normal to high rhythmical jumpingon the spot with simultaneous ground contact of both feet	1.8 to 3.4	• fitness training with jumping, skipping and running to rhythmical music • jazz dance training	~ 1.5 to 3.4 ~ 1.8 to 3.5	• gymnasia, sport halls • gymnastic training rooms
"dancing"	approximatly equivalent to "brisk walking"	1.5 to 3.0	• social events with classical and modern dancing (e.g. English Waltz, Rumba etc.)	~ 1.5 to 3.0	• dance halls • concert halls and other community halls without fixed seating
"hand clapping with body bouncing while standing"	rhythmical hand clapping in front of one's chest or above the head while bouncing vertically by forward and backward knee movement of about 50 mm	1.5 to 3.0	• pop concerts with enthusiastic audience	~ 1.5 to 3.0	• concert halls and spectator galleries with and without fixed seating and "hard" pop concerts
"hand clapping"	rhythmical hand clapping in front of one's chest	1.5 to 3.0	• classical concerts, "soft" pop concerts	~ 1.5 to 3.0	• concert halls with fixed seating (no "hard" pop concerts)
"lateral body swaying"	rhythmical lateral body swaying while being seated or standing	0.4 to 0.7	• concerts, social events		• spectator galleries

Table G.1: Representative types of activities and their applicability to different actual activities and types of structures

Representative type of activity	Activity rate [Hz]		Fourier coefficient and phase lag					Design density [persons/m^2]
			α_1	α_2	ϕ_2	α_3	ϕ_3	
"walking"	vertical	2.0	0.4	0.1	$\pi/2$	0.1	$\pi/2$	~ 1
		2.4	0.5					
	forward	2.0	0.2	0.1				
			$\alpha_{1/2} = 0.1$					
	lateral	2.0	$\alpha_{1/2} = 0.1$	$\alpha_{3/2} = 0.1$				
"running"		2.0 to 3.0	1.6	0.7		0.2		-
"jumping"	normal	2.0	1.8	1.3	*)	0.7	*)	in fitness training ~ 0.25
		3.0	1.7	1.1	*)	0.5	*)	(in extreme cases up to 0.5)
	high	2.0	1.9	1.6	*)	1.1	*)	
		3.0	1.8	1.3	*)	0.8	*)	*) $\phi_2 = \phi_3 = \pi(1 - f_p t_p)$
"dancing"		2.0 to 3.0	0.5	0.15		0.1		~ 4 (in extreme cases up to 6)
"hand clapping with body bouncing while standing"		1.6	0.17	0.10		0.04		no fixed seating ~ 4
		2.4	0.38	0.12		0.02		(in extreme cases up to ~ 6)
								with fixed seating ~ 2 to 3
"hand clapping"	normal	1.6	0.024	0.010		0.009		
		2.4	0.047	0.024		0.015		~ 2 to 3
	intensive	2.0	0.170	0.047		0.037		
"lateral body swaying"	seated	0.6	$\alpha_{1/2} = 0.4$	-		-		
	standing	0.6	$\alpha_{1/2} = 0.5$	-		-		~ 3 to 4

Table G.2: Normalized dynamic forces assigned to the representative types of activity defined in Table G.1

H Dynamic effects from wind

G. Hirsch, H. Bachmann

H.1 Basic theory

H.1.1 Wind speed and pressure

Wind is caused by temperature and pressure differences in the atmosphere. The wind blows irregularly over a period of time and its intensity and direction are continually changing. At a particular site and a particular time the wind speed increases over the height of a turbulent boundary layer several hundred meters thick up to the so-called gradient speed, which is no longer influenced by the roughness of the earth's surface. Figure H.1 shows a "quasi-stationary" wind profile, with superimposed stochastically varying turbulences (vortices and wind speed fluctuations):

$$u_{tot}(z, t) = \bar{u}(z) + u(z, t) \tag{H.1}$$

where $\bar{u}(z) = \bar{u}_z$ = mean wind speed at height z
$u(z, t) = u_z$ = fraction of wind speed due to turbulence
$u_{tot}(z, t) = u_{z,\,tot}$ = gust speed (total speed)

But $\bar{u}(z)$ also changes with time, so that the representation in Figure H.1 corresponds to a particular instant in time. Turbulence is caused by flow around different natural and artificial bluff objects and is distributed spectrally over a large range of frequencies.

The wind profile can be described by the following exponential law

$$\bar{u}_z = \bar{u}_{10} \cdot \left(\frac{z}{10}\right)^{\alpha} \tag{H.2}$$

where $\bar{u}_{10} = u_{ref}$ = reference wind speed at a height of $z_{ref} = 10$ m above ground in a free field (wind measurements at weather stations)
α = roughness coefficient ($\alpha = 0.16$ for open country, $\alpha = 0.28$ for woods, villages and towns, $\alpha = 0.40$ for large city centres)

The reference wind speed $u_{ref} = \bar{u}_{10}$ (and also $\bar{u}_z$) is generally defined for a 10-minute-mean value and a return period of 50 years. Depending on the geographical location of the

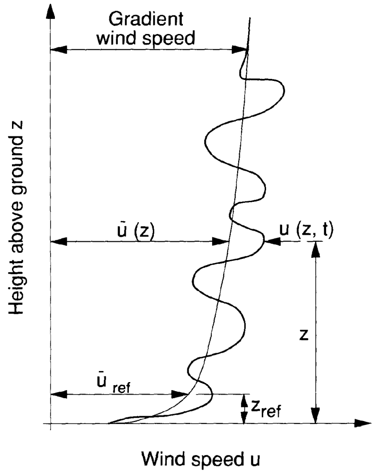


Figure H.1: Wind profile with superimposed turbulence

measuring site, $\bar{u}_{ref}$ lies usually between 24 and 34 m/s (see e.g. wind map for Europe with lines of equal wind velocity in [H.6]).

$\bar{u}_{10}$ and $\bar{u}_z$ can be related to or converted to other return periods [H.6]. In some codes a reference speed for a 5-second-mean value is used, which corresponds to a factor of 1.4 to 1.5. $\bar{u}_{10}$ can be converted with the help of Equation (H.1) to the gradient speed (defined at heights of 300 to 500 m depending on the roughness) and vice versa.

The gust fraction $u(z, t)$ and thus the gust speed (total speed) u_{tot} are usually defined for the 5-second-mean value of wind measurements.

From the mean wind speed, the wind pressure is

$$\bar{q} = \frac{\rho}{2} \cdot \bar{u}_z^2 \qquad \text{(H.3)}$$

where ρ = air density (1.2 kg/m^3 at normal temperatures)

H.1.2 Statistical characteristics

The stochastic part of the wind speed $u(z, t)$, which varies over the height above ground and with time, requires the use of statistical methods for the calculation of the dynamic forces on the structure. The basic statistical properties for the variation of the wind speed are described by the following quantities [H.1],[H.6], [H.7], [H.10]:

- degree of turbulence (turbulence intensity)
- power spectral density
- correlation between wind speed at several points
- probability distribution.

From these the following functions may be derived:

- gust spectrum
- aerodynamic admittance function
- spectral density of the wind force.

These functions are shown in Figure H.2 (symbols which are not defined are not used in this book) and are described qualitatively in the following.

a) Gust spectrum

The time variation of wind speed and thus the wind turbulence at a particular point contains a variety of frequencies with differing amplitudes. A Fourier analysis and some further numerical operations allow the turbulence spectrum (= gust spectrum) to be calculated, whose ordinate corresponds to the energy contained in the wind (energy spectrum, spectral density). A peak value in the curve signifies an energy maximum at a particular frequency. Measured spectra usually exhibit a peak at frequencies between 0.05 and 0.01 Hz or between periods of 20 to 100 seconds. Although very large structures (towers, high-rise buildings with fundamental periods in the range 5 to 8 s) and numerous smaller structures (with frequencies of 1 Hz and beyond) fall into the low energy part of the spectrum, the dynamic response may still be quite significant, especially if the structure's damping is small.

Measured gust spectra exhibit some jagged variation about a mean value, but design spectra are usually presented as smoothed curves (Figure H.2a).

b) Aerodynamic admittance function

There exists a relationship between the gust frequency and its area of influence and therefore between the effect of the gust at a certain frequency and the structure's effective dimensions (e.g. width or square root of width times height/2). Gusts of higher frequency have a smaller area of influence and vice versa, i.e. the lower the frequency of a gust, the larger its area of influence. If small gusts hit a structure of large surface area, only a part of the structure is affected and therefore the influence of the gusts is reduced. For a particular surface area this is taken into consideration by the aerodynamic admittance function, which can also be viewed as a surface correction factor (Figure H.2b).

c) Spectral density of the wind force

The product of the gust spectrum and the square of the aerodynamic admittance function gives the spectral density of the wind force, i.e. the force effectively acting on the structure (Figure H.2c).

H.1.3 Dynamic effects

Figure H.3 shows a possible classification of dynamic effects of wind on structures.

Vibrations in the wind direction can result directly from pressure fluctuations of gusts or indirectly from increased turbulence from vortex-shedding in the wake of obstacles (buffeting). The latter effect belongs to the class of aerodynamic interference between different structures.

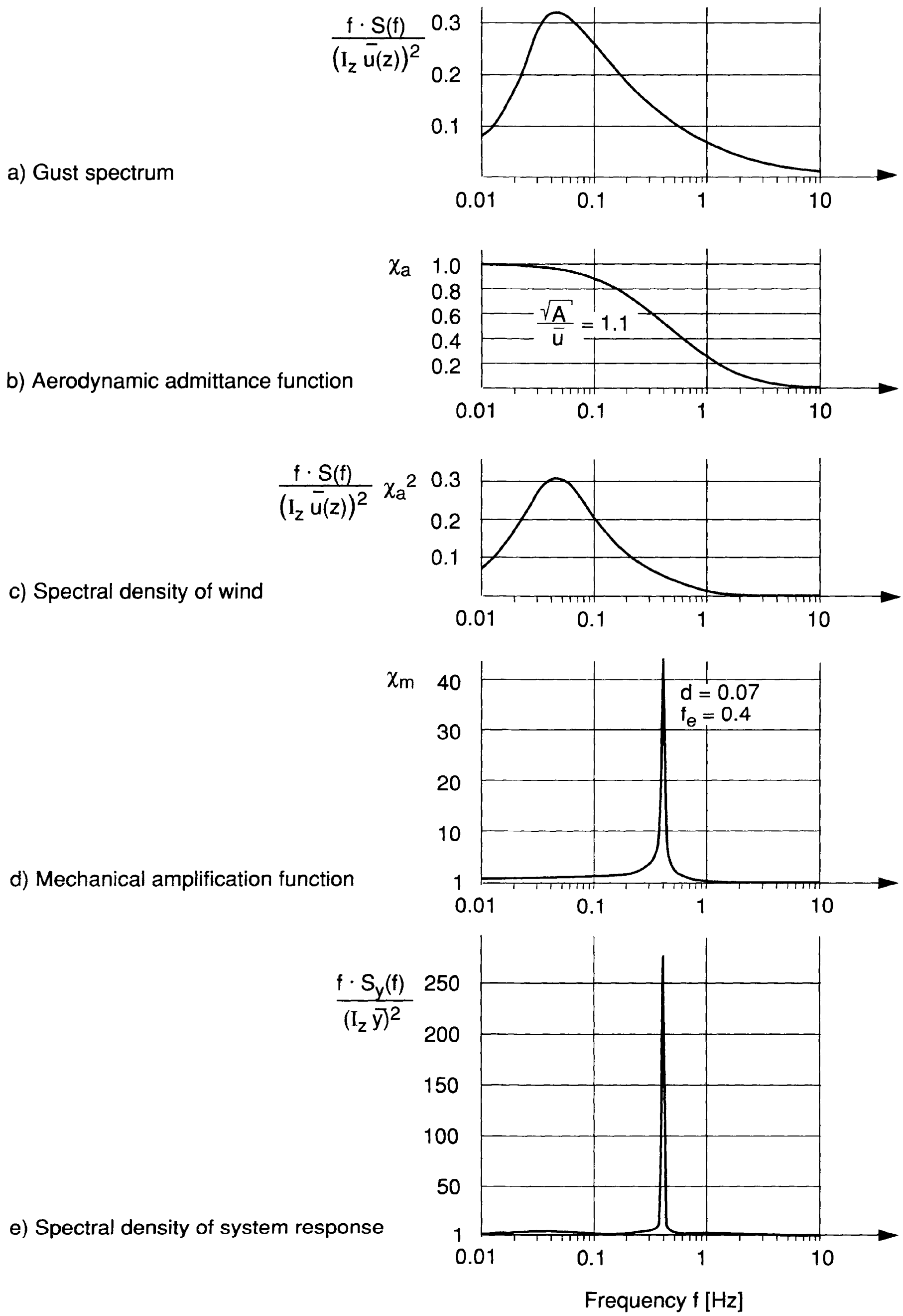


Figure H.2: Spectral densities and magnification functions

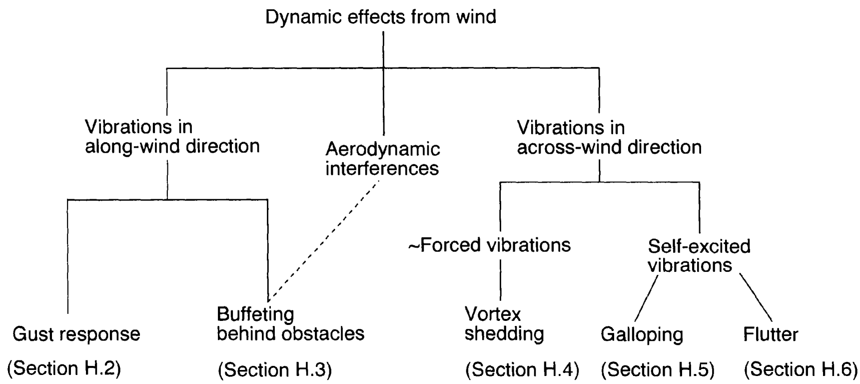


Figure H.3: Classification of dynamic effects from wind

Vibrations in the across-wind direction can arise from vortex-shedding - mainly as forced vibrations. Finally, one may distinguish between self-excited vibrations of galloping and flutter. These latter ones are based on interaction between an elastic structure and the wind and are also called "aeroelastic effects".

H.2 Vibrations in along-wind direction induced by gusts

The variation of the wind speed (gusts) causes a dynamic effect on the structure exposed to the wind. Gust-induced structural vibrations along the wind direction can result.

H.2.1 Spectral methods

Based on the spectral density of the wind force (Figure H.2c) the spectral density of the system response (dynamic response of structure) can be determined with the aid of the mechanical amplification function.

a) Mechanical amplification function

The mechanical amplification function (Figure H.2d) represents the dynamic magnification factor for a particular excitation frequency (see Figure A.2). It represents the response of the structure to harmonic excitation and exhibits resonance at the natural frequencies of the structure.

b) Spectral density of the system response

The spectral density of the system response (Figure H.2e) is obtained by multiplying the spectral density of the wind force with the square of the mechanical amplification function. In this way most of the maximum stresses induced in the structure at its fundamental frequency can be estimated (Example given in [H.6]).

H.2.2 Static equivalent force method based on stochastic loading

For structural design purposes the dynamic effect of the wind can be taken into account by defining static equivalent forces.

The static equivalent wind force W can be represented by the following equation [H.5]

$$W = \varphi \cdot c_f \cdot \bar{q}_{rel} \cdot A \tag{H.4}$$

where $\bar{q}_{rel} = \left(\sum_{i=1}^{n} \bar{q}_i \cdot A_i \right) / A$

φ = gust factor

c_f = aerodynamic resistance coefficient of the complete structure according to Figure H.4 (rectangular cross-sections) and Table H.1 (polygonal and circular cross-sections)

A = area of surface loaded by wind

A_i = area of wind-loaded sub-surface in section i of structure

n = number of sub-surfaces A_i of surface A

$\bar{q}_i$ = $\rho/2 \cdot \bar{u}_z^2$ = wind pressure on A_i

$\bar{u}_z$ = mean wind speed at height z of sub-surface A_i, determined from Equation (H.2) from the reference wind speed $u_{ref} = \bar{u}_{10}$

The gust factor φ considers both the stochastically varying aerodynamic properties of the natural wind and the vibrational behaviour of the structure in its fundamental mode (natural frequency and damping). Based on the spectral methods proposed in [H.7] the following equation is also used:

$$\varphi = 1 + R\sqrt{B + \frac{s \cdot F}{\Lambda}} \tag{H.5}$$

where R = terrain factor (roughness factor) according to Figure H.5, which depends on the height of structure h and the statistical peak factor g (i.e. $g = 3.5$). The straight line in Figure H.5 is valid for $\alpha = 0.16$. The value given in Figure H.5 has to be multiplied by 1.3 for $\alpha = 0.22$ and by 1.8 for $\alpha = 0.35$ [H.2].

B = basic gust factor (takes into account the influence of gusts that are not amplified by the dynamic properties of the structure), and according to Figure H.6, is a function of d/h and h

d = width of structure subjected to wind loading

h = height of structure

s = size factor according to Figure H.7 which depends on the "reference frequency" $f_e \cdot h/\bar{u}_h$ and ratio d/h

f_e = fundamental frequency of structure in the wind direction

$\bar{u}_h$ = mean wind speed at height h

F = gust energy factor according to Figure H.8, which depends on the wave number $f_e/\bar{u}_h$

Λ = damping (expressed as a logarithmic decrement) of the fundamental mode of the structure (see Chapter 3 and Appendices A, and C)

The second expression under the square-root sign accounts for the effects of gusts, as enhanced by the dynamic properties of the structure. Equations (H.4) to (H.6) represent the formulas for the spectral analysis method.

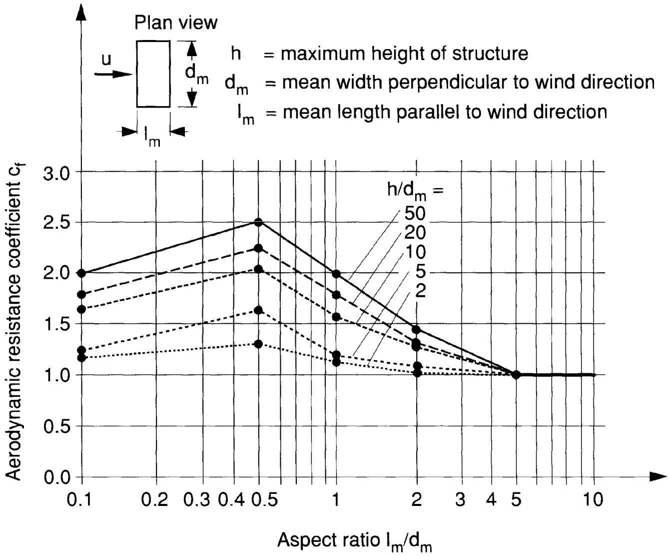


Figure H.4: Aerodynamic resistance coefficient for rectangular cross-sections (for cross-sections other than rectangular, use as an approximation the circumscribed rectangle)

h/d_m	Polygon with n =			Circle with diameter d_m = (Only valid for reinforced concrete structures because surface roughness coefficient is included)					
⬡ d_m	6	8	10	1m	2m	3m	10m	20m	50m	100m
≤ 2	0.85	0.80	0.75	0.65	0.63	0.61	0.60	0.54	0.49	0.48
5	0.90	0.85	0.80	0.77	0.74	0.71	0.70	0.63	0.57	0.56
10	1.20	1.15	1.05	0.88	0.84	0.82	0.80	0.72	0.66	0.64
20	1.35	1.25	1.20	1.00	0.95	0.92	0.90	0.81	0.74	0.72
≥ 50	1.50	1.40	1.30	1.10	1.05	1.02	1.00	0.90	0.82	0.80

Table H.1: Aerodynamic resistance coefficient for polygonal and circular cross-sections (h = maximum height of structure, d_m = mean width perpendicular to wind direction or mean diameter, n = number of edges)

If it is assumed that the structure vibrates harmonically at its fundamental frequency, then the maximum velocity and acceleration due to the dynamic part of the equivalent wind force W is

$$v_{max} = y_{tot}(\varphi - 1)\, 2\,(\pi \cdot f_e) \quad \text{and} \quad a_{max} = y_{tot}(\varphi - 1)\, 2\,(\pi \cdot f_e)^2 \qquad \text{(H.6)}$$

where y_{tot} = bending displacement at the top of the structure due to wind force W from Equation (H.4).

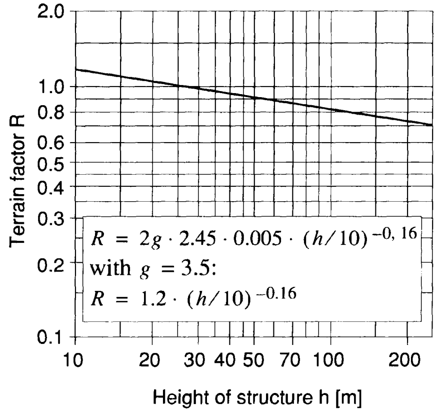


Figure H.5: Terrain factor

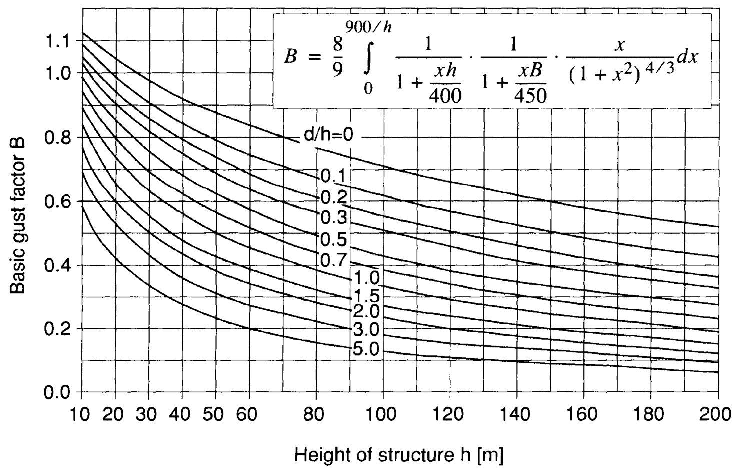


Figure H.6: Basic gust factor

For the case of gust-induced vibrations of structures in the wind direction it is found that *an increase of the structure's damping* by an amount normally possible *does not result in any substantial decrease of the dynamic response.* Passive "mass-dampers" (see Section 3.1.8) offer only slightly better results. This is due to the fact that the peak vibrational response of a structure with various amounts of damping changes little when subjected to a transient (impulse-type) excitation. The dynamic effect of a single gust corresponds more or less to an impulse load. In the past, this fact was generally utilized in the calculation of the wind forces on the structure by deterministic static equivalent force methods.

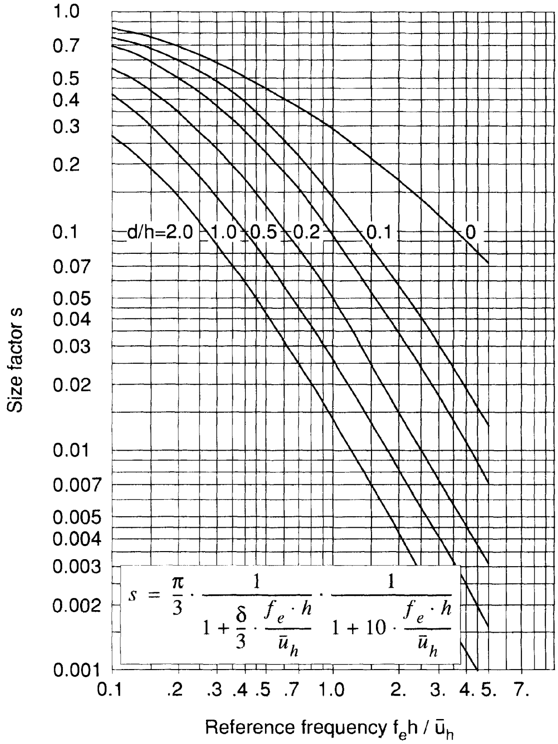


Figure H.7: Size factor

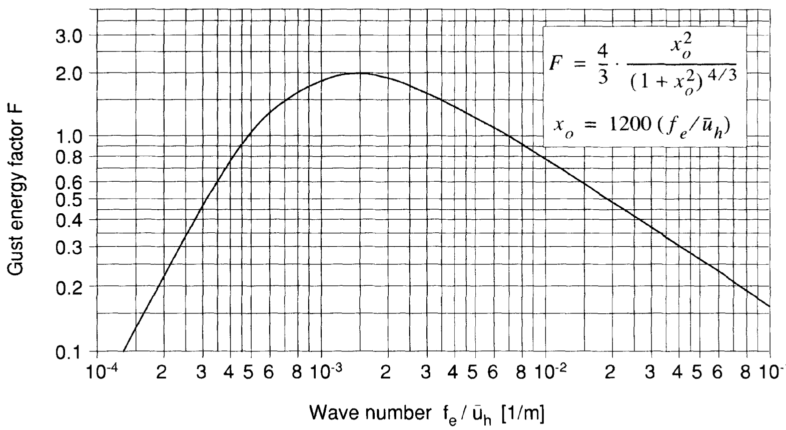


Figure H.8: Gust energy factor

H.2.3 Static equivalent force method based on deterministic loading

The static equivalent force methods based on deterministic loading have been frequently used in the past. However, various assumptions made in the derivation of d are not fully justified from the point of view of the statistical character of wind turbulence. The spectral method and the method derived from it for determining a static equivalent force is thus the preferred one for structural design.

H.2.4 Remedial measures

A rather ineffective measure for reducing the gust-induced vibration in the wind direction - as presented in Appendix H.2.2 - is to increase the damping of the structure. Even mass-dampers are not effective in the case of stochastic single gusts. This is in contrast to buffeting which occurs in urban centres containing numerous high-rise buildings; see Section H.3. The only remaining possibility is in general a *stiffer design of the structure.*

H.3 Vibrations in along-wind direction induced by buffeting

The airflow behind obstacles exhibits increased turbulence. This can be of a stochastic nature, but it may also consist of regular, i.e. periodic, vortices that are shed in the wake of cylindrical obstacles. These vortices give rise to periodic dynamic forces and act on structures that lie within such an air flow. This phenomenon is called buffeting. Buffeting can lead to substantial vibrations in the wind direction, particularly in the case of resonance, and to correspondingly high stresses in the structure. The frequency of the vortices in the airflow behind the obstacle and thus the frequency of the periodic excitation force acting on the structure is given by

$$f_w = \frac{S \cdot u}{d} \tag{H.7}$$

where f_w = frequency of vortices
S = Strouhal number
u = wind speed [m/s]
d = diameter of obstacle [m] (or the characteristic dimension of body for non-circular section; see Figure H.9 and Table H.2).

The Strouhal number relates the frequency of vortex shedding to a characteristic dimension of the body (e.g. width in across-wind direction) and the speed of air flow. For cylindrical bodies it may be assumed that $S = 0.2$ and is independent of Reynold's number. Figure H.9 shows Strouhal numbers for rectangular sections as a function of the aspect ratio. For other sectional shapes the Strouhal numbers and the corresponding range of Reynold's number are summarized in Table H.2.

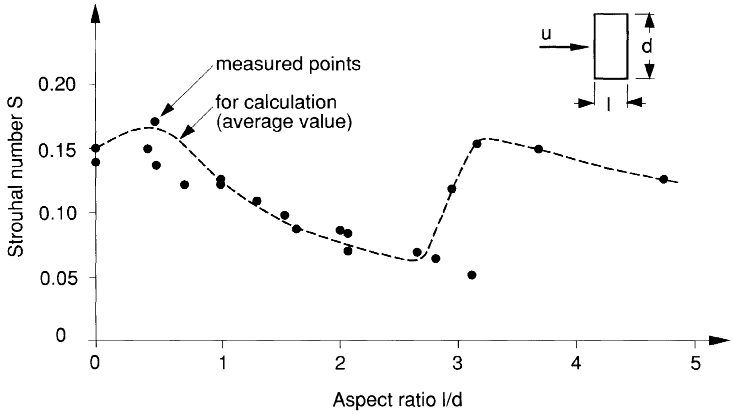


Figure H.9: Strouhal numbers for rectangular sections

The Reynold's number characterizes the ratio of inertia to frictional forces and is defined as

$$Re = \frac{d \cdot u}{\vartheta} \tag{H.8}$$

where ϑ = kinematic viscosity of the flowing medium
$\vartheta = \vartheta_{air} = 15 \cdot 10^{-6}$ m^2/s for practical purposes

At the frequency defined by the Strouhal number, the spectrum of the gusts acting in the wake of a structure is modified. The energy increases at this frequency, but it is difficult to give exact numbers. Often it is necessary to carry out wind tunnel tests to obtain the data.

Of importance is the influence of the ratio of the distance a between the obstacle and the structure (i.e. between the disturbing and the disturbed structures) and the width d of the obstacle. Considerable dynamic effects are obtained over values of a/d ranging from about 2 to 16. The maximum is reached at a/d of approximately 8 [H.6]. It must also be remembered that the dynamic effects on existing structures can change significantly when another structure is built in their windward direction. The centres of large cities thus remain in the "construction stage" for as long as other tall buildings are being built.

As a preventive measure against vibrations from buffeting the disturbed structure downwind can be strengthened, thereby increasing its fundamental frequency f_e *(frequency tuning)* and avoiding resonance. However, this is often not possible because the wind speeds can vary between wide limits. Since the wind effect is periodic its influence can be effectively reduced by *increasing the damping* such as by using *mass-dampers*. In simple cases the vortex shedding behaviour of the disturbing structure can be modified by *aerodynamic means* (e.g. the Scruton helical stabilizing device, see Sub-Chapter 4.1).

In addition to unfavourable dynamic influences, sheltered zones can also occur where the stationary wind force is reduced. This effect dies away quickly with increasing distance from the disturbing structure. In the case of rectangular-shaped buildings wind shelter effect is negligible if the distance-to-width ratio is above 14.

a)

Shape of cross-section	Strouhal number $S = \frac{f_w \cdot d}{u_\infty}$	Valid range of Reynold's number
u_∞; B; r; d; $\frac{B}{d} = 1$, $\frac{r}{d} = \frac{1}{3}$	0.33	$2 \times 10^6 > Re > 4 \times 10^5$
B; r; w; d; $\frac{B}{d} = 1$, $\frac{r}{w} = \frac{1}{3}$	$0.2 \rightarrow 0.35$ 0.35	$7 \times 10^5 > Re > 4 \times 10^5$ $2 \times 10^6 > Re > 7 \times 10^5$
B; r; d; w; $\frac{B}{d} = 1$, $\frac{r}{w} = \frac{1}{4}$	0.2 0.3	$8 \times 10^5 > Re > 3 \times 10^5$ $Re > 3 \times 10^5$
B; r; d; w; $\frac{B}{d} = 1$, $\frac{r}{w} = \frac{1}{4}$	0.2 0.65	$5 \times 10^5 > Re > 3 \times 10^5$ $1.6 \times 10^6 > Re > 6 \times 10^5$
B; r; d; $\frac{B}{d} = 2$, $\frac{r}{d} = \frac{1}{2}$	0.4	$2.5 \times 10^6 > Re > 3 \times 10^5$
B; r; d; $\frac{B}{d} = \frac{1}{2}$, $\frac{r}{d} = \frac{1}{4}$	$0.2 \rightarrow 0.35$ 0.35	$6 \times 10^5 > Re > 2 \times 10^5$ $1 \times 10^6 > Re > 6 \times 10^5$
B; d; Ellipse $\frac{B}{d} = 2$	0.12 0.60	$5 \times 10^5 > Re > 3 \times 10^5$ $2 \times 10^6 > Re > 1 \times 10^6$
B; d; Ellipse $\frac{B}{d} = \frac{1}{2}$	0.2	$7 \times 10^5 > Re > 1 \times 10^5$
d	0.22 0.125	$Re > 8 \times 10^4$ $Re > 5 \times 10^4$
d	$0.13 \rightarrow 0.22$	$Re = 0.3 \div 1.4 \times 10^5$
d	$0.14 \rightarrow 0.22$	$Re > 0.8 \times 10^5$

b)

Profile	Wind-direction	$S = f_w \cdot \frac{d}{u_\infty}$	Profile	Wind-direction	$S = f_w \cdot \frac{d}{u_\infty}$	Profile	Wind-direction	$S = f_w \cdot \frac{d}{u_\infty}$
d; d		0.14 0.12	0.125d; d		0.17 0.18	d		0.15
0.5d; d		0.14 0.18	0.062d; d		0.18 0.18	d; d; d		0.12
0.25d; d		0.15	d		0.15 0.16	0.5d; d; d		0.14
0.125d; d		0.18 0.16	0.5d; 0.25d; d		0.18	0.5d; d		0.11 0.15
0.5d; d		0.15 0.14 0.18	2x0.25d; d		0.15	2d; d		0.16
0.25d; d		0.17 0.15	d; d		0.14 0.15	d		0.20

Table H.2: Strouhal numbers a) for closed sectional shapes, b) for open and for circular sectional shapes [H.6]

H.4 Vibrations in across-wind direction induced by vortex-shedding

H.4.1 Single structures

For slender cylindrical structures, vortices are shed along the direction of the wind alternatively to the left and the right of the cross-section. This produces pulsating excitation forces in the across-wind direction. For vertical structures, such as chimneys etc., the lines of vortex shedding are vertical, i.e. with respect to the height the vortices are always shed at the same place on the cross-section and at the same time. Thus, due to the airflow the structure can be significantly excited especially in the bending mode. If the vortex shedding frequency f_w is the same as the natural frequency f_e of the structure then resonance occurs. This is the case for the critical wind speed u_{cr} (see Equation (H.7))

$$u_{cr} = \frac{f_e \cdot d}{S} \tag{H.9}$$

Since the process occurs for a structure that is not in motion, one may consider this to be a forced vibration. But it can also lead to a so-called lock-in effect, where due to the motion of the structure the vortex shedding synchronizes with the natural frequency of the structure in a certain range below and above the critical wind speed. As shown in Figure H.10a the vortex shedding frequency remains practically constant in the region of synchronization despite changes in wind speed. The result is a considerable broadening of the region of induced vibration. Figure H.10b shows a narrowband mechanical amplification function which would be decisive for a pure forced vibration for the case of a weakly damped structure ($\Lambda = 0.01$). However, for a critical vortex excitation with lock-in effect, the response amplitudes are increased greatly on both sides of the critical wind speed, effectively broadening the amplification function.

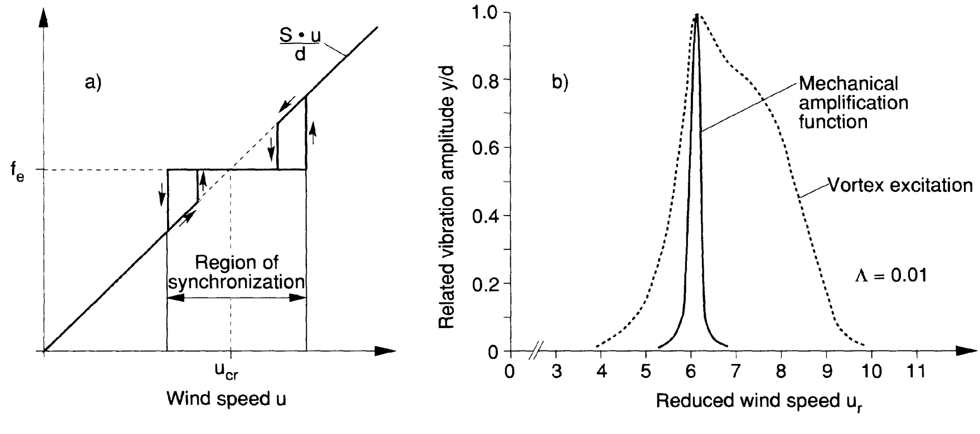


Figure H.10: Vortex resonance; a) synchronization of vortex shedding and structure frequencies, b) across-wind vibration amplitude (y/d) referred to reduced wind speed $u_r = u/df_e$

The across-wind vibration amplitude y is best related to the dimension d of the cylinder subjected to airflow. There are limits to the size of resonance amplification, and these limits are primarily due to the nonlinear aerodynamics at large amplitude vibration. The damping and mass distributions in the structure play an important role. Both values are incorporated in the Scruton number, the so-called mass-damping parameter:

$$Sc = \frac{2 \cdot m \cdot \Lambda}{\rho \cdot d^2} \tag{H.10}$$

where m = mass/unit length [kg/m]
Λ = logarithmic decrement
ρ = air density (1.2 kg/m^3)
d = dimension in across-wind direction

The ratio y_o/d of the across-wind vibration amplitude y_o at the top of the cantilever to the dimension d may be determined from the following relation given in the draft of DIN 4133, Appendix A (1991):

$$y_o/d = 0.123 \cdot c_L \cdot (1/S^2) \cdot (1/Sc) \tag{H.11}$$

where S = Strouhal number (see Equation (H.7))
c_L = aerodynamic lift coefficient
Sc = Scruton number

c_L depends on the Reynold's number (see Equation (H.8)) and is given in Figure H.11 for a circular cylinder. If two cylinders are located one behind the other, c_L increases by a factor 1.5 for a ratio of distance between axes to diameter, a/d, less than 15. Such "interference galloping" [3.13] can be avoided by use of structural connections. It is interesting to note that wind speed only enters Equation (H.11) through the Reynold's number (in c_L) and the natural frequency f_e does not appear at all, i.e. it cancels out.

An alternative to Equation (H.11) is obtained by using the amplitude of the dynamic force for the structure under vortex-resonance conditions [DIN 4133]:

$$p_{cr} = c_L \cdot \frac{\rho}{2} \cdot u_{cr}^2 \cdot d \tag{H.12}$$

The static response to p_{cr} [N/m] increases with the dynamic amplification factor $\pi/\Lambda = 1/(2\zeta)$ (case of resonance due to vortex excitation).

For the case of relatively large damping and widely distributed mass, vortex excitation can also cause critical resonance and can lead to considerable dynamic stresses.

Now a word concerning the conditions at the top of the cantilever and the so-called correlation length. The disturbances which emanate from the three-dimensional flow around the end of the cylinder do not form any regular vortices over a distance of about 1.5 diameters from the end. As a result, the correlation length for vortex shedding and thus the dynamic deflection is reduced. In Equation (H.11) a correlation length was conservatively assumed to be equal to the cylinder length. In general, however, the effective correlation length amounts to about 60% of the cylinder length so that the dynamic deflection would be about 40% less than the value conservatively calculated.

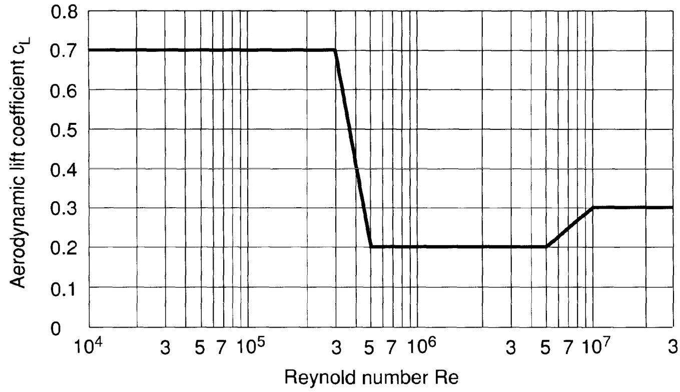


Figure H.11: Aerodynamic lift coefficient for circular cylinder

To calculate the stresses caused by the across-wind vibration amplitude y_o at the top of the cantilever it may be assumed that at maximum deflection the inertia forces are in equilibrium with the elastic deformation forces in the structure. Therefore the inertia forces can be applied like static forces. With mass m_i per increment of length Δz_i and the normalized mode shape (bending mode), which at height z_i of section i has the factor Φ_i, the inertia force can be formulated at this height as:

$$F_i = m_i \cdot \Phi_i \cdot y_o \cdot (2 \cdot \pi \cdot f_e)^2 \tag{H.13}$$

The normalised mode shape can be extracted from the previous calculation of the bending natural frequency. Alternatively, the following expression can be used:

$$\Phi_i = 1 - \cos\left(\frac{\pi \cdot z_i}{2 \cdot h}\right) \tag{H.14}$$

Figure H.12 shows that for low damping the across-wind vibrations in the case of resonance are approximately harmonic. With increasing damping the vibrations get smaller and tend to become irregular, i.e. they develop a more random character (the representations i), ii), and iii) of Figure H.12a correspond to the zones i), ii), and iii) of Figure H.12b).

If for purposes of approximation it is assumed that the structure vibrates harmonically at its fundamental frequency, then the maximum velocity and acceleration due to the dynamic part of the equivalant wind force W is

$$v_{max} = y_{tot}(2\pi \cdot f_e) \tag{H.15}$$

$$a_{max} = y_{tot}(2\pi \cdot f_e)^2 \tag{H.16}$$

where y_{tot} = bending displacement at the top of the structure due to W according to Equation (H.4).

A preventive measure against vortex-induced vibration in the across-wind direction is to increase the critical wind speed, primarily by increasing the fundamental frequency f_e (frequency tuning). Also effective is an *increase of damping* possibly using a *tuned vibration absorber.* In simple cases the *Scruton helical device* can be used which ensures that the vortex shedding lines are no longer vertical lines but spirals, so that the dynamic effect is considerably reduced (cf. Sub-Chapter 3.3).

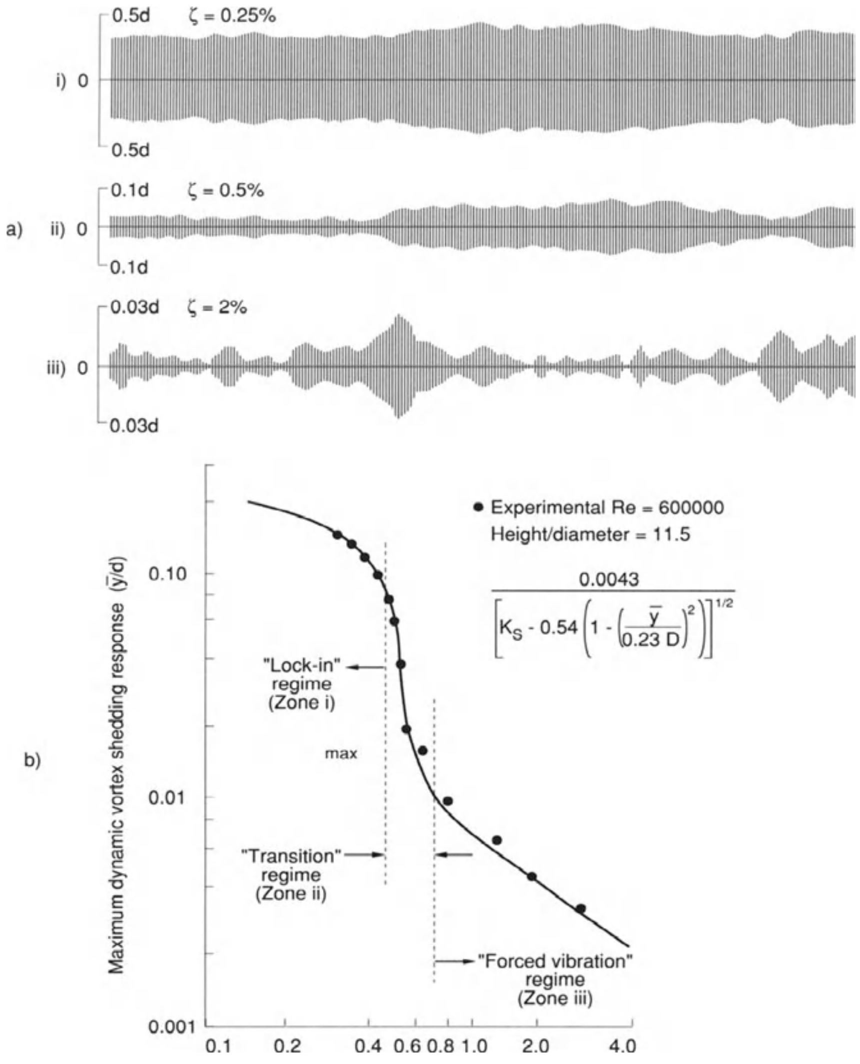


Figure H.12: Across-wind vibration amplitude of circular cylinder as a function of damping (after [H.4])

H.4.2 Several structures one behind another

The vibrational behaviour described above can change if the vortices emanating from one structure impinge on a second or more structures, e.g. in the case of a row of structures (cf. Appendix H.3), and are superimposed on the other vortex shedding processes. The interference effect that is obtained can lead to an increase of the dynamic reactions, but only if the structures exhibit roughly the same dimensions and dynamic characteristics.

In addition, the Strouhal number is also affected. For circular cylinders the following relation applies:

$$S = 0.1 + 0.085 \cdot \log (a/d) \tag{H.17}$$

where a = distance from the obstacle (distance between axes)
d = diameter

As a result, the Strouhal number reduces from 0.20 (for $a/d >$ approx. 15) to 0.14 (for a/d approx. 3). Thus the critical wind speed increases by about 40% and the dynamic effect nearly doubles. Consequently, special attention must be paid to the arrangement of similar structures.

H.4.3 Conical structures

Tapered cantilever structures of a conical form can be treated as cylinders if the cone angle is small. The aerodynamic excitation forces, however, already begin to decrease at relatively small cone angles Section H.6.

For cone angles greater than 1.5° two or three frequency ranges exist where separation of the flow occurs. Several Strouhal numbers can be expected. This also occurs if a cylindrical structure changes dimensions stepwise with height. For changes of diameter less than about 5% it is reasonable to assume a constant value (the mean diameter). Otherwise, depending on the actual diameter, different critical wind speeds would be expected.

H.4.4 Vibrations of shells

Finally, under the theme of vortex-induced vibrations, the question of shell vibrations is discussed briefly. There are some well-known cases where such behaviour has led to the collapse of cooling towers. The shell vibrations occur in the form of "ovalizing" with two or three circulatory waves observed in the plane. Figure H.13 shows how the vortex frequencies f_w of 1/2 and 1/4 of the natural frequency f_e of the shell interact with the two wave forms; the vortex shedding force at a certain shell meridian (e.g. left in wind direction) acts always after two and four natural periods of the shell.

The danger of ovalizing vibrations can be effectively minimized by means of stiffening rings. Stiffening measures have to be employed if

$$d/t \geq \sim 150 \text{ (for steel shells)} \tag{H.18}$$

where d = diameter of the shell
t = thickness of the wall.

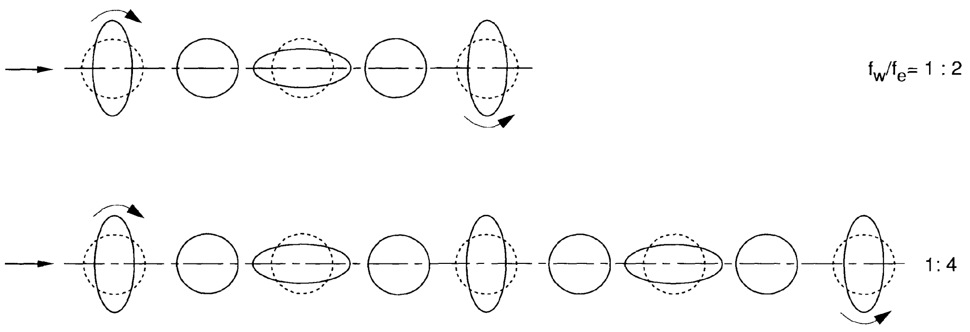


Figure H.13: Ovalizing vibrations for two circulatory waves in plane

H.5 Vibrations in across-wind direction: Galloping

In contrast to the wind-induced vibration discussed above - except for the lock-in effects of vortex resonance - this section deals with self-induced vibrations. A characteristic feature is that the aerodynamic excitation forces depend on the motion of the structure itself. Circular cylinders are not affected by this kind of vibration, but all other sectional shapes are more or less endangered by the so-called galloping phenomenon. Galloping vibrations induced by oblique airflow were first observed on iced-up electrical transmission lines in Canada. Due to ice accretion on the conductors the shape of the cross-section changed to a D-profile, which, for airflow on the flat surface, tends to flutter under a negative lift slope, as is the case with the classical profile of aeroplane wings (aerofoils) at the corresponding angle.

Under certain conditions (profile shape, incidence angle) the so-called aerodynamic damping can be negative, and, where structural damping is small, galloping instability will occur. This gives rise to a strong growth of vibrations in the across-wind direction and can endanger the structure.

According to Den Hartog (from the year 1930) the stability criterion depends on the instability parameter

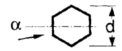
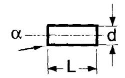
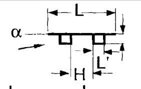
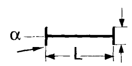
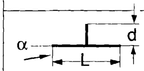
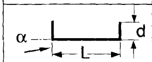
$$dc_L/d\alpha \tag{H.19}$$

where c_L = lift coefficient
α = angle of incidence of airflow
d = differential operator

The following relation gives the critical wind speed that initiates galloping:

$$u_{cr} = 2 \cdot d \cdot f_e \cdot Sc \cdot \frac{1}{dc_L/d\alpha} \tag{H.20}$$

Table H.3 provides values of the instability parameter $dc_L/d\alpha$ for some profiles susceptible to galloping.

Profile	$\frac{L}{d}$	α	$\frac{dc_L}{d\alpha}$
	-	4° - 10°	1.2
	1 2 3	2° - 8° 2° - 6° 2° - 6°	4 11 0.2
 $\frac{L}{L'} = 6.6$; $\frac{L}{H} = 2.2$	10	12° - 16°	1.0
	2.7 5.0	2° - 5° 25° - 27°	9 11
	2	2° - 25°	5.5
	3	0° - 4°	7.5

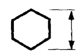


Table H.3: Galloping instability parameters for various cross-section profiles

Galloping instability is strongly influenced by the turbulence of the airflow. Some profiles are very sensitive to this effect. For rectangular profiles with an aspect ratio of 2:1, for example, galloping instability can disappear in turbulent flow, whereas in smoother flow (e.g. in a wind tunnel test) the instability may be present.

Safe predictions of the amplitudes of galloping vibrations are not possible due to the nonlinear aeroelastic behaviour of the system, among other things. In practice, wind tunnel tests on models under conditions of modified turbulent flow are essential in most cases. In natural winds, structures always experience turbulent flow, and this can be of major importance.

An effective countermeasure to galloping is to increase the damping or the Scruton-number, so that the critical speed at which galloping starts can be increased.

If circular cylindrical bodies are connected to one another, then under certain circumstances galloping can occur even though the individual cylinders are stable by themselves. This is because this modified section can exhibit negative aerodynamic damping. The topic is particularly relevant to steel chimney stacks and is thus treated separately in Sub-Chapter 3.3. The wealth of possible variants makes it impossible to give any general formulations.

H.6 Vibrations in across-wind direction: flutter

Just as for aeroplane wings, bridge flutter occurs under combined torsional and bending degrees-of-freedom, whereby the reactions caused by torsional vibrations predominate. Flutter arises when for a particular phase between torsion and bending, vibrational energy is extracted by the structure from the constant flow of air. Figure H.14 shows the motion of a bridge section where the torsional and bending frequencies are equal. The first case shows no phase difference and no energy absorption; in the second case a phase difference is present and some energy absorption is possible.

Bridge flutter is initiated at a certain critical wind speed. A theoretical treatment of the subject with an estimate of the critical flutter speed is, however, not possible. Consequently, extensive wind tunnel tests have been carried out. As is the case for aeroplane wing profiles, stability curves have been established for various bridge profiles [H.6]. Figure H.15 shows some of these curves for the so-called aerodynamic damping A_2^* as a function of the reduced wind speed u_r

$$u_r = \frac{u}{f_T \cdot b} \tag{H.21}$$

where u = wind speed
f_T = fundamental torsional frequency
b = width of the bridge

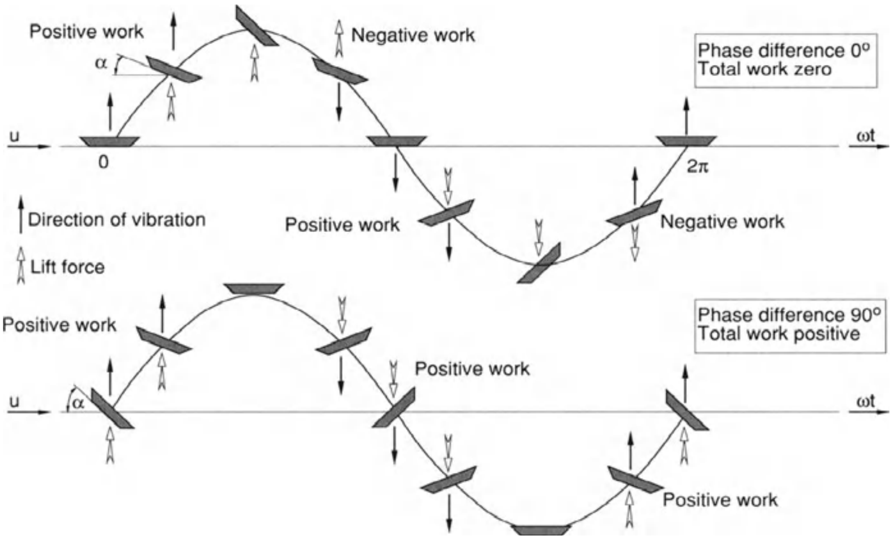


Figure H.14: Work of wind forces inducing flutter in torsional- bending modes

An unstable situation with $u_r > u_{cr}$ results when A_2^* is greater than the level of structural damping D^*. Enhancing the structural damping by ΔD^* is not effective since the main influence comes from the bridge profile. It may be observed that the Tacoma Narrows bridge, which collapsed due to flutter instability in 1940, exhibits a relatively low flutter speed.

From Figure H.15 it is also evident that an increase of the critical flutter speed may be obtained primarily by choosing a section with a more suitable shape. An increase in damping (by

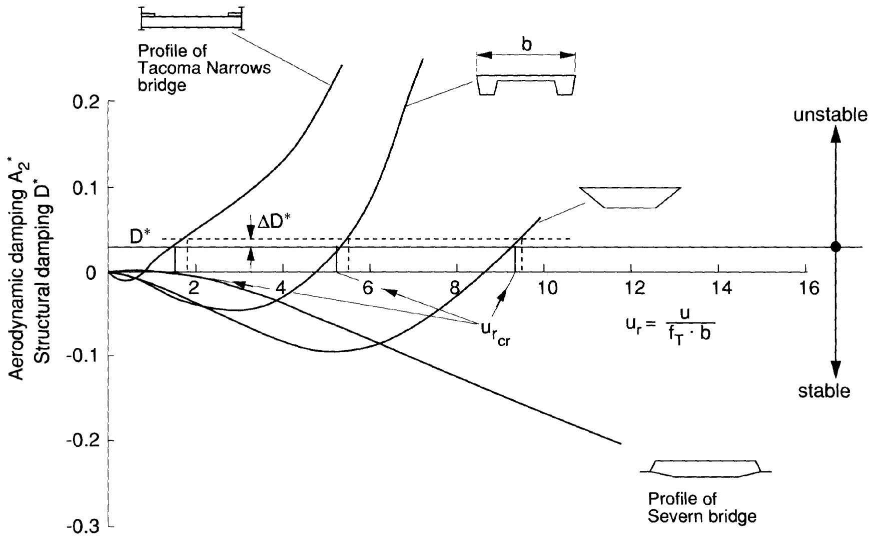


Figure H.15: Stability curves for bridge profiles

ΔD^*, for example) does not achieve the same improvements as for vortex resonance or galloping instability.

Importang for avoiding dangerous bridge flutter is the ratio of torsional to bending natural frequencies (f_T/f_B). This should be as large as possible (about 3). An estimate of the critical flutter speed may be obtained from the following relationship for $f_T/f_B > 1.2$:

$$u_{cr} = \eta \cdot 1 + \left[\left(\frac{f_T}{f_B} - 0.5\right) \cdot \sqrt{\frac{0.72 \cdot m \cdot r}{\pi \cdot \rho \cdot b^2}}\right] \cdot 2 \cdot \pi \cdot f_B \cdot b \qquad \text{(H.22)}$$

where
- η = shape factor for the bridge profile according to Table H.4 obtained from wind-tunnel tests [H.5]
- f_B = fundamental frequency for bending vibration in across-wind direction
- f_T = fundamental frequency for torsional vibration around the longitudinal axis of bridge
- m = mass/length of bridge
- r = radius of gyration
- b = effective width of bridge along wind direction
- ρ = air density (1.2 kg/m^3)

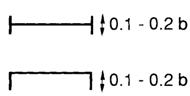
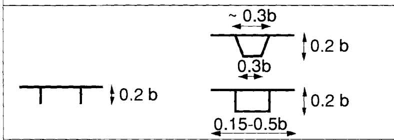
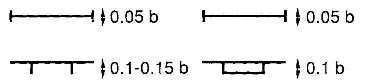
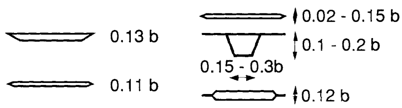
Cross-section	η
 0.1 - 0.2 b; 0.1 - 0.2 b	0.2
 0.2 b; ~ 0.3b; 0.2 b; 0.3b; 0.2 b; 0.15-0.5b	0.3
 0.05 b; 0.05 b; 0.1-0.15 b; 0.1 b	0.5
 0.13 b; 0.11 b; 0.02 - 0.15 b; 0.1 - 0.2 b; 0.15 - 0.3b; 0.12 b	0.7
 b; b	1

Table H.4: Shape factors for bridge profiles

H.7 Damping of high and slender RC structures subjected to wind

I. Floegl, H. Bachmann

The damping quantities of reinforced concrete (RC) structures subjected to wind can be estimated as follows:

The overall equivalent viscous damping ratio is

$$\zeta = \zeta_1 + \zeta_2 + \zeta_3 \tag{H.23}$$

where ζ_1 = material damping
ζ_2 = structural damping
ζ_3 = foundation damping

Relevant viscous damping ratios and logarithmic decrements, respectively, are given in Table H.5.

		ζ (damping ratio)	Λ (logarithmic decrement)
	Material:		
ζ_1	Reinforced concrete		
	• uncracked or prestressed	0.0040	0.025
	• cracked	0.0072	0.045
	Structure:		
ζ_2	Shell and box-type construcion		
	• without stiffening	0.0032	0.020
	• with stiffening	0.0056	0.035
	Frame construction		
	• without non-structural elements	0.0040	0.025
	• with non-structural elements	0.0064	0.040
	Chimneys or tower blocks		
	• without non-structural elements	0.0016	0.010
	• with non-structural elements	0.0024	0.015
	Foundation:		
ζ_3	Support on hinge or roller	0.0008	0.005
	Sliding elastomeric bearings	0.0024	0.015
	Restraint frame-construction	0.0016	0.010
	Fixed cantilever construction:		
	• on steel support	0.0016	0.010
	• on concrete support	0.0008	0.000
	• on foundations:		
	· rock	0.0008	0.005
	· gravel	0.0013	0.008
	· sand	0.0016	0.010
	· piles	0.0024	0.015

Table H.5: Common values of contributions to damping of concrete structures subjected to wind excitation ($\Lambda \approx 2\pi\zeta$)

As an example, the overall damping ratio of a reinforced concrete chimney (with a circular section of 8 m diameter and a height of 120 m) of cantilever construction with a concrete slab foundation on gravel and with 4 internal masonry flues is as follows:

$$\zeta = 0.0040 + 0.0024 + 0.0013 \approx 0.008 \tag{H.24}$$

Note that the subdivision of the overall damping quantity is different from that described in Appendix C.5. Comparing values of parts of the overall damping of the two concepts is thus hardly possible. Comparison of overall damping quantities resulting from Table H.5 and those given in the Sub-Chapters 3.1 to 3.7 may also exhibit some differences.

I Human response to vibrations

A.J. Pretlove, J.H. Rainer

I.1 Introduction

Human sensitivity to vibration is very acute. The human body can sense vibration displacement amplitudes as low as 0.001 mm whilst finger-tips are 20 times more sensitive than this. However, reaction to vibration depends very much on circumstances. For example, discomfort is different sitting at an office desk from driving a car. Personal attitude is also important. Sensitivity will depend on personal dedication to a task and to acclimatisation. Parameters which affect *human sensitivity* are as follows:

- position (standing, sitting, lying down)
- direction of incidence with respect to the spine
- personal activity (resting, walking, running)
- sharing of the experience with others
- age and sex
- frequency of occurrence and time of day
- the character of vibration decay.

The *intensity of perception* will depend upon the following factors:

- displacement, velocity and acceleration amplitudes
- duration of exposure
- vibration frequency.

The intensity of perception has been researched by many authors and their results are generally in broad agreement with the data in Table I.1. Broadly speaking, in the range 1 to 10 Hz perceptibility is proportional to acceleration, whilst in the range 10 to 100 Hz perceptibility is proportional to velocity. This division varies somewhat with the level of the stimulus but in this basic presentation this factor will be ignored.

I.2 Codes of practice

In order to set realistic criteria, codes of practice also take into account some of the sensitivity factors listed above. Two widely used codes which illustrate the general features described above are [ISO 2631] and [DIN 4150]. These are discussed in the paragraphs which follow.

Description	Frequency range 1 to 10 Hz Peak acceleration [mm/s^2]	Frequency range 10 to 100 Hz Peak velocity [mm/s]
just perceptible	34	0.5
clearly perceptible	100	1.3
disturbing/unpleasant	550	6.8
intolerable	1800	13.8

Table I.1: An indication of human perceptibility thresholds for vertical harmonic vibrations (person standing). Data combined from various authorities. There is scatter by a factor of up to about 2 on the values given

I.2.1 ISO 2631

The International Standard [ISO 2631] applies to vibrations in both vertical and horizontal directions and it deals with random and shock vibration as well as harmonic vibration. The frequency range covered is 1 to 80 Hz and criteria are expressed solely in relation to measured effective accelerations (rms):

$$a_{eff} = \sqrt{\frac{1}{T}\int_{o}^{T} a^2(t) \cdot dt} \tag{I.1}$$

T is the period of time over which the effective acceleration is measured.

Note that for pure sinusoidal vibrations the rms value represents 0.707 times the peak value. In [ISO 2631/1] three different levels of human discomfort are distinguished:

- The "reduced comfort boundary", which applies to the threshold at which activities such as eating, reading or writing are disturbed.
- The "fatigue-decreased proficiency boundary", which applies to the level at which recurrent vibrations cause fatigue to working personnel with consequent reduction in efficiency. This occurs at about three times the reduced comfort boundary.
- The "exposure limit", which defines the maximum tolerable vibration with respect to health and safety and is set at about six times the reduced comfort boundary.

The basic criteria are given in graphical form for both longitudinal vibration (vertical if the subject is standing) and transverse vibration (horizontal if the subject is standing). By way of example, Figure I.1 shows the set of graphical criteria for the longitudinal vibration case. Different criteria apply for different exposure times as indicated on the Figure. This form of presentation clearly indicates the adverse effects of body resonances upon acceptability, chiefly resonances within the torso but also those within the cranium. Figure I.2 shows the corresponding set of graphical criteria for transverse vibrations.

For acceptable vibration criteria in buildings [ISO 2631/2] gives base curves to which various suggested multipliers are applied depending on location, duration and time of day.

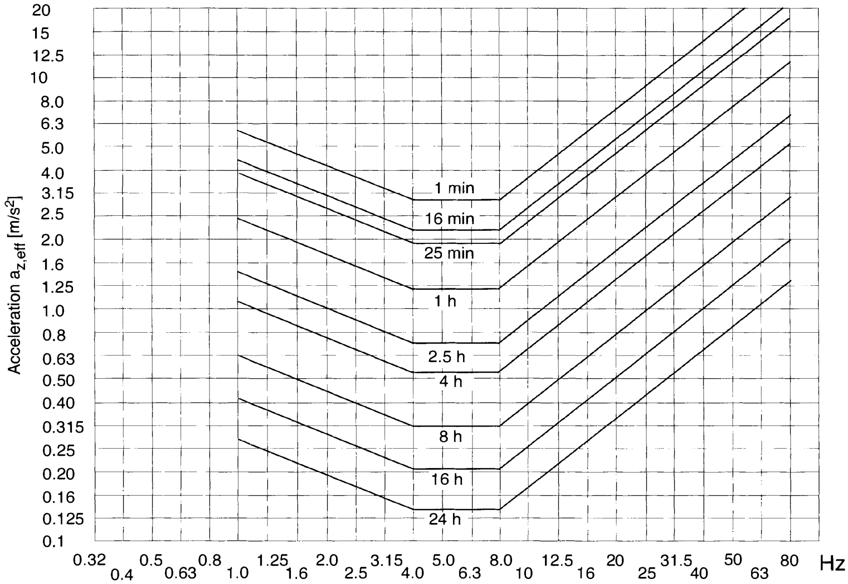


Figure I.1: Bounds on longitudinal vibration for fatigue-decreased proficiency. The exposure limit is obtained by multiplying by 2; the reduced-comfort boundary by dividing by 3.15 [ISO 2631/1]

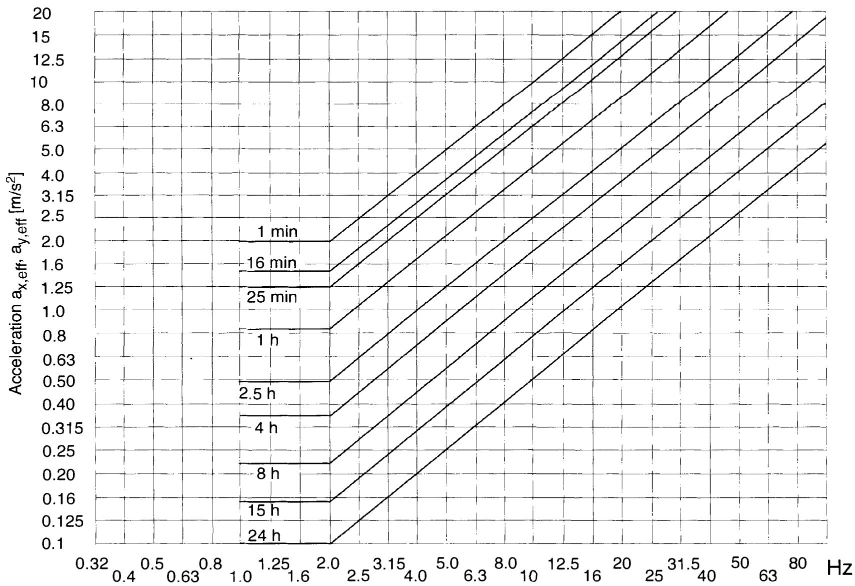


Figure I.2: Bounds on transverse vibration for fatigue-decreased proficiency. Factors for other boundaries as given in the caption of Figure I.1 [ISO 2631/1]

I.2.2 DIN 4150/2

The German Standard [DIN 4150/2] deals largely with the effects of externally sourced vibrations on people in residential buildings. The frequency range considered is 1 to 80 Hz and the change from acceleration to velocity sensitivity occurs at 8 Hz. The measured value of (principal harmonic) vibration together with the frequency is used to calculate a derived intensity of perception factor KB using the formula

$$KB = d \cdot \frac{0.8 \cdot f^{2}}{\sqrt{1 + 0.032 \cdot f^{2}}} \tag{I.2}$$

where d = displacement amplitude [mm]
f = principal vibration frequency [Hz]

or an equivalent equation derived from measured velocity or acceleration values The calculated KB-value has the dimension of a velocity [mm/s]. It is then compared with an acceptable reference value, as shown in Table I.2 below, according to:

- use of the building
- frequency of occurrence
- duration of the vibration
- time of day

In making these comparisons of derived KB-values with the criteria of acceptability the standard provides useful graphs, derived from the equation above. These permit the derivation of KB-values for given measured vibration values without the need for calculation.

Building Type	Time	Acceptable KB value	
		continuous or repeated	infrequent
rural, residential and holiday resort	day night	0.2 (0.15*) 0.15 (0.1*)	4.0 0.15
small town and mixed residential	day night	0.3 (0.2*) 0.2	8.0 0.2
small business and office premises	day night	0.4 0.3	12.0 0.3
industrial	day night	0.6 0.4	12.0 0.4
* These values should be complied with if buildings are excited horizontally at frequencies below 5 Hz			

Table I.2: Acceptable KB intensities for residential buildings (abstracted from [DIN 4150/2], 1975)

J Building response to vibrations

J.H. Rainer, G. Klein

J.1 General

Serviceability limit states for building structures are those of hairline or minor cracking, spalling of paint or plaster, excessive deflection or accelerated aging.

The *recommended values* of particle velocity (or sometimes acceleration or displacement) have been obtained by experience and are therefore of an empirical nature. They depend greatly on the type of structure, type of soil and many other parameters whose influence cannot be quantified at present. The values recommended also depend on the type of excitation and the frequency content and duration. For this reason the limit values for blasting differ substantially from those for traffic. It is therefore also not surprising that the tolerable values vary greatly from country to country and from structure to structure, and no single set of criteria seems to satisfy all requirements. It should be noted that the recommended criteria do not guarantee absence of damage, but reduce its probability of occurrence to acceptably low levels (see [J.7], [J.2]).

The following are examples of available criteria and standards that are used in some countries; this is, however, not an exhaustive or exclusive list of existing requirements.

The *measurement techniques* that are associated with these recommendations can vary. Some use the vectorial sum v_i of the instantaneous values

$$v_i = \sqrt{v_x^2 + v_y^2 + v_z^2} \tag{J.1}$$

others use the maximum value v_{max} in the direction normal to a wall or in a particular designated direction. Some standards refer to the recommended values at the foundation, others to the ground near the building. The relevant governing quantities will be given with the following examples.

J.2 Examples of recommended limit values

Building Class	Frequency range where the standard value is applicable [Hz]	Maximum resultant velocity, v_i [mm/s]	Estimated maximum vertical particle velocity, v_{ma} [mm/s]
1. Industrial buildings of reinforced concrete, steel construction	10 - 30 30 - 60	12 12 - 18	7.2 - 12 7.2 - 18
2. Buildings on concrete foundation. Concrete walls or brick walls	10 - 30 30 - 60	8 8 - 12	4.8 - 8 4.8 - 12
3. Buildings with brick cellar walls. Upper apartment floors on wooden beams	10 - 30 30 - 60	5 5 - 8	3 - 5 3 - 8
4. Especally sensitive buildings and historical buildings	10 - 30 30 - 60	3 3 - 5	1.8 - 3 1.8 - 5

Table J.1: Standard values for piling, sheet piling, vibratory compaction and traffic [J.5]

Maximum vertical particle velocity v_{max} [mm/s]	Effect on buildings
2	• Risk of damage to ruins and buildings of great historical value
5	• Risk of cracking in normal residential buildings with plastered walls and ceilings
10	• Risk of damage to normal residential buldings (no plastered walls and ceilings)
10 - 40	• Risk of damage to concrete buildings, industrial premises, etc.

Table J.2: Recommended values for vibratory compactor [J.3]

Type of building and foundation	Recommended vertical velocity v_{max} [mm/s]
• Especially sensitive buildings and buildings of cultural and historical value	1
• Newly-built buildings and/or foundations of a foot plate (spread footings)	2
• Buildings on cohesion piles	3
• Buildings on bearing piles or friction piles	5

Table J.3: Recommended limit values for traffic [J.1]

Maximum particle velocity [mm/s]			Effects
Sand, Gravel, Clay	Moraine, Slate-stone, Lime-stone	Granite, Gneis, Sandstone	
18	35	70	• No noticeable cracking
30	55	110	• Fine cracks and fall of plaster (threshold value)
40	80	160	• Cracking
60	115	230	• Serious cracking

Table J.4: Risk of damage in ordinary dwelling houses with varying ground conditions [J.4]

Type of structure	Ground vibration - peak particle velocity, v_{max} [mm/s] ([in/s])	
	At low frequency* < 40 Hz	At high frequency > 40 Hz
• Modern homes, drywall interiors	19 (0.75)	51 (2.0)
• Old homes, plaster on wood, lath construction for interior walls	13 (0.5)	51 (2.0)
* All spectral peaks within 6 dB (50%) amplitude of the predominant frequency must be analyzed.		

Table J.5: Safe levels of blasting vibrations for residential type structures [J.6]

Type of structure	Vibration velocity v_i [mm/s]			
	At foundation			At plane of floor of uppermost full storey (all frequencies)
	< 10 Hz	10 - 50 Hz	50 - 100 Hz*	
1. Buildings used for commercial purposes, industrial buildings and buildings of similar design	20	20 - 40	40 - 50	40
2. Dwellings and buildings of similar design and/or use	5	5 - 15	15 - 20	15
3. Structures that, because of their particular sensitivity to vibration, do not correspond to those listed in lines 1 and 2 and are of great intrinsic value (e.g. buildings that are under preservation order)	3	3 - 8	8 - 10	8
* For frequencies above 100 Hz, at least the values specified in this column shall be applied				

Table J.6: Guideline values of vibration velocity for evaluating the effects of short-term vibration [DIN 4150/3]

References to the Appendices

[A.1] Thomson W.T: "Theory of Vibration". Third edition. Prentice-Hall International Inc., Englewood Cliffs, New Jersey, 1988.

[A.2] Clough R.W. and Penzien J.: "Dynamics of Structures". McGraw-Hill, New York, 1975.

[C.1] Irvine H.M.: "Structural Dynamics for the Practising Engineer". Allen and Unwin, London, Boston, Sidney, 1986.

[C.2] Dieterle R., Bachmann H.: "Experiments and Models for the Damping Behaviour of Vibrating Reinforced Concrete Beams in the Uncracked and Cracked Condition". Institut für Baustatik und Konstruktion, ETH Zürich, Bericht Nr. 119, Birkhäuser Verlag Basel, 1981.

[D.1] Den Hartog J.P.: "Mechanical Vibrations". Fourth edition. McGraw- Hill, New York, 1956.

[D.2] Bachmann H., Ammann W.: "Vibrations in Structures - Induced by Man and Machines". Structural Engineering Documents No. 3e. International Association for Bridge and Structural Engineering (IABSE), Zürich, 1987.

[D.3] Gerasch W.J., Natke H.G.: "Vibration Reduction of Two Structures". International Symposium on Vibration Protection in Construction. Scientific report, volume 1, pp. 132-142, Leningrad (USSR), 1984.

[D.4] Matsumoto Y., Nishioka T., Shiojiri H., Matsuzaki K.: "Dynamic Design of Footbridges". International Association of Bridge and Structural Engineering (IABSE). Proceedings P-17/78, Zürich, 1978.

[D.5] Jones R.T., Pretlove A.J.: "Vibration Absorbers and Bridges". Journal of the Institution of Highway Engineers, pp. 2-8, 1979.

[D.6] Hunt J.B.: "Dynamic Vibration Absorbers". Mechanical Engineering Publications Ltd., London, 1979.

[D.7] Inman D.J.: "Vibrations, Control, Measurement and Stability". Prentice-Hall International Inc., Englewood Cliffs, New Jersey, 1989.

[D.8] Bachmann H., Weber B.: "Tuned Vibration Absorbers for damping of 'lively' structures". Submitted to appear in Structural Engineering International, 1995.

[E.1] Prange B.: "Dynamic Response and Wave Propagation in Soils". Proceedings, Dynamical Methods in Soil and Rock Mechanics, Karlsruhe, 5-16 September, 1977. A.A. Balkema, Rotterdam, 1978.

[E.2] Ewing W.M., Jardetzky W.S., Press F.: "Elastic Waves in Layered Media". McGraw-Hill, New York, 1957.

[E.3] Chouw N., Le R., Schmid G.: "Ausbreitung von Erschütterungen in homogenem Boden; Numerische Untersuchungen mit der Randelementmethode im Frequenzbereich" ("Spreading of Vibrations in Homogeneous Ground; Numerical Investigation by the Boundary Element Method in the Frequency Domain"). Bauingenieur 65, Heft 9, 1990.

[F.1] Comité Euro-International du Béton (CEB): "Concrete Structures Under Impact and Impulsive Loading". Bulletin d'Information No. 187, 1988.

[F.2] Vos E., Reinhardt H.W.: "Influence of Loading Rate on Bond in Reinforced Concrete". Proceedings RILEM-CEB-IABSE-IASS- Interassociation Symposium on Concrete Structures under Impact and Impulsive Loading, pp. 170-181. Bundes-anstalt für Materialprüfung (BAM), Berlin, 1982.

[F.3] Ammann W., Mühlematter M., Bachmann H.: "Versuche an Stahlbeton- und Spannbetontragwerken unter stossartiger Beanspruchung, Teil 1: Zugversuche an Bewehrungs- und Spannstahl mit erhöhter Dehngeschwindigkeit" ("Experiments with Structures of Reinforced and Prestressed Concrete Under Impact Loading, Part 1: Tension Tests on Reinforcing and Prestressing Steel under Elevated Loading Rates"). Institut für Baustatik und Konstruktion, ETH Zürich, Versuchsbericht Nr. 7709-1, Birkhäuser Verlag Basel, 1982.

[F.4] Berner K.: "Der Einfluss der Dehngeschwindigkeit auf das mechanische Ver-halten von Betonstählen" ("Influence of Loading Rate on the Mechanical Behaviour of Reinforcing Steel for Concrete"). 12. Forschungskolloquium des Deutschen Ausschusses für Stahlbeton, Seiten 181-195, Bundesamt für Materialprüfung (BAM), Berlin, 1981.

[G.1] Rainer J.H., Pernica G.: "Vertical Dynamic Forces from Footsteps". Canadian Acoustics, Volume 14, No. 2, 12-21, 1986.

[G.2] Tilly G.P., Cullington D.W., Eyre R.: "Dynamic Behaviour of Footbridges". International Association for Bridge and Structural Engineering (IABSE), Surveys S-26/84, Zürich, 1984.

[G.3] Bachmann H., Ammann W.: "Vibrations in Structures - Induced by Man and Machines". Structural Engineering Documents No. 3e, International Association of Bridge and Structural Engineering (IABSE), Zürich, 1987.

[G.4] Baumann K., Bachmann H.: "Durch Menschen verursachte dynamische Lasten und deren Auswirkungen auf Balkentragwerke" (Man-induced Dynamic Forces and the Response of Beam Structures). Institut für Baustatik und Konstruktion, ETH, Zürich, Versuchsbericht Nr.7501- 3, Birkhäuser Verlag Basel, 1988.

[G.5] Vogt R., Bachmann H.: "Dynamische Lasten durch rhythmisches Klatschen, Fussstampfen und Wippen" (Dynamic Loads from Rhythmic Hand Clapping, Footstamping and Moving Up and Down). Institut für Baustatik und Konstruktion, ETH, Zürich, Versuchsbericht Nr.7501- 4, Birkhäuser Verlag Basel, 1987.

[H.1] Kolousek V., Pirner M., Fischer O., Naprstek J.: "Wind Effects on Civil Engineering Structures". Academia, Praha, 1983.

[H.2] Blevins R.D.: "Flow-Induced Vibration". Van Nostrand Reinhold, New York, 1990.

[H.3] Harris C.: "Shock and Vibration Handbook", 29 Part II: Vibrations of Structures Induced by Wind. McGraw-Hill, New York, 1989.

[H.4] Harris R.I.: "Wind Engineering", 6 ICWE, Australia, 1983, Journal of Wind Engineering and Industrial Aerodynamics, Vol. 13-15, Elsevier, Amsterdam, 1983.

[H.5] Zerna W.: "Gebäudeaerodynamik" ("Aerodynamics of Buildings"). Heft 35/36 Konstruktiver Ingenieurbau, Ruhr-Universität, Bochum, Vulkan-Verlag Dr. W. Classen Nachf., Essen, 1981.

[H.6] Ruscheweyh H.: "Dynamische Windwirkung an Bauwerken" ("Dynamic Actions of Wind on Structures"). Band I und II, Bauverlag, Wiesbaden und Berlin, 1982.

[H.7] Davenport A.G.: "The Application of Statistical Concepts to the Wind Loading of Structures". Proceedings of the Institution of Civil Engineers, 1961.

[H.8] Simiu E., Scanlan R.H.: "Wind Effects on Structures". John Wiley & Sons, New York, 1986.

[H.9] Cook, N.J.: "The Designer's Guide to Wind Loading of Building Structures". Part 1, Butterworths, London, 1985.

[H.10] Sockel H.: "Damping Measures to Control Wind Induced Vibrations". CISM Courses and Lectures No. 335. ISBN 3-211-82516-9. Springer-Verlag Wien, New York, 1994.

[J.1] Bonde G., Rundquist G. et al.: "Criteria for Acceptable Traffic- Induced Vibrations". Institute of Technology, Uppsala University, UPTEC 81 42 R, TRAVI-K (in Swedish), 1981. See also Holmberg et al., 1984.

[J.2] Holmberg R., et al.: "Vibrations Generated by Traffic and Building Construction Activities". Swedish Council for Building Research, Stockholm, 1984.

[J.3] Forssblad I.: "Vibratory Soil and Rock Fill Compaction". Dynapac Maskin AB, Solna, Stockholm, 1981.

[J.4] Langefors U., Kihlstrom B.: "The Modern Technique of Rock Blasting". Third edition, Almqvist & Wiksells Boktryckeri AB, Uppsala, Sweden, 1978.

[J.5] Studer J., Suesstrunk A.: "Swiss Standard for Vibration Damage to Buildings". Proceedings of the Tenth International Conference on Soil Mechanics and Foundation Engineering, Volume 3, pp. 307-312, Stockholm, 1981.

[J.6] Siskind D.E., Stagg M.S., Kopp J.W., Dowding C.H.: "Structure Response and Damage Produced by Ground Vibration from Surface Mine Blasting". U.S. Bureau of Mines, Report of Investigation RI 8507, 1980. U.S. Government Printing Office, Washington DC, 1980.

[J.7] Dowding C.H.,: "Blast Vibration Monitoring and Control". Page 297. Prentice-Hall International Inc., Englewood Cliffs, New Jersey,1985.

List of Codes and Standards

[BRE] "Vibrations: Building and Human Response". Building Research Establishment (UK), BRE Digest 278, 1983.

[BS 5400] "Steel, Concrete and Composite Bridges: Specification for Loads". British Standard BS 5400, Part 2, Appendix C, 1978.

[BS 6472] "Guide to Evaluation of Human Exposure to Vibration in Buildings (1 Hz to 80 Hz)". British Standard BS 6472, 1984.

[BS CP 2012/1] "Code of Practice for Foundations for Machinery: Foundations for Reciprocating Machines". British Standard Code of Practice BS CP 2012, Part 1, 1974.

[CSA 84] "Steel Structures for Buildings". Standard CAN3-S16.1- M84, Appendix G, Canadian Standards Association, Rexdale, Ontario, 1984.

[CICIND] "Model Code for Steel Chimneys". Comité International des Cheminées Industriel (CICIND), Beckenham, England.

[DIN 1055/4] "Design Loads for Buildings; Imposed Loads; Windloads on Structures Unsusceptible to Vibration" (available also in German). Norm DIN 1055 Blatt 4, Deutsches Institut für Normung, Beuth Verlag GmbH, Berlin, 1986; Aenderung 1 1987.

[DIN 4024/1] "Maschinenfundamente; Elastische Stützkonstruktionen für Maschinen mit rotierenden Massen" ("Machine Foundations; Elastic Supporting Construction for Machines with Rotating Masses"). Norm DIN 4024 Teil 1, Deutsches Institut für Normung, Beuth Verlag GmbH, Berlin, 1988.

[DIN 4024/2] "Maschinenfundamente; Steife (starre) Stützkonstruktionen für Maschinen mit periodischer Erregung" ("Machine Foundations; Rigid Supporting Constructions for Machines with Periodic Excitation"). Entwurf Norm DIN 4024 Teil 2, Deutsches Institut für Normung, Beuth Verlag GmbH, Berlin, 1988.

[DIN 4133] "Schornsteine aus Stahl; Statische-Berechnung und Ausführung" ("Steel Stacks; Structural Analysis and Design"). Norm DIN 4133, Deutsches Institut für Normung, Beuth Verlag GmbH, Berlin, 1973.

[DIN 4150/1] "Erschütterungen im Bauwesen; Vorermittlung und Messung von Schwingungs–grössen" ("Vibrations in Building; Principles, Predetermination and Measurement of the Amplitude of Oscillations"). Vornorm DIN 4150 Teil 1, Deutsches Institut für Normung, Beuth Verlag GmbH, Berlin, 1975.

[DIN 4150/2] "Erschütterungen im Bauwesen; Einwirkungen auf Menschen in Gebäuden" ("Vibrations in Building; Influence on Persons in Buildings"). Vornorm DIN 4150 Teil 2 bzw. Entwurf Norm DIN 4150 Teil 2, Deutsches Institut für Normung, Beuth Verlag GmbH, Berlin, 1975 bzw. 1990.

[DIN 4150/3] "Structural Vibration in Buildings; Effects on Structures" (available also in German). Norm DIN 4150 Teil 3, Deutsches Institut für Normung, Beuth Verlag GmbH, Berlin, 1986.

[DIN 4178] "Glockentürme; Berechnung und Ausführung" ("Belltowers: Calculation and Constructional Design"). Norm DIN 4178, Deutsches Institut für Normung, Beuth Verlag GmbH, Berlin, 1978.

[DIN 45630/1] "Grundlagen der Schallmessung; Physikalische und subjektive Grössen von Schall" ("Bases for Measurement of Sound; Physical and Subjective Measures of Sound"). Norm DIN 45630, Blatt 1, Deutsches Institut für Normung, Beuth Verlag GmbH, Berlin, 1971.

[DIN IEC 651] "Schallpegelmesser" ("Sound Level Meters"). Norm DIN IEC 651, Deutsches Institut für Normung, International Electrotechnical Commission, Beuth Verlag GmbH, Berlin, 1981.

[EC 1:2-4] "Basis of design and actions on structures - Wind actions". EUROCODE 1, Part 2-4, European Prestandard ENV 1991-2-4, 1994.

[ISO 131] "Acoustics - Expression of Physical and Subjective Magnitudes of Sound or Noise in the Air". ISO 131 International Standard Organisation, Geneva 1979.

[ISO 2372] "Mechanical Vibration of Machines with Operating Speeds from 10 to 200 rev/s - Basis for Specifying Evaluation Standards". ISO 2372, International Standards Oranisation, Geneva, 1974, Amendement 1, 1983.

[ISO 2373] "Mechanical Vibration of certain Rotating Electrical Machinery with Shaft Heights between 80 and 400 mm - Measurement and Evaluation of the Vibration Severity". ISO 2373, International Standard Organisation, Geneva, 1987.

[ISO 2631/1] "Evaluation of Human Exposure to Whole-body Vibration: General Requirements". ISO 2631, Part 1, International Standards Organisation, Geneva, 1985.

[ISO 2631/2] "Evaluation of Human Exposure to Whole-body Vibration: Continuous and Shock-induced Vibration in Buildings (1 to 80 Hz)". ISO 2631, Part 2, International Standards Organisation, Geneva, 1989.

[ISO 3945] "Mechanical Vibration of large Rotating Machines with Speed ranging from 10 to 200 r/s - Measurements and Evaluation of Vibration Severity in Situ". ISO 3945, International Standards Organisation, Geneva, 1985.

[ISO/DIS 4354] "Wind actions on structures". Draft ISO/DIS 4354. International Standard Organisation, Geneva, 1991.

[ISO/DIS 4866] "Mechanical Vibration and Shock - Measurement and Evaluation of Vibration Effects on Buildings - Guidelines for the use of Basic Standard Methods". Draft ISO/DIS 4866. International Standard Organisation, Geneva, 1986.

[ISO/DIS 10137] "Bases for Design of Structures - Serviceability of Buildings against Vibration". Draft ISO/DIS 10137, International Standards Organisation, Geneva, 1991.

[ISO/R 357] "Expression of the Power and Intensity Levels of Sound or Noise". Recommendation ISO/R 357, International Standard Organisation, Geneva, 1963.

[ISO/TC98/SC3/WG2] "Wind Loading (Static and Dynamic)". Draft ISO/TC98/SC3/WG2. International Standard Organisation, Geneva, 1991.

[NBCC 90] "Serviceability Criteria for Deflections and Vibrations". Commentary A. Supplement to the National Building Code of Canada. National Research Council Canada, Ottawa, 1990.

[ONT 83] Ontario Highway Bridge Design Code. Ontario Ministry of Transportation, Toronto, 1983.

[SIA 160] "Actions on Structures". Code SIA 160 (available also in German). Schweizer Ingenieur- und Architekten-Verein (SIA), Zürich, 1989.

[SNV 640312] "Erschütterungen im Bauwesen" ("Vibrations in Construction Work"). SNV 640 312, Schweizerische Normenvereinigung, Zürich, 1978.

[VDI 2056] "Beurteilungsmassstäbe für mechanische Schwingungen von Maschinen" ("Effects of Mechanical Vibrations on Machines"). Richtlinie VDI 2056, Verein Deutscher Ingenieure, Beuth Verlag GmbH, Berlin, 1964.

[VDI 2057/1] “Einwirkung mechanischer Schwingungen auf den Menschen; Grundlagen; Gliederung; Begriffe” (“Effects of Mechanical Vibrations on People: Basics, Structure, Definitions”). Entwurf Richtlinie VDI 2057, Blatt 1, Verein Deutscher Ingenieure, Beuth Verlag GmbH, Berlin, 1987.

[VDI 2057/2] “Einwirkung mechanischer Schwingungen auf den Menschen; Bewertung” (“Effects of Mechanical Vibrations on People: Assessment”). Entwurf Richtlinie VDI 2057, Blatt 2, Verein Deutscher Ingenieure, Beuth Verlag GmbH, Berlin, 1987.

[VDI 2057/3] “Einwirkung mechanischer Schwingungen auf den Menschen; Beurteilung” (“Effects of Mechanical Vibrations on People: Evaluation”). Entwurf Richtlinie VDI 2057, Blatt 3, Verein Deutscher Ingenieure, Beuth Verlag GmbH, Berlin, 1987.

[VDI 2058/1] “Beurteilung von Arbeitslärm in der Nachbarschaft” (“Evaluation of Industrial Noise in the Environment”). Richtlinie VDI 2058, Blatt 1, Verein Deutscher Ingenieure, Beuth Verlag GmbH, Berlin, 1985.

[VDI 2062/1] “Schwingungsisolierung: Begriffe und Methoden" ("Vibration Isolation: Concepts and Methods”). Richtlinie VDI 2062, Blatt 1, Verein Deutscher Ingenieure, Beuth Verlag GmbH, Berlin, 1976.

[VDI 2062/2] “Schwingungsisolierung: Isolierelemente” (“Vibration Isolation: Isolation Elements”). Richtlinie VDI 2062, Blatt 2, Verein Deutscher Ingenieure, Beuth Verlag GmbH, Berlin, 1976.

[VDI 2063] “Measurement and Evaluation of Mechanical Vibrations of Reciprocating Piston Engines and Piston Compressors” (available also in German). Richtlinie VDI 2063, Verein Deutscher Ingenieure, Beuth Verlag GmbH, Berlin, 1985.

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