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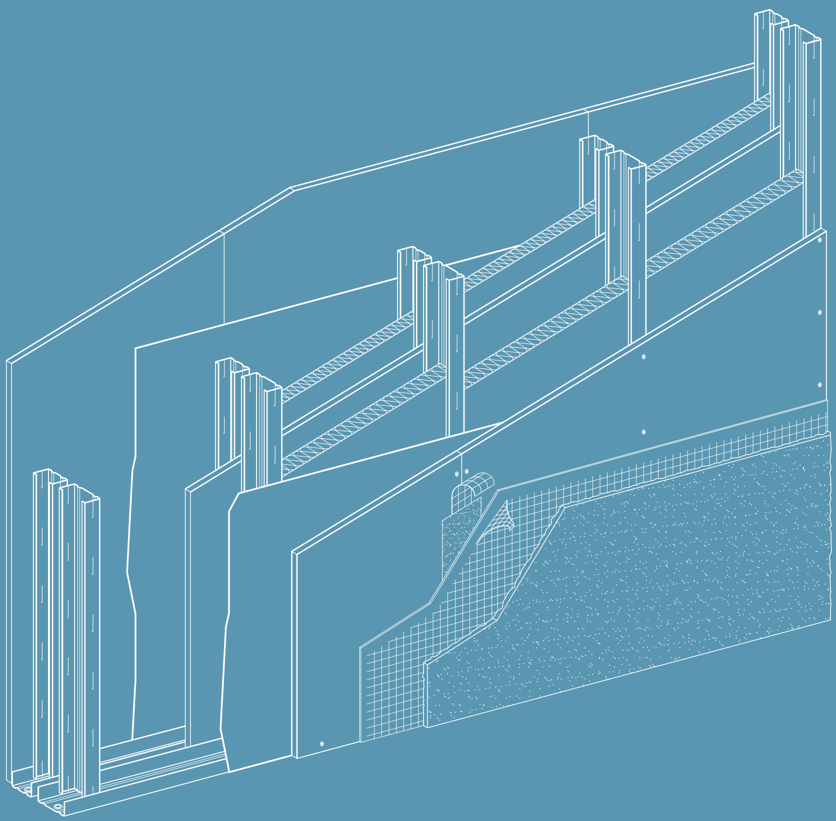

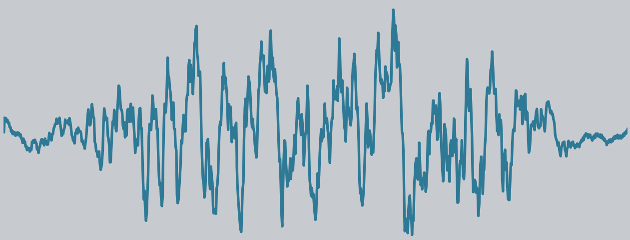

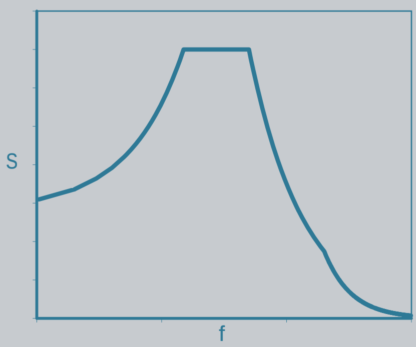

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Foreword

World population is continuously growing. Consequently, the settlement density on our blue planet is increasing inevitably, more and more leading to settlement in zones with increased seismic hazard. Thus and in view of the more sophisticated infrastructure of buildings and cities the seismic risk (loss of human lives and assets per year) is steadily increasing.

The risk is characterized by three components (cf. Chapter 1.3): the natural geologic processes in the earth's crust, the reaction of the structures on such processes and the disaster management measures after an earthquake.

We humans cannot influence any geologic process at all, but can only take action with regard to the last named aspects, with special importance being attached to the reaction of the structures: An appropriate design involves a favorable structural behavior so that the risk caused by earthquake is reduced significantly. Earthquake resistant construction is necessary now more than ever.

Earthquake resistant construction is less a question of cost, but rather of thorough planning, with observation of essential constructive rules. In addition, it is essential to utilise building materials with favorable behavior characteristics under earthquake loads.

The standard books and publications are mainly concerned with classical structures made of reinforced concrete, masonry and steel. It is essential to withstand the forces that arise in a swaying building by means of appropriate design, or - according to contemporary thought - to dissipate the induced seismic energy sufficiently by means of ductility (plastification capability) specifically provoked. At the same time the deformations must be limited as to meet the protection objectives planned.

The earthquake induced forces are inertial forces and thus are proportional to masses. Consequently, this does also apply to the seismic energy which is to be dissipated. As a result, lightweight structures a priori enjoy a fundamental advantage compared to the heavy solid constructions and become increasingly significant.

This aspect is taken into account by the present book, which covers the spectrum of those capable lightweight structures that are preferably realized as drywall construction, showing a possibility for the erection of earthquake resistant buildings or for the back-fitting of existing buildings against earthquake.

Following an introduction in chapter 1, chapter 2 first describes some fundamental facts as to seismology, structural behavior, load bearing resistance and ductility as well as to structural dynamics. The last named will be complemented in Appendix A, dealing with systems with several degrees of freedom.

Chapter 3 treats the principles of earthquake engineering, with an emphasis on the earthquake resistant structural design. In addition, the technological feasibilities and consequences of the use of lightweight respectively drywall constructions are discussed, with a general explanation of earthquake codes to follow exemplified by means of Eurocode 8. The chapter concludes with some comments on practical earthquake analyses - mathematical models, soil effects and implementation of results in the detailed design. Furthermore, the handling of structural parts as well as of possibly important components is addressed because this is or may be basis for the design of some drywall constructions.

Chapter 4 specifically is dedicated to drywall constructions with particular emphasis on the application in seismic zones. Following a general overview over their principal areas of application the individual drywall constructions and their components are presented and earthquake relevant structural details are addressed. Appendices B to E provide data, design backing as well as technical information of the drywall manufacturers. Thus, the designer has a tool that allows for the favorable design of drywall systems even for buildings in zones with increased seismic hazard.

In order to particularly appeal to the practically minded engineer and architect the authors have utilized highly lucid explanations of the basic principles and phenomena. The unique aspects of drywalling should also serve as suggestion and increase of know-how to the earthquake specialist.

Iphofen, January 2008

Contents

1 Introduction

1.1 Earthquake Damages and Protection Objectives

Earthquakes are an ongoing occurrence on earth. Most of them are not even noticed by man. Larger earthquakes cause damages and some few earthquakes per annum cause - at least locally - disasters, the loss of human lives and considerable damage to property and financial loss. Due to the increasing settlement density (ever closer and higher), the settlement of higher risk areas and the more sophisticated infrastructure of buildings and cities the seismic risk (losses per annum) increases continuously. Here earthquake-resistant design serves the purpose of taking countermeasures in order to reduce the risk, but at least to limit the rise in risk.

A good - unfortunately rather sad - example for the value of seismic design is documented by Bachmann (/1.1/): The impacts of two earthquakes are compared: the Spitak earthquake in Armenia of December 7, 1988, with a magnitude of $M = 6.9$, and the Lome Prieta earthquake in California of October 17, 1989, with a magnitude of $M = 7.1$. Both earthquakes were of about similar intensity, and also the topographical conditions as well as the type of settlement and the population density are similar. Table 1.1 compares the number of victims and the damage statistics. It is evident that the differences are up to nearly three orders of magnitude.

Table 1.1 Victim and damage statistics of two earthquakes

	Spitak	Lome Prieta
Death toll	> 25,000	67
Injured	31,000	2,435
Unsheltered	514,000	7,362
Property damage	unknown	7 bn. €

Here the outcome of decades of efforts undertaken by the scientists and engineers active in the field of earthquake engineering becomes evident. For in California intensive earthquake research was started in the 1950s, along with the training of the students and engineers direct involved in earthquake engineering. In addition, measures were undertaken to inspect and refurbish older buildings, along with organizational preparations for emergencies. Furthermore, the relevant building standards were continuously updated. Therefore, most of the damage that occurred during the Lome Prieta earthquake was confined to older buildings that had not yet been adequately refurbished. Only a negligible number of buildings that had been built according to more recent findings experienced minor damage. Unfortunately, this was not the case during the Spitak earthquake. Even buildings that had been built more recently experienced very severe damage since the rules of earthquake-resistant construction were largely ignored or insufficiently taken into account. At the same time the comparison in a spectacular manner proves the progress achieved in earthquake engineering in the last decades.

The damage situation during the Spitak earthquake is all the more woeful when one considers that seismic design is less a question of cost, but primarily a question of intelligent planning – and this finding is almost as old as earthquake engineering itself. It is always better and cheaper to observe some basic rules of construction and thus design an earthquake resistant structure rather than to retrofit a poorly designed structure against earthquake or analyze it by highly sophisticated calculations and investigations.

At this stage the protection objectives to be achieved in case of an earthquake must be reconsidered in more detail. Table 1.2 shows the protection objectives graduated according to requirements, which are listed in ascending order from top to bottom.

Table 1.2 Earthquake design - protection objectives

Protection Objective	Exemplification
Personal safety	■ Stability of buildings ■ Enclosure of combustible liquids and gases
Investment protection	■ Limitation of cost of repair ■ Limitation of unavailability
Operability	■ Serviceability/usability of buildings after an earthquake ■ Repairs while utilisation
Damage free	■ Preclusion of structural damages (shock protection)

Personal safety is the primary and usually the sole concern of the authorities responsible for public safety and thus also the primary objective of all seismic codes. In particular, these codes deal with the stability of structures. It is only to some extent that they deal with other public safety aspects, if indeed. In the ordinary structural engineering (domestic and commercial housing as well as public buildings such as schools, hospitals and event buildings), however, these additional aspects hardly play a roll at all.
Investment protection can be the more far-reaching concern of an owner: to limit the economic risk in seismic active zones, at least for the smaller, more frequent earthquakes. The desire for operability is even more acute. In addition to economic aspects, public interest can again play a role here, if it is a matter of the operability of important buildings making up the infrastructure (e. g. hospitals). The highest de-

mands placed on damage protection build the bridge to classical shock protection which can be useful for very frequent, small earthquakes.
Needless to mention that the building safety addressed in Table 1.2 covers only part of the protection objects. There are, of course, additional aspects e. g. of fire safety and supply guarantee (infrastructure, energy, communication, traffic ...), but these sections go beyond the scope of purpose of this book.
All in all, the table lists protection objectives graduated according to the expected or desired performance of the building. The design is graduated accordingly. Therefore, this design method is called “Performance Based Design”, i. e. design specifically focused on the achievement of a desired performance during earthquake.
For the implementation of the protection objectives a classification of the buildings is necessary. Depending on such classification a building is assigned a certain importance factor by the various national seismic codes (also see Chapter 3.2.2).

1.2 Historical Earthquakes

Areas with regularly recurring earthquakes of high intensity can be subdivided into two groups:

- Edges of major tectonic plates. Such edges of continental plates of the earth's crust are e. g. the central Mediterranean – the Balkans – Turkey – Iran – northern India/ Himalayas – Indonesia – Oceania; the Philippines – Japan – Alaska – California – South American Andes; East-African Rift Valley.
- Edges of regional tectonic plates and adjacent fault zones. These include parts of the Alps, the Rhine valley with the Swabian Alb, Romania, the Caucasus and China.

Figure 1.1 shows a general map in which areas of high seismic activity are highlighted in color.

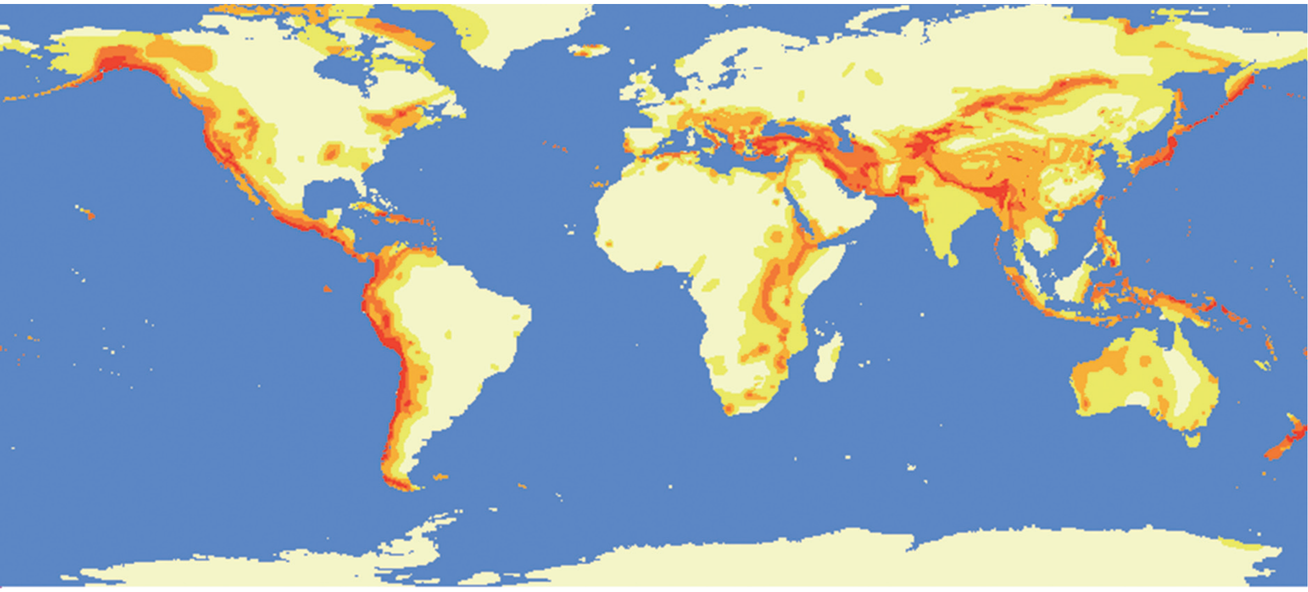


Figure 1.1 World map of earthquake zones

Table 1.3 contains a list of selected historical earthquakes with information as to magnitude and approximate number of victims. Especially the earthquakes from the 1980s and 90s have contributed significantly to the understanding of the performance characteristics of old and new buildings during earthquakes.

Table 1.3 Important historical earthquakes

Year	Location	Magnitude	Death toll
1556	Shensi, China		800,000
1737	Calcutta, India		300,000
1755	Lisbon, Portugal		60,000
1906	San Francisco, California	*8.3*	750
1923	Tokyo, Japan	*8.3*	140,000
1960	Agadir, Morocco	*5.9*	12,000
1863	Skopje, Macedonia	*6.0*	1,000
1976	Friaul, Italy	*6.5*	1,000
1976	Tangshan, China	*7.8*	250,000
1980	El Asnam, Algeria	*7.5*	3,000
1985	Mexico City	*8.1*	20,000
1988	Spitak, Armenia	*6.9*	25,000
1989	Lome Prieta, California	*7.1*	67
1995	Kobe, Japan	*6.9*	6,000
1999	Izmit, Turkey	*7.4*	20,000
2005	Kashmir, Pakistan / India	*7.6*	70,000

The probability of a major earthquake decreases with increasing distance from the near fault zones. This applies to Central Europe as well as to the more northern areas of the Central Asian states. Nevertheless, strong-motion earthquakes can occur also in these areas, and especially with high population density enormous damage potential then exists. Earthquake resistant construction is therefore not only justified in farther areas, but imperative necessity.

1.3 Seismic Risk and Design Earthquake

What earthquake intensity should one plan for? In order to obtain an answer to this question it is essential to understand the term risk in more detail: Risk is generally defined as follows:

Risk	**=**	***Frequency of occurrence of an event***	**x**	***Extent of damage resulting therefrom***

With regard to the load case earthquake the definition reads more exactly (in symbolic spelling):

$$\textit{Seismic risk} = H \bullet R \bullet C \tag{1.1}$$

with

H = Frequency of occurrence of an earthquake of a defined intensity as result of the Seismic **H**azard Analysis,

R = Conditional probability that this earthquake causes a crucial state as result of a Seismic **R**esponse and Fragility Analysis,

C = Extent of damage due to the crucial state as result of a **C**onsequence Analysis.

The frequency of occurrence *H* is given by nature. For the time beyond the period under observation (500 to 1000 years) it can be extrapolated mathematically by means of extreme value statistics.
The conditional probability of a crucial state *R* is defined by the load bearing performance of the structures and their energy dissipation potential. Earthquake engineers focus on the reduction of the probability of such damage for both new buildings and within the framework of inspection and refurbishment of older buildings.
The extent of damage *C* is determined by parameters of use, type of settlement and settlement density, traffic and supply infrastructures etc. Here administrative measures and organizational preparations for emergencies may diminish the extent of damage.
If due to common consensus there is agreement about the tolerable seismic risk, the frequency of occurrence of the earthquake resp. its reciprocal, i. e. the return period, which serve as basis for the seismic design, can be defined, under consideration of the design criteria covered by *R* and the boundary conditions covered by *C*. For that purpose European and many other countries agreed to design buildings against an earthquake which occurs with a probability of 10 % within 50 years. Taking as a

basis the Poisson distribution over the time (the approximation is valid only with little frequency of occurrence),

$$P(T, I \geq i) = 1 - e^{(-pT)} \approx 1 - (1 - p)^T \qquad (1.2)$$

a return period of 475 years results. Within this period the design earthquake occurs with a probability of 63 %. In equation (1.2) *P* is the probability assumed with 10 % that the design earthquake of intensity *I* is exceeded within the period of *T* = 50 years. The value *p* then is the frequency of occurrence of the event itself ($2.1 \cdot 10^{-3}$), its reciprocal is the return period of 475 years searched for. The International Building Code (IBC) of the United States of America currently proceeds from the assumption of a probability of 2 % within 50 years instead, leading to a stronger earthquake with a return period of 2450 years.
Furthermore, the earthquake codes would use so called importance factors by means of which the seismic forces are scaled. Thus, buildings – dependent on their importance and their danger potential – are designed actually against an earthquake with respectively smaller or larger return period.

The last step of a seismic hazard analysis consists in the assignment of the frequency of occurrence of the earthquake and its intensity for a specific site and – as an enhancement – for a whole area. Chapter 2.1.5 will deal with it in more detail; the more precise definition of the intensity is given in Chapter 2.1.2.
The intensity assigned to the return period of 475 years per site is the objective of the hazard analysis. Isoseismals, i. e. lines of same intensity, can be derived from these pointwisely determined intensities. Such isoseismals again serve as basis for the earthquake zone maps, as they are regulars of every national earthquake code; the boundaries of the earthquake zones run along the isoseismals.

Figure 1.2 as an example shows the earthquake zone map of Greece.

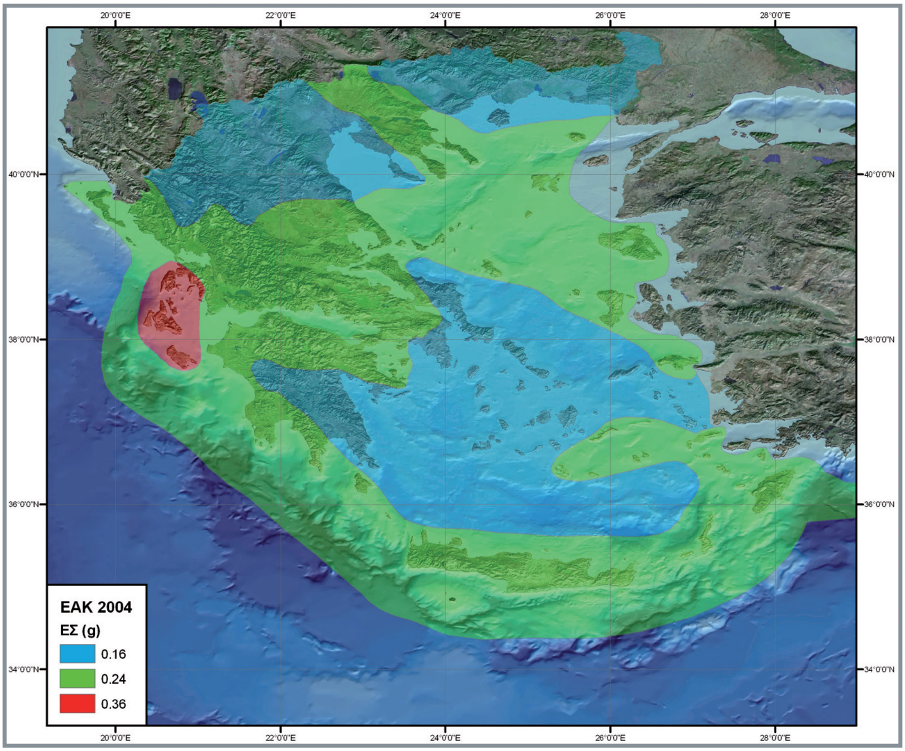


Figure 1.2 Earthquake zone map of Greece from EAK 2004

2 Basics

2.1 Seismology

2.1.1 Formation and Propagation of Seismic Waves

Most earthquakes are tectonic earthquakes. The occasionally occurring volcanic earthquakes, collapse earthquakes, artificial lake induced earthquakes and earthquakes initiated by people (e. g. blasting) are of local importance only and thus are not treated herein any further.
A tectonic earthquake occurs because of a sudden break in the earth's crust. This crust is only a thin skin with a thickness of about 10 km (sea range) to 100 km (mountain range) swimming on the viscous-plastic material of the upper earth's mantle. Due to convection processes there going on the crust is incessantly in motion. The earth's crust is broken into individual clods moving slowly against each other. The movements lead to stresses in the edge zones of the clods, but also at other places. With these stresses reaching the shearing strength of the rock, part of the potential energy there stored is suddenly released and radiated as kinetic energy in the form of seismic waves. This formation of an earthquake is shown schematically in Figure 2.1.

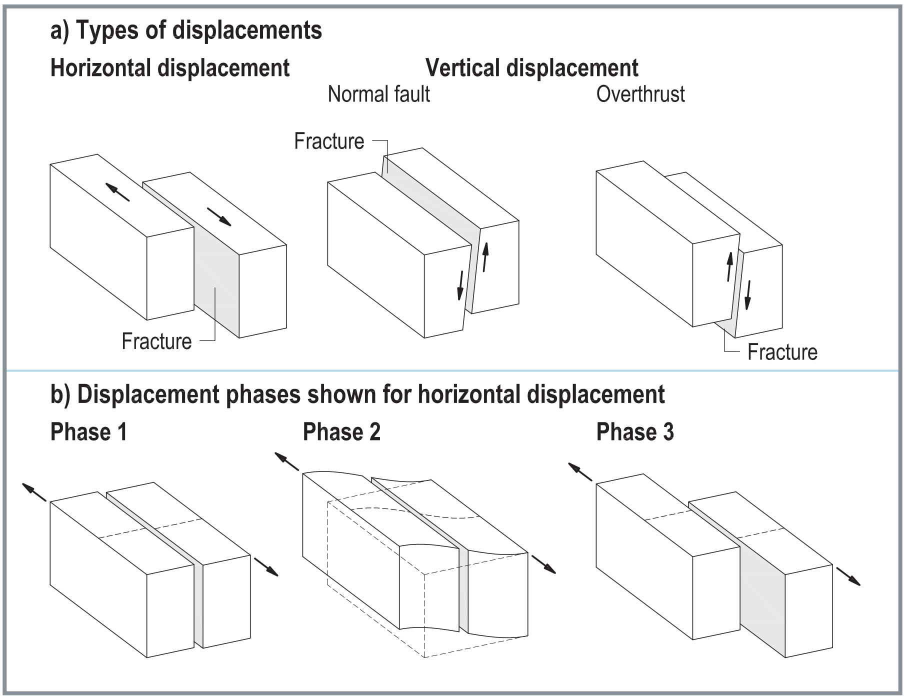


Figure 2.1 Schematic description of the formation of earthquakes

Figure 2.2 illustrates the most important terms of earthquake theory. The point of origin of an earthquake is the hypocenter as center of the source area in the depth *h*. The point on the earth's surface vertical to and directly above the hypocenter is called epicenter. The building to be considered is in epicentral distance Δ from the epicenter and in hypocentral distance *s* from the point of origin of the earthquake. Magnitude *M* and intensity *I* are measured quantities of earthquakes, they are explained in Chapter 2.1.2 in detail. Lines of equal intensity on the earth's surface are called isoseismals.

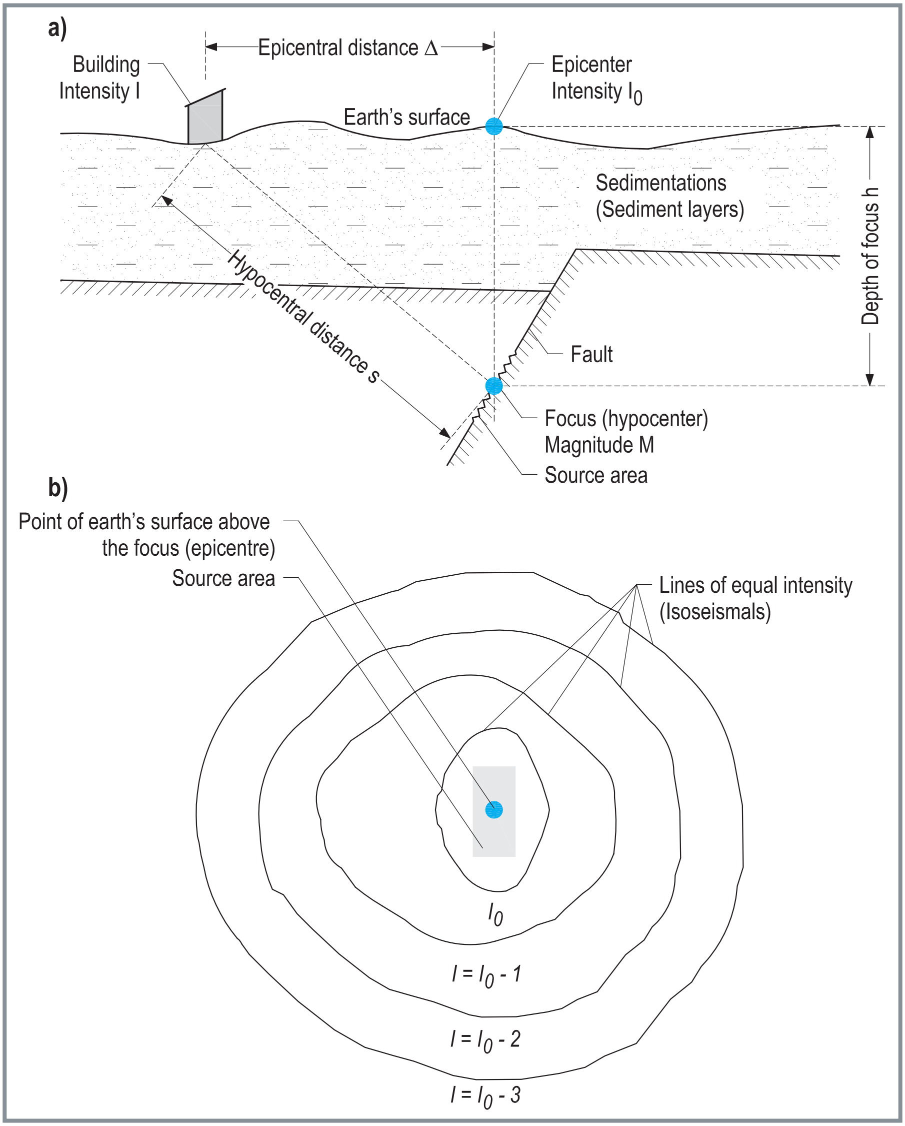


Figure 2.2 Important terms of the earthquake
a) Section through the hypocenter b) Map of isoseismals

Partly the seismic waves pass the interior of the earth as longitudinal and transversal body waves, partly they propagate as surface waves along the earth‘s surface. With longitudinal or compression waves the particles of the propagating medium swing in the direction of the wave propagation, whereas with transversal or shear waves they swing perpendicular to the direction of propagation of the wave. The propagation of longitudinal waves is caused by volume change of the medium, whereas that of transversal waves is caused by shape change. With regard to surface waves one distinguishes between Rayleigh waves with particles moving in a vertical plane, and Love waves with particles swinging horizontally only. Figure 2.3 shows vividly the wave pattern.

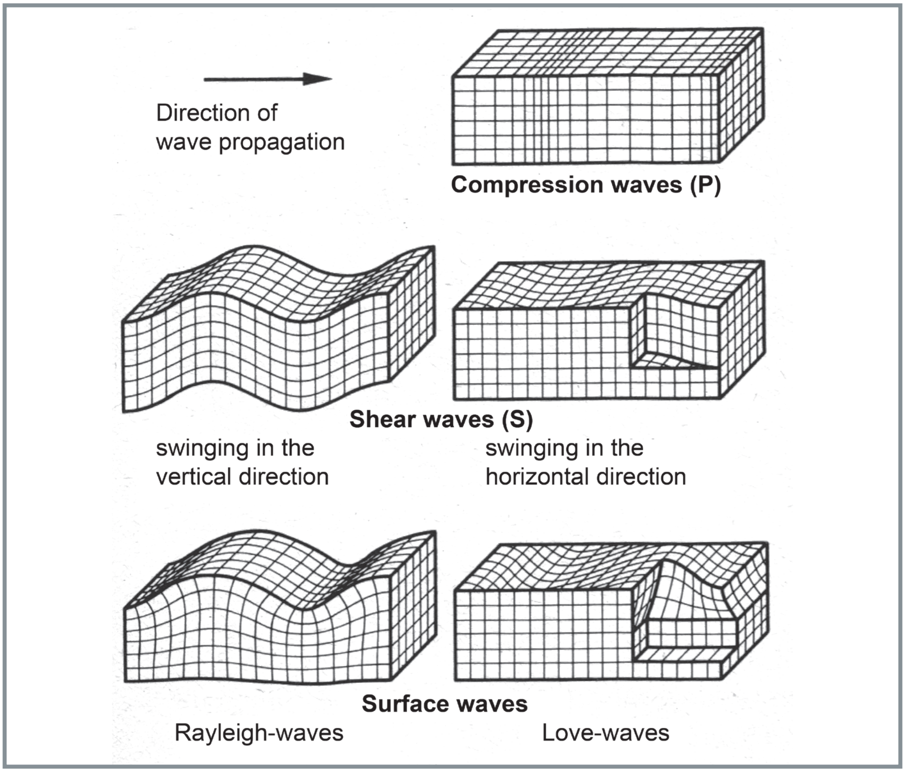


Figure 2.3 Seismic waves (acc. to /2.2/)

In theory shear waves propagate with the velocity

$$v_s = \sqrt{\frac{G}{\rho}} \tag{2.1}$$

compression waves with the velocity

$$v_p = \sqrt{\frac{G}{\rho}} \cdot \sqrt{\frac{2 \cdot (1-\nu)}{1-2 \cdot \nu}} \tag{2.2}$$

with:

$G = \dfrac{E}{2 \cdot (1+\nu)}$		[kN/m²]	Dynamic modulus of shear of the soil
E		[kN/m²]	Dynamic Young's modulus of the soil
ρ		[t/m³]	Density of the soil
ν	$(0 < \nu < 0.5)$	[-]	Poisson's ratio of the soil

According to (2.2) the double shear wave velocity results for the compression wave velocity when assuming an average value of ν = 0.33 - as usual for gravel/sand - for Poisson's ratio of the propagating medium. Compared to the shear waves the compression waves thus arrive earlier at a point distant from the source. Therefore they are described as p-waves (undae primae) and the shear waves as s-waves (undae secundae) respectively. The Rayleigh wave is about 10 % slower than the shear wave.

The propagation velocity v is associated with the frequency f in Hz and the wavelength λ following the common relationship

$$v = f \cdot \lambda \quad . \tag{2.3}$$

The amplitudes of the seismic waves decrease with increasing distance from the source. As of a certain distance r_1 from the source the attenuation depends on the distance r following the relationship

$$v(r) = v(r_1) \cdot \left(\frac{r_1}{r}\right)^n \cdot e^{-2\pi D \cdot (r - r_1)/\lambda} \tag{2.4}$$

with n being the exponent of the frequency-independent geometric amplitude decrease due to energy radiation. With body waves it is 1 and with surface waves 0.5. Parameter D represents the soil damping from absorption, λ is the wavelength. Because of (2.3) the amplitude attenuation due to absorption depends on frequency. This leads to that with increasing distance the low frequent, i. e. the long wavelength content, more and more dominates the earthquake signal, whereas the high frequent, short wavelength content of the signal is damped out.

At stratum boundaries seismic waves are refracted and reflected. For the passing waves every stratum acts as an additional filter, leading to a modification of the spectrum of the earthquake excitation. This is of great practical importance for earthquake engineering since the spectrum of the earthquake at the earth's surface determines the impact on the building. When reducing the seismic wave to a simple harmonious oscillation and neglecting the influence of the stratification on the frequency, the ratio of the acceleration *a* with the pass-through of the wave from medium 1 to medium 2 can be assumed as follows:

$$a_2 = a_1 \cdot \sqrt{\frac{\rho_1 \cdot v_1}{\rho_2 \cdot v_2}} \tag{2.5}$$

with $\rho \cdot v$ being the impedance of the soil. If the second medium is dynamically more flexible than the first one, i. e. $\rho_2 \cdot v_2 < \rho_1 \cdot v_1$, an amplification of the acceleration occurs. This leads to an increased seismic hazard of buildings which were constructed on a weak stratum (sediment or fill) overlying the rock horizon. The great damage of the Mexico earthquake of 1985 e. g. mostly results from this effect.

2.1.2 Earthquake Scales (Magnitude, Intensity)

Earthquake scales are used for the quantitative description of an earthquake, whereas it has to be distinguished between the magnitude scale (Richter scale) and the intensity scale.

The magnitude *M* is a measure of the source energy, i. e. a measure for the energy radiated during an earthquake from the source in by elastic waves. It is derived from the maximum amplitudes of velocity seismograms. One distinguishes between three definitions with respect to the frequency range in which the measurements and/or evaluations are carried out:

- Near field earthquake magnitude (local magnitude) M_l, determined for frequencies above 1 Hz, defined in 1935 by Richter, valid for epicentral distances Δ up to 500 km
- Body wave magnitude M_b, determined at 1 Hz, defined in 1945 by Gutenberg, computed from the maximum amplitudes of the body waves (also referred to as far field earthquake magnitude)
- Surface wave magnitude M_s, determined at 0.05 Hz, defined in 1945 by Gutenberg, computed from the maximum amplitudes of surface waves.

For the interrelation between magnitude *M* and the energy *E* released during the earthquake the following empirical relationship exists:

$$log\,E[erg] = 11.8 + 1.5 \cdot M \quad (2.6)$$

So the "open ended" magnitude scale (Richter scale) is a logarithmic scale. An increase by one unit, e. g. from 7 to 8, corresponds to an increase of the source energy by the factor $10^{1.5} \approx 32 = 2^5$. Two tenths then mean a factor 2, two units a factor 1000. Since the released energy is correlated with the size resp. length of the source, the scale is open ended in theory only. To a magnitude $M = 9$ a source length of more than 1000 km would belong, which represents a realistic actual upper limit on Earth.

Furthermore, interrelations exist between magnitude and frequency content as well as duration of the strong motion phase. With increasing magnitude the main excitation range of the earthquake is shifted to lower frequencies and the duration increases.

Compared to the magnitude the intensity I of an earthquake is a descriptive measure for the local macroseismic (which means detectable without instrumental aids) effects on people, nature and buildings. An earthquake thus has only one magnitude, but an infinite number of intensities, changing from site to site. The maximum intensity observed in the epicenter (epicentral intensity) is designated with I_0.

The earthquake intensity at a site depends on many parameters:

- Magnitude
- Frequency content of the source signal
- Focal depth and distance between source and site
- Geology and topography in the transmission medium
- Local subsoil and foundation soil at the site
- Frequency content at the site, dependent on the filter effect of the transmission path
- Duration of the earthquake (strong motion duration) at the site

Thus, the magnitude is only ONE cause variable, its correlation with the intensity is limited. Therefore, empirical formulae that generally only consider the distance for the determination of the intensity from the magnitude lead to strongly scattering results.

For the description of the intensity different scales are used. The most important ones for Europe and America are compiled in Table 2.1. The scale representing the state-of-the-art and as currently valid for Europe is the European Macroseismic Scale (EMS), which has been elaborated on behalf of EU and finally issued in 1998 (/2.3/).

Table 2.1 The most important intensity scales

Abbreviation	Year	Name	Note
RF	1883	Rossi-Forel	Oldest, 10-fold scale
MM	1956	Mercalli	Modified Mercalli Scale, particularly USA
MS		Mercalli-Sieberg	Particularly Germany
MSK	1964	Medvedev-Sponheuer-Karnik	Europe
EMS	1998	European Macroseismic Scale	Latest state Europe

All except for the RF-scale are 12-fold, with relatively slight differences as to the division. The decisive aspect consists in the degree of the detailing of the description and classification of the effects on people, nature and buildings. Table 2.2 shows an abstract of the EMS Scale. Compared to the low intensities, where the description focuses on the effects on people, the higher intensities concentrate on the effect on buildings. For a sharper distinction the buildings are differentiated into six Vulnerability Classes (A to F) and the damage into five Damage Categories (negligible/slight damage to destruction/very heavy structural damage). Based on a careful evaluation this classification allows for an exact determination of the intensity - independent on the quality of the existing basic structure of a building. Earthquake engineers altogether focus on intensities VI to X. The lower intensities primarily are of geophysical importance, the higher ones practically cannot be controlled from the structural point of view. For more detailed definitions and comments as to the application please refer to /2.3/.

Note: Often empirical relationships between intensity and maximum ground acceleration are indicated. The correlation, however, is weak and the values scatter up to the factor 3 and more. The ground acceleration tends to duplicate with every intensity stage.

Table 2.2 European Macroseismic Scale, Abstract

EMS Intensity	Definition	Description of the maximum Effect (short)
I	Not felt	Not felt.
II	Scarcely felt	Felt only by very few individual people at rest in houses.
III	Weak	Felt indoors by a few people. People at rest feel a swaying or light trembling.
IV	Largely observed	Felt indoors by many people, outdoors by very few. A few people are awakened. Windows, doors and dishes rattle.
V	Strong	Felt indoors by most, outdoors by few. Many sleeping people awake. A few are frightened. Buildings tremble throughout. Hanging objects swing considerably. Small objects are shifted. Doors and windows swing open or shut.
VI	Slightly damaging	Many people are frightened and run outdoors. Some objects fall. Many houses suffer slight non-structural damage like hairline cracks and fall of small pieces of plaster.
VII	Damaging	Most people are frightened and run outdoors. Furniture is shifted and objects fall from shelves in large numbers. Many well built ordinary buildings suffer moderate damage: small cracks in walls, fall of plaster, parts of chimneys fall down; older buildings may show large cracks in walls and failure of fill-in walls.
VIII	Heavily damaging	Many people find it difficult to stand. Many houses have large cracks in walls. A few well built ordinary buildings show serious failure of walls, while weak older structures may collapse.
IX	Destructive	General panic. Many weak constructions collapse. Even well built ordinary buildings show very heavy damage: serious failure of walls and partial struc-tural failure.
X	Very destructive	Many ordinary well built buildings collapse.
XI	Devastating	Most ordinary well built buildings collapse, even some with good earthquake resistant design are destroyed.
XII	Completely devastating	Almost all buildings are destroyed.

2.1.3 Earthquake Registrations and Response Spectra

Although magnitude and intensity present measured values for the definition of the seismic impact, such values cannot yet serve as basis for a structural design. For this purpose information on amplitude, frequency content and duration of the earthquake is necessary, based on the earthquake registrations which are recorded worldwide by seismic measurement stations.

With regard to the measuring instruments it has to be distinguished between velocity measuring equipment as used preferably for the registration of smaller earthquakes, and acceleration measuring equipment for the recording of strong motion earthquakes. The equipment mentioned first particularly serves geophysical investigation purposes, the latter as basis for engineering seismic evaluations. Figure 2.4 shows an acceleration time history of the Roermond Earthquake of 1992 in the Lower Rhine Bay.

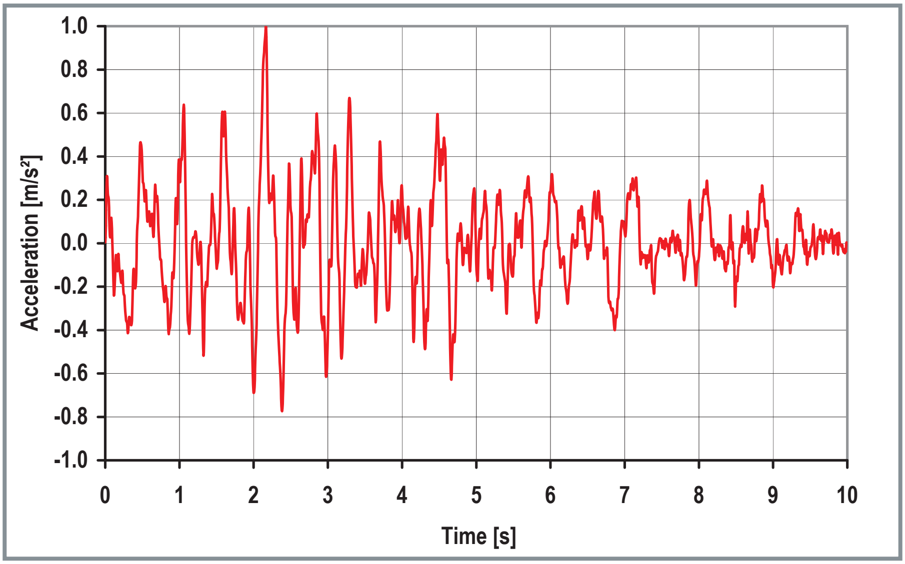


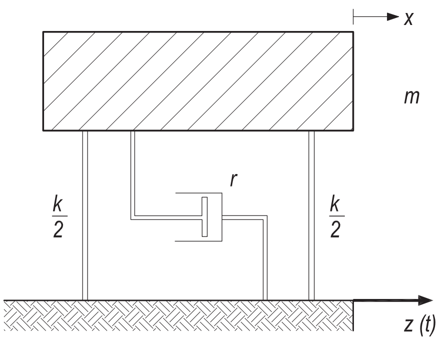
Figure 2.4 Acceleration time history of the Roermond Earthquake 1992

Its maximum acceleration is scaled to 1 m/s², the overall duration is 10 s with a strong motion phase of approx. 5 s.

The measured acceleration time history of an individual earthquake, of course, cannot serve as basis for the structural design against a future earthquake. Here, a generalization is necessary, leading from the specific characteristics of the individual earthquake to a generalized “design earthquake“. This method of generalization considers a derived parameter: the acceleration response spectrum. For explanation purposes a little excursion to the fields of structural dynamics and mathematics is necessary.

(Compared to Chapter 2.3 and Appendix A dealing with the basics of structural dynamics in more detail, this chapter just serves to make the response spectrum comprehensible.)

Figure 2.5 shows a one-mass oscillator, or - more generally - a single-degree-of-freedom-system. It is characterized by three physical parameters: the (rigid) mass *m*, the (massless) spring of the stiffness *k* and the dissipative, i. e. energy consuming, damper element of the damper resistance *r*. From the afore mentioned three physical parameters two mathematical parameters can be derived, which characterize the equation of motion: the natural (undamped) angular frequency ω and the damping factor *D*. The definitions are also given in Figure 2.5. The absolute motion of the mass be *x*, the horizontal excitation at the base point of the one-mass oscillator *z(t)* and the relative motion $y = x - z$.



$$\omega = \sqrt{\frac{k}{m}} \quad , \quad D = \frac{r}{2 \cdot \sqrt{k \cdot m}}$$

Figure 2.5 One-mass oscillator

With the parameters from Figure 2.5 the equation of motion (cf. Chapter 2.3.2.1) becomes (points mean derivations with respect to time):

$$\ddot{y} + 2D\omega \cdot \dot{y} + \omega^2 \cdot y = -\ddot{z}(t) \qquad \text{with} \qquad \ddot{y} = \ddot{x} - \ddot{z} \tag{2.7}$$

On the right side there is the acceleration time history of the ground. The response quantities searched for are:

$y, F = k \cdot y$	Relative displacement, spring force
$\dot{y}, R = r \cdot \dot{y}$	Relative velocity, damping force
$\ddot{x} = \ddot{y} + \ddot{z}$	Absolute acceleration

With given $\ddot{z}(t)$ – e. g. the measured acceleration time history of the earthquake as available in digital form – it is easy to integrate, i. e. solve equation (2.7) by means of mathematical routines, leading to the afore mentioned quantities searched for, as a function of parameters ω, D and the time t. Usually only the absolute peak value of the three quantities within the time history is of interest to the engineer. Thus, the time parameter is eliminated so that only the two parameters ω and D remain. These absolute peak values, plotted over frequency $f = \omega / 2\pi$ (or its reciprocal, the period $T = 1 / f$) with D as family parameter, are called response spectra S_a, S_v and S_d for acceleration, velocity and displacement:

$$\begin{aligned} S_a &= max\left|\ddot{x}(t,\omega,D)\right| \\ S_v &= max\left|\dot{y}(t,\omega,D)\right| \approx S_a / \omega = S_{pv} \\ S_d &= max\left|y(t,\omega,D)\right| \approx S_a / \omega^2 = S_{pd} \end{aligned} \tag{2.8}$$

As equation (2.8) shows, the response spectra of velocity and displacement can be expressed approximately by the response spectrum of the acceleration. That is called the spectrum of the pseudo velocity S_{pv} *and pseudo displacement* S_{pd}. The approximation for the displacement is very good with small damping, in the low frequency range the approximation for the velocity deviates more strongly. Figure 2.6 shows the acceleration response spectrum for the time history of Figure 2.4 for three damping factors D.

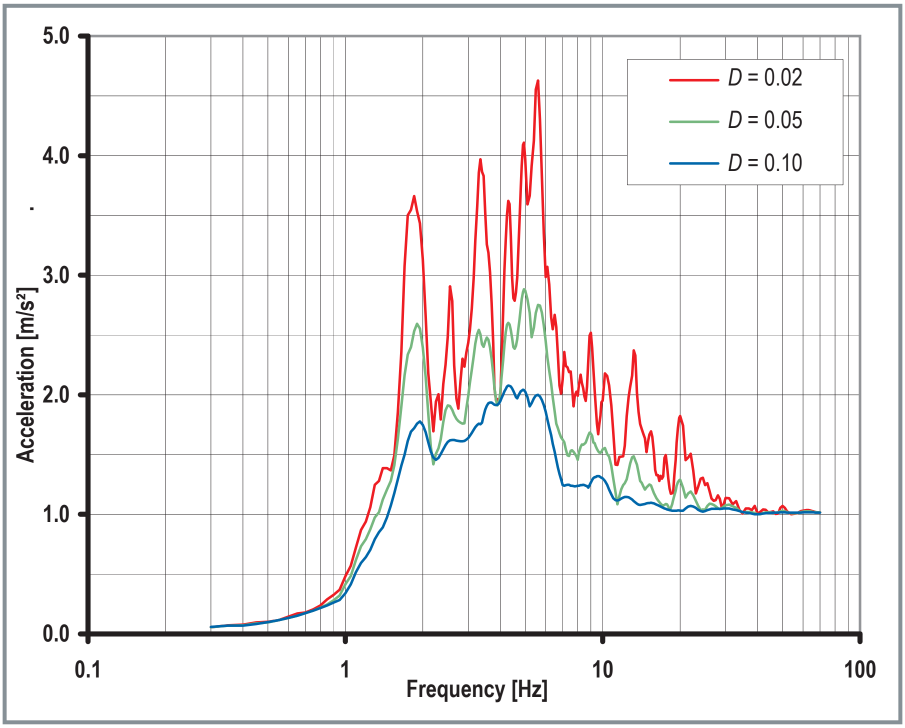


Figure 2.6 (Linear) acceleration response spectrum over f for the time history of Figure 2.4

As the three spectra of (2.8) in the approximation only distinguish by the factor ω, all three curves can be represented together in a double logarithmic plot, again either over the frequency *f,* or its reciprocal, the period *T*. Figure 2.7 shows this graph for the spectra from Figure 2.6.

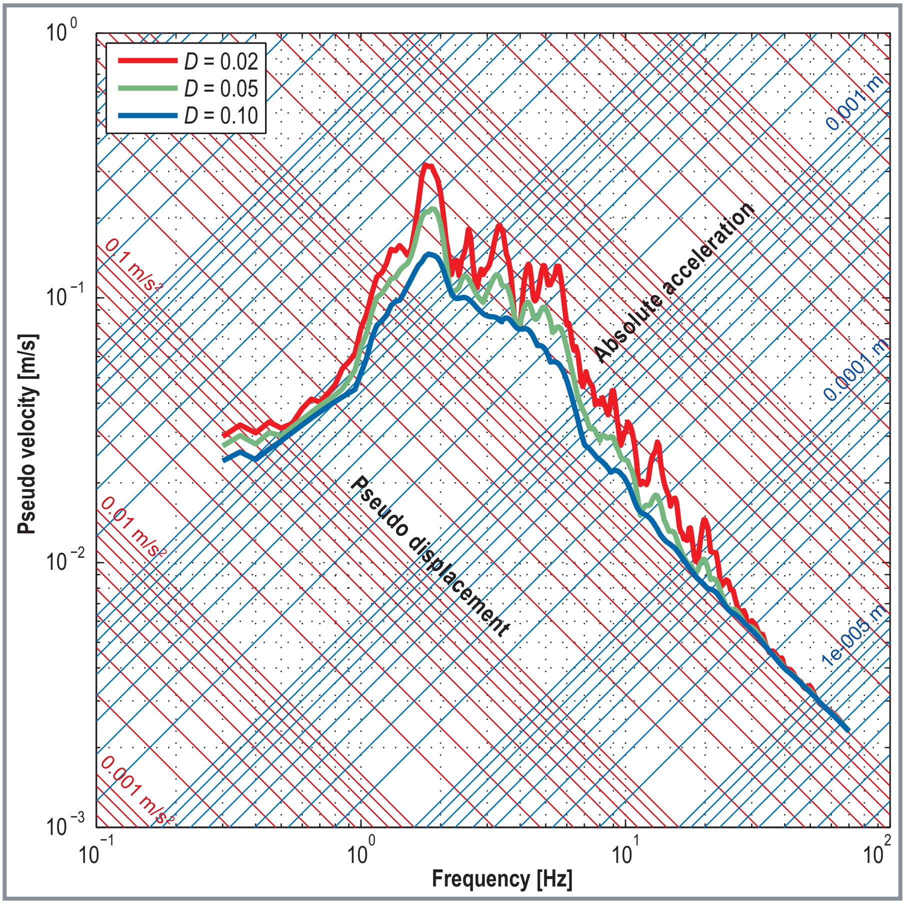


Figure 2.7 Combined representation of the response spectra over f

Characteristics and meaning of the spectra can be summarized as follows:

- The response spectrum indicates the absolute peak value of the response of a one-mass oscillator of arbitrary natural frequency and damping onto the respective base excitation; it thus shows the frequency content of the excitation without regard of the phase relationship.
- With very stiff oscillators the mass in effect moves with the ground, the acceleration spectrum therefore asymptotically reaches the maximum ground acceleration, i. e. the peak value of the corresponding time history, the spectra of the relative velocity and the relative displacement approach zero. The corresponding upper limiting

frequency for earthquake excitation generally lies at 25 to 30 Hz.

- With very flexible oscillators the ground moves below the mass without taking it, the acceleration approaches zero, the relative displacement in turn approaches the ground displacement, the relative velocity the ground velocity, while the pseudo velocity approaches zero.
- In the medium frequency range an amplification of the oscillation responses occurs relative to the ground motion, the one-mass oscillator responds resonance-like to the excitation time history. With decreasing damping the amplification increases, with increasing damping it decreases. Because of the transient character of the excitation, however, it remains finite also for diminishing damping.
- The degree of amplification does not only depend on the damping, but also on the characteristics of the earthquake (e. g. magnitude, distance from the epicenter, transmission behavior of the formations).

The individual response spectrum still shows an irregular shape caused by the specific characteristics of this earthquake. Thus, it still does not qualify for the design of a structure against a future earthquake. This means that further generalization measures are required. The utilisation of a statistically sufficient number of earthquake registrations – which, of course, must be characteristic for the respective region – a proper standardization and statistical evaluation of such registrations already lead to much smoother curves for median (50 % fractile) and median ± standard deviation (84 % resp. 16 % fractile). The utilisation of the median as design basis is today‘s state-of-the-art internationally. Thus, the smoothed median curve provides the basis for the form of the linear, so called elastic ground response spectrum of the design earthquake. It contains the characteristics of the future earthquake on average. Some smoothed spectra are shown in Figures 2.12 and 3.21.

2.1.4 Spectrum Compatible Time Histories

Chapter 2.1.3 described the way from the time history of the acceleration to the corresponding response spectrum. Strictly speaking this way is irreversible since in the course of the determination of the spectrum the phase information of the individual spectral parts went lost. There are analysis methods, however, which work in the time domain, and for non-linear analyses the time domain is indispensable. Consequently, a possibility is required to at least approximately generate time histories which “fit” the specified design spectrum. There is no unique return path, but an infinite number of time histories exists, the response spectra of which do approximate the specified design spectrum. They are generated by the definition of a frequency content of the assumed time function with statistically independent phase relationship of the individual spectral parts by iterative approximation of the amplitudes to that the target spectrum is reached accurately enough. The statistical uncertainty is taken into account by the use of several artificial time histories and by the averaging over the results obtained with these time histories.

Figure 2.8 shows a generated artificial time history to an EC 8 spectrum (spectrum type 1, soil class A, damping coefficient η = 1, ground acceleration $a_g = \gamma_1 \cdot a_{gR}$ = 1.0 m/s²) and the degree of matching of response spectra and target spectra. (Chapter 3.2.2 will deal with the EC 8 spectra in detail.)

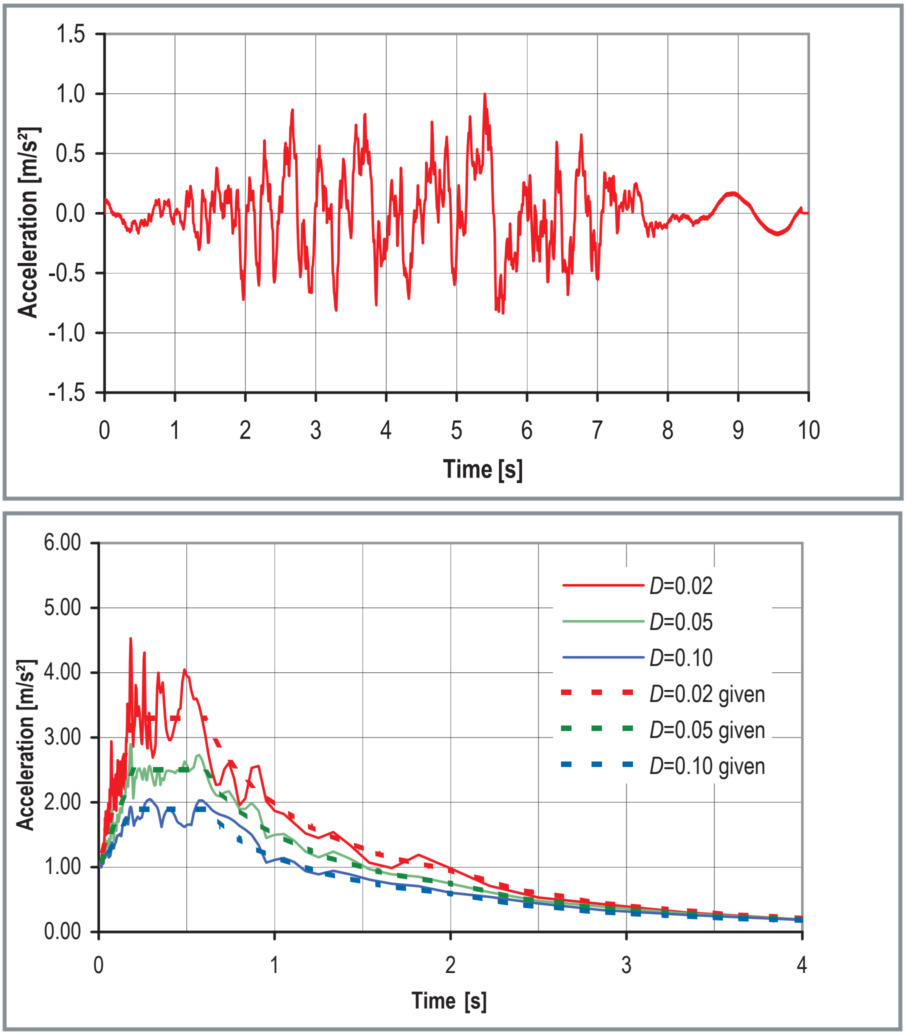


Figure 2.8 Artificial time histories with response spectra and target spectra, over T

Note: Due to the averaging of many earthquakes the constructed design earthquake and the corresponding artificial acceleration time histories are much richer in energy than any single earthquake. Therefore, analyses that are carried out with such artificial time histories are more or less clearly conservative, but show the advantage that actually all structural frequencies are covered. This especially applies to non-linear analyses, the results of which strongly depend on the number of cycles within the strong motion phase.

2.1.5 Earthquake Hazard and Zone Maps

After the statistical definition of the size of the design earthquake via the return period of 475 years (Chapter 1.3) and its mathematical formulation via a smoothed design spectrum (Chapter 2.1.3), for the practical application only the scaling by means of the design intensity is missing. Therefore, as part of a seismic hazard analysis, the assignment of the frequency of occurrence of the earthquake and its intensity is required for both, a specific location as well as – as an enhancement – a whole area.

For this purpose, the area under investigation is subdivided into individual seismotectonic units, the source regions. For every source region a magnitude frequency curve is defined dependent on the local factors as well as an intensity attenuation law for the distance to the location to be investigated. Integration over every source region provides the contribution of this region to the frequency intensity curve $P(I \geq i)$ at the location. The summation over the contributions of all source regions then leads to the global curve $P(I \geq i)$ for the location.

Figure 2.9 shows such a frequency intensity curve for three locations of different seismic hazard exemplarily. Then, for a specific frequency, e. g. the reciprocal of the return period of 475 years as determined above, the corresponding intensity $I \approx 9$ / ≈ 7 / ≈ 5.5.
Vice versa it becomes evident how different frequencies of occurrence can belong to a specific intensity, e. g. $I = 6$, at different locations ($p \approx 10^{-1}$ / 10^{-2} / 10^{-3}).

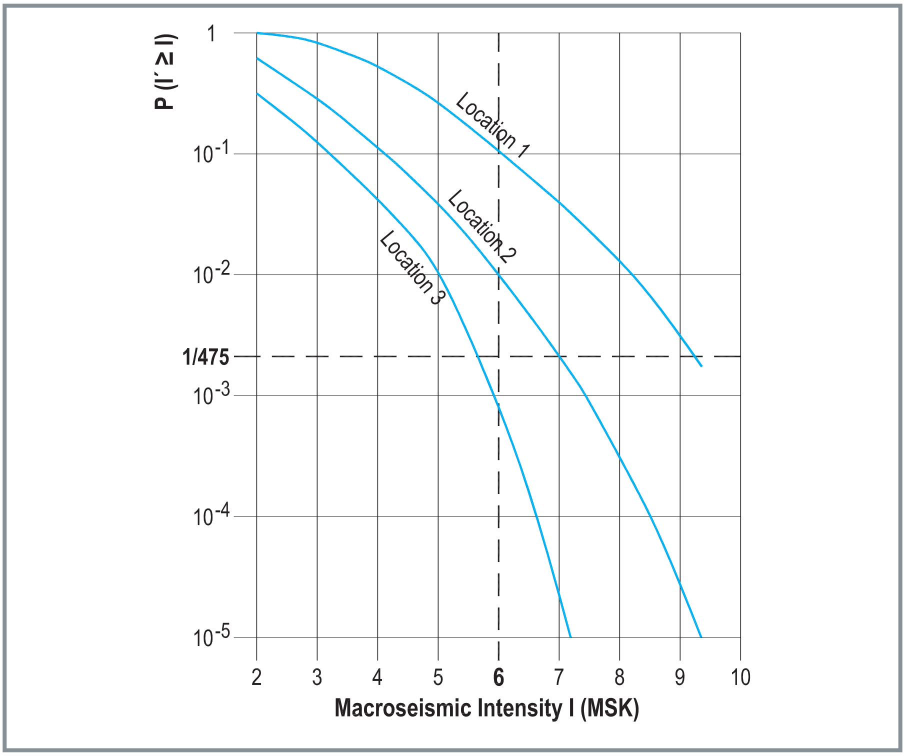


Figure 2.9 Frequency intensity curves for three locations

When now covering the area to be considered with a regular raster and determining such a curve for every pixel, then the intensity as assigned to the return period results, namely for each of the pixels. Interpolation between the pixels allows for the construction of lines of same intensity, the so called isoseismals. These are the basis of earthquake zone maps; the boundaries of the earthquake zones run along the isoseismals. Such an earthquake zone map has been shown in Figure 1.2 already.

In the 1990s, within the framework of the GSHAP project (Global Seismic Hazard Analysis Programme), an international specialised working group investigated the entire earth's surface on the basis of the method described above. The result – an earthquake zone map of the whole world for the return period of 475 years – is available in the Internet under www.seismo.ethz.ch/GSHAP. For every location worldwide, this map indicates the earthquake intensity to be expected for this return period with a probability of 50 %.

2.2 Building Reactions

2.2.1 Overview

Earthquake effects on structures can arise directly from faults at the earth‘s surface as well as from ground motion due to seismic waves. In addition there are indirect effects due to earthquake induced land slide, ground liquefaction or flood waves (tsunamis). Faults of the earth‘s surface can cause great damage in the strong motion earthquake zones, outside of the strong motion earthquake zones they are of little importance.

The potential critical effect of ground motion due to seismic waves on structures especially arises from the high horizontal accelerations which are transmitted by the ground on the structures. Buildings primarily are constructed and analyzed for the transfer of vertical loads, so that their load bearing capacity for horizontal loads is limited. For the same reason vertical accelerations from earthquakes usually are a minor problem for buildings. Here, often the safety factors from the design against vertical dead and live loads are already sufficient to prove the load bearing capacity for the vertical additional loads from earthquake. Furthermore, the vertical earthquake accelerations are smaller than the horizontal ones (usually factor 0.5 to 0.7). Therefore, some earthquake codes – apart from special cases – do not pursue the vertical seismic forces at all. EC 8, for example, recommends the investigation of vertical vibrations only with vertical ground acceleration exceeding 2.5 m/s^2, and even then this recommendation only addresses large spanned horizontal components, especially those bearing columns. The case may be different, of course, when dealing with internals, e. g. drywall ceilings, which due to a subsystem effect possibly have to be designed against vertical earthquake.

The hazard of a structure through seismic vibrations depends on several factors:

- Seismic hazard in accordance with the global seismic zone map (cf. Chapter 2.1.5) and the corresponding design spectra
- Geological underground at the site because of possible amplification effects (cf. Chapter 2.1.1), with influence on form and height of the local design spectra
- Free vibration behavior, i. e. natural frequencies and modes of the structure itself and on the flexible foundation soil
- Load bearing capacity of the structure for higher horizontal loads
- Energy dissipation capacity of the structure in the form of damping, and particularly by non-linear (elasto-plastic) behavior of the stiffening elements (ductility)

The first two points present characteristics of the site chosen. The third point depends on the design of the building and its distribution of mass and stiffness. The two last points are decisive; for according to Bachmann (/1.1/) the following is valid:

“Quality” of the seismic performance ~ load bearing capacity x ductility

Consequently, a structure with small ductility that shall survive an earthquake without collapse must show a high load bearing capacity. With high ductility, however, the load bearing capacity can be small, and a building with medium load bearing capacity and medium ductility has a chance to survive, too.
Thus, for a specific design earthquake a building can be designed in highly different ways: The simplest solution is to provide the supporting structure with a high load bearing capacity to an extent that it can carry the earthquake elastically, ductility is not necessary then. This method can be taken into account primarily in low seismic zones or if only little damage from earthquake is tolerable (cf. Chapter 1.1). For high seismic zones this method generally is too uneconomical. Here a solution with smaller load bearing capacity, but higher ductility should be considered. Greater damage must be tolerable, but a collapse does not occur. However, the high ductility requirement must be safeguarded, of course, by respective design and constructional detailing. In medium seismic zones a medium solution can accordingly be of advantage.

Theoretically, with exploitation of the ductility a structure could be designed on the basis of non-linear dynamic analyses, with acceleration times histories matched to the design spectra (cf. Chapter 2.1.4). This method is used in the areas of research and development. For engineering practice, however, it is not reasonable. Here quasi-linear dynamic analyses with equivalent spectra (cf. Chapter 2.2.3) or linear static analyses with equivalent forces, that are reduced corresponding to the available ductility, are applied. Modern earthquake codes (e. g. EC 8) describe in detail which reductions under which constructional conditions can be applied for which structures. Decisive for the success, however, is that the ductility is “guided“ correctly so that a “favorable“ load bearing behavior results. This is the basic idea of the capacity design (see Chapter 2.2.4).

2.2.2 Ductility

A bilinear force-deflection-diagram as shown in Figure 2.10 serves as basis for the definition of ductility. Generally, this diagram presents an idealization of the bent curve in reality. The ductility then is the ratio between the total elastic-plastic deformation and the elastic deformation at the yield point.

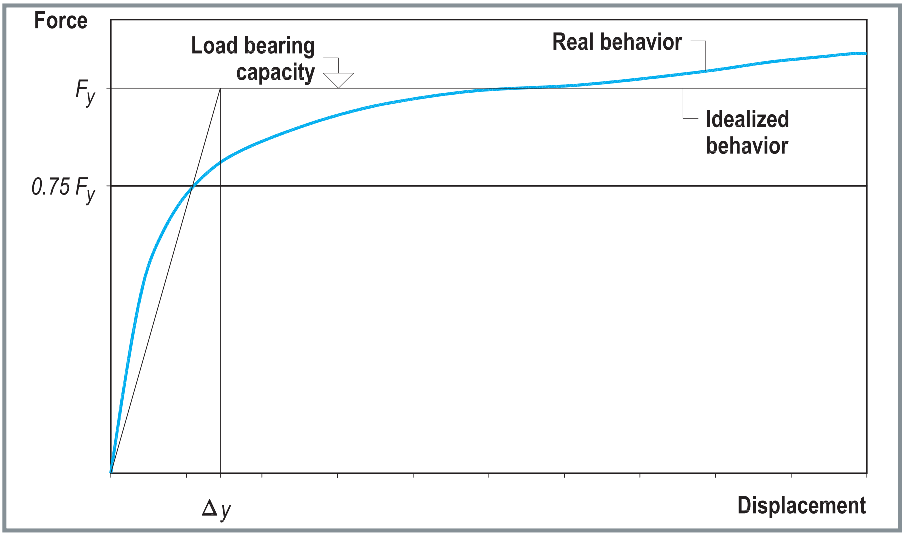


Figure 2.10 Bilinear force-deflection-diagram from measured force-deflection-curves of metal stud partitions with drywalling

There are different types of ductility to be distinguished as per definition:

- Strain ductility μ_ε, defined at a beam element of length 1 under centric tension. It indicates up to which multiple of the strain at the yield point ε_y the element is stretched.
- Curvature ductility μ_φ, defined at a beam element of length 1 under bending stress. It indicates up to which multiple of the curvature angle at the yield point φ_y the element is curved.
- Rotation ductility μ_θ, defined at a plastic hinge of length l_p (plastic length) under combined bending/normal force stress. It indicates up to which multiple of the rotation angle at the yield point θ_y the plastic hinge is distorted.
- Displacement ductility μ_Δ, defined at a whole frame or load bearing structure that can be subject to several forces. It indicates up to which multiple of the displacement at the yield point Δ_y the frame or load bearing structure is displaced at a specific point.

The ductility prior to structural failure is called ultimate ductility. There must be a sufficient safety margin between the design ductility and the ultimate ductility.

The displacement ductility is a global quantity, whereas rotation and curvature ductility are local ones. The local ductilities are the basis for the global ductility and numerically much larger than that in most cases. (E. g. /2.4/ will present related formulae.) According to that the displacement ductility can be around a factor 2 to 3 smaller than the necessary curvature ductility. In addition and strictly speaking, it must be distinguished between the ductility with monotonous stress (static) and cyclic stress (dynamic, earthquake), where hysteresis curves develop. More information to this regard can also be taken from /2.4/. The achievable ductility, however, only presents a very first, coarse damage parameter; the topic is still under research.

2.2.3 Elastic and Inelastic Design Response Spectra

Chapter 2.2.1 explained that a collapse of buildings - though they provide limited load bearing capacity - can be prevented if there is sufficient ductility. The displacement ductility μ_Δ of the whole load bearing structure (see Chapter 2.2.2) serves as measure for the global ductility of buildings, with Δ showing the displacement of the top floor relative to the foundation. In order to provide for the global displacement ductility, all plastic regions must show a corresponding local ductility (curvature/rotation ductility) so that an early structural failure is excluded. Principally a reduction of the elastic load bearing capacity or of the corresponding elastic equivalent load respectively is only possible if such ductilities are guaranteed in the sense of “design ductility“ (cf. Chapter 2.2.1).

The utilisation of higher design ductility, however, involves larger deformations and thus greater damage with the design basis earthquake that must be tolerated. Earthquakes which are very much smaller than the design basis earthquake would cause first damage. The design basis earthquake, however, is a rare event with a probability of occurrence of 63 % in 475 years, so that a certain extent of damage is absolutely tolerable, as long as the collapse is prevented. With the reduction of the elastic load bearing capacity or the elastic equivalent forces respectively it is therefore necessary to take the following aspects into account:

- Costs of load bearing structure; with higher reduction of the load bearing capacity the cost of the load bearing structure is less
- Design and constructional detailing; these require more attention – particularly in the plasticizing areas – because of the higher ductility that is necessary
- Damage at the load bearing structure; the higher reduction of the load bearing capacity involves greater damage there, and - due to the smaller load bearing capacity - damage occurs already with smaller earthquakes

- Damage at non-load bearing components, e. g. lightweight partitions; the higher reduction of the load bearing capacity involves greater damage at non-load bearing components which, however, can be reduced considerably by special measures (e. g. avoiding the use of brittle components, jointing; also see chapter 4).

The standards provide for the reduction by so called behaviour factors as reciprocal of the reduction factors according to the approach

$$F_y = \alpha_\mu \cdot F_{el} = \frac{F_{el}}{q} \tag{2.9}$$

where

F_y	Reduced equivalent force (load bearing capacity)
F_{el}	Elastic equivalent force for which the structure would have to be designed (load bearing capacity which the building should have) to survive the design earthquake elastically
α_μ	Reduction factor
q	Behaviour factor (= $1/\alpha_\mu$).

Simple mathematical approaches exist for the reduction and/or behavior factors. From the principle of the same maximum displacement (Figure 2.11a) of an unlimited elastic and elasto-plastic oscillator with one degree of freedom follows with $\mu_\Delta = \Delta_u / \Delta_y$

$$\alpha_\mu = \frac{F_y}{F_{el}} = \frac{1}{\mu_\Delta} \qquad \text{resp.} \qquad q = \mu_\Delta \tag{2.10}$$

And from the principle of the same deformation energy (Figure 2.11b, same area under the material laws) results

$$\alpha_\mu = \frac{F_y}{F_{el}} = \frac{1}{\sqrt{2 \cdot \mu_\Delta - 1}} \qquad \text{resp.} \qquad q = \sqrt{2 \cdot \mu_\Delta - 1} \tag{2.11}$$

Equation (2.10) applies better in the low frequency range, equation (2.11) in the medium frequency range. Above the rigid body frequency no more plastic deformation and thus no more reduction occurs; the reduction factor is 1 (cf. equation (3.3 a) and the related explanations). Another approach to the reduction factors consists in the direct non-linear analysis of specific load bearing structures or types of load bearing structures under representative earthquake time histories. In particular, this method was applied in the area of research and development, in order to derive and guarantee for the reduction or behavior factors respectively as indicated in the relevant codes and standards.

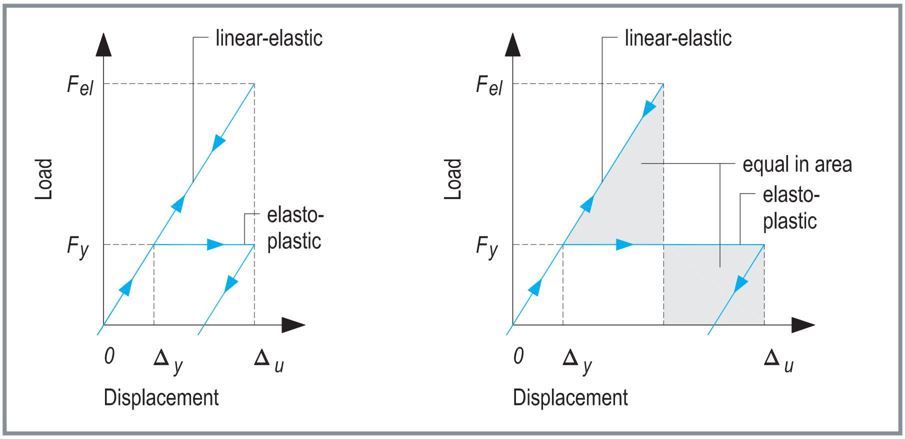


Figure 2.11 Reduction of equivalent force or load bearing capacity respectively
a) Principle of equal displacement b) Principle of equal energy

The idea of the reduced equivalent load described above is now transferred to the design response spectra: By the indicated behavior factors q dependent on the load bearing structure, the elastic design response spectra of the foundation soil as provided zone-dependent in the standards are reduced to an equivalent load, which on the basis of a linear analysis and related design enables the load bearing structure to carry the real load via plastic deformation without loss of stability. These reduced response spectra are also called “inelastic design response spectra“. The constructional qualifications and measures as described in the codes as well, being necessary to guarantee the ductility as required for the behavior factor q, of course, must be fulfilled.

Figure 2.12 shows the elastic response spectrum according to EC 8 (spectrum type 1, soil class C, $\gamma_I \cdot a_{gR}$ = 1.0 m/s²) and two inelastic design response spectra which have been derived therefrom, with behavior factors q = 2 and q = 4. With regard to Figure a) the spectra are plotted over period T, whereas in Figure b) they are plotted about frequency $f = 1/T$. The mathematical definitions of the individual curve sections are described in Chapter 3.2.2.

In the individual national codes and standards the determination of the inelastic spectra by means of the behavior factor q partly is carried out in a simplified manner, however, the basic principle is not influenced.

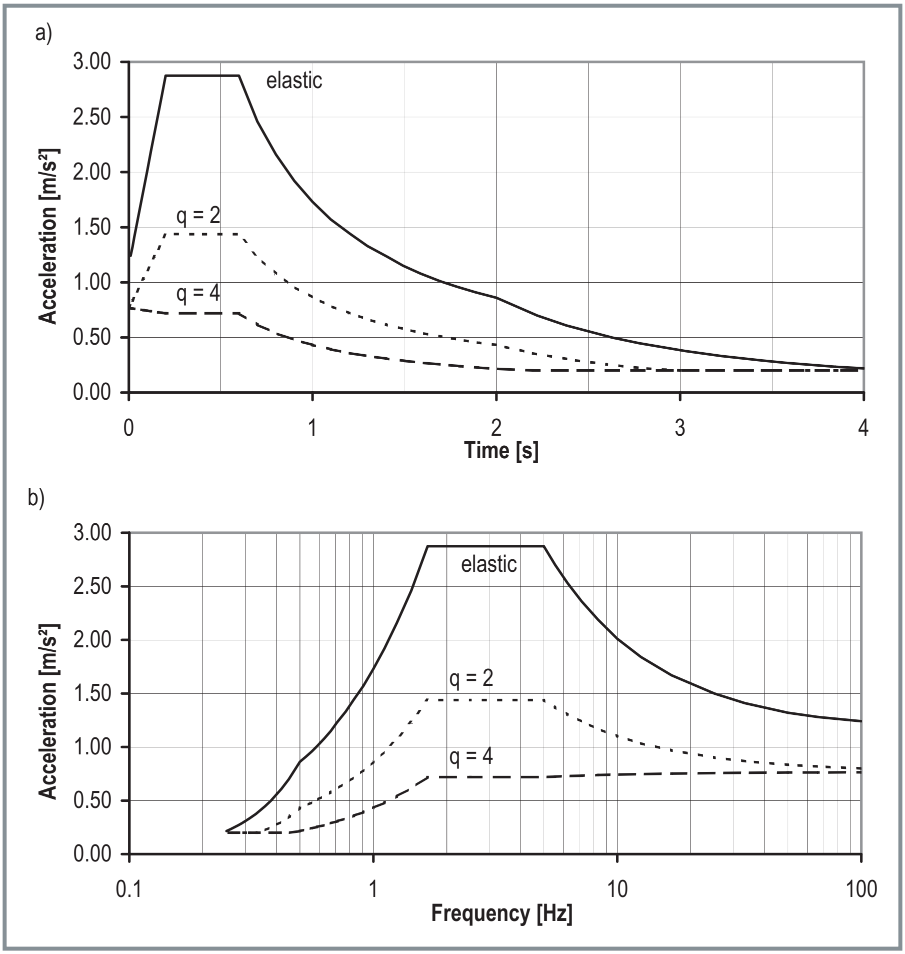


Figure 2.12 Elastic and inelastic design response spectra according to EC 8: Spectrum Type 1 for soil class C
a) Plotted over the vibration period T
b) Plotted over frequency f = 1/T

2.2.4 Capacity Design

In sections 2.2.1 to 2.2.3 the significance of the ductility of load bearing structures for an appropriate and economical design of buildings against earthquakes was elaborated. With this information a procedure has to be developed for the optimization of the degree of protection and the predictability of the behavior of a structure under earthquake. The "capacity design" which was developed by Pauley and colleagues (/2.4/, /2.5/), and which has proved internationally step by step, serves for this aim. Following T. Pauley, the definition reads as follows:

- In a load bearing structure with earthquake action the plasticizing areas are chosen reasonably and determined to that a suitable plastic mechanism develops.
- The plasticizing areas are designed and constructionally detailed to provide sufficient ductility.
- The remaining areas are provided with additional load bearing capacity (capacity) so that they remain elastic when and as far as the plasticizing areas develop overstrength.

By doing so it is guaranteed that only the chosen mechanism can develop for energy dissipation and that it will survive even large deformations without loss of stability. A clear hierarchy of the load bearing capacity develops. Due to the guidance of the ductile behavior to suitable known places, local ductility and required global ductility are synchronised so that the behavior under earthquake action is well predictable, contrary to the conventional design with the local ductilities possibly varying considerably and with the real structural behavior to a large extent remaining unclear. Thus, with a conventional design it can happen that, prior to reaching or exploiting the ductility at the suitable places, a brittle failure occurs at other places, which leads to the collapse of the whole structure. The shear failure of columns before developing the plastic hinges in the girders, or the early failure of struts in steel constructions should be mentioned as known examples.

Of course, the plastic mechanism to be chosen according to the definition mentioned above must be suitable, i. e. is it should allow for as many places as possible developing with local ductility and thus for a high energy dissipation before loss of stability. The left graph in Figure 2.13 shows an unsuitable plastic mechanism in a story frame, whereas the right graph shows a better suitable one. In addition to the higher number of activated plastic hinges, the local rotation ductilities as required for the same global ductility, and so the constructional measures at these places are much smaller.

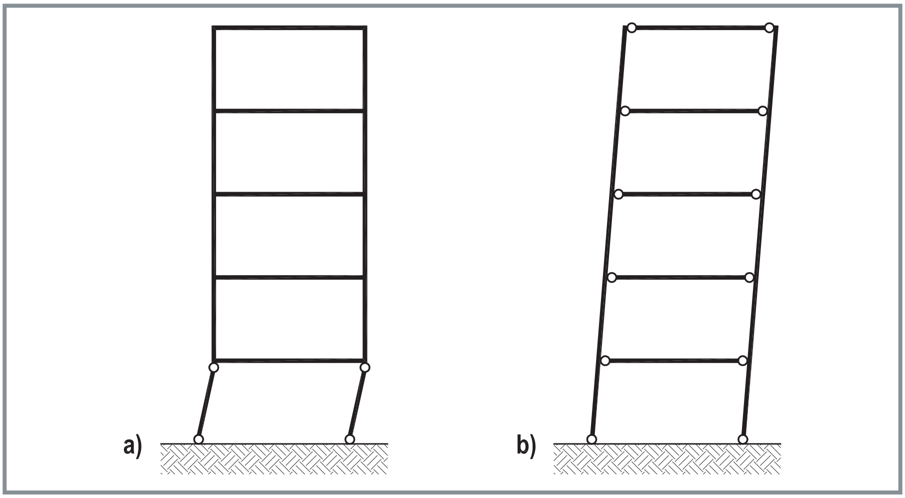


Figure 2.13 Plastic mechanisms of a frame under earthquake action
a) Unsuitable column mechanism b) Better suitable girder mechanism

The superiority of the capacity design concept described above can be emphasized by numerous experiments; please refer to the relevant literature (e. g. /1.1/, /2.4/, /2.5/).

There still is something to add with regard to the overstrength aspect or, more exactly, the "resistance with overstrength" R_0.

$$R_0 = S_E \cdot \Phi_0 \tag{2.12}$$

applies, with the design value of stress due to earthquakes SE and the overstrength factor Φ_0. The overstrength can result from:

- Reserves of the material strength
- Safety factors of the resistance side with the design conforming to standards
- Overdimensioning for constructional reasons
- Rearrangement reserves with statically undetermined systems.

Both literature and standards mention resulting overstrength factors Φ_0 of 1.3 to 2.2. The capacity design principle is increasingly accepted for earthquake standards. The regulations of New Zealand are entirely based on this principle, the ones in EC 8 and DIN 4149:2005 to a large extent. Older standards are still based on conventional design, only exploiting the natural global ductility via moderate behavior factors. However, it would easily be possible to carry out a capacity design on the basis of such standards as well. A development of all standards in the direction of capacity design can be expected.

2.3 Basics of Structural Dynamics

2.3.1 Introduction

Earthquake is a dynamic action. For the handling of dynamic actions analysis methods are necessary that are based on a dynamic principle, although simplified analysis methods for many standard cases are available. The application of such simplified analysis methods - and all the more of real dynamic methods -, however, requires adequate basic knowledge in the field of structural dynamics. Thus, following the basics as to seismology (Chapter 2.1) and building reactions (Chapter 2.2), also the most important basics as to structural dynamics have to be described.
In the following the field of structural dynamics is addressed only to an extent that an oscillator with one degree of freedom can be handled and as it is necessary for the understanding of the simplified methods and for the development of elementary checks. Multi-degree-of-freedom-systems and the specific questions and problems arising in this connection - especially those related to the so called spectrum method - are not addressed in this chapter, but are dealt with separately in Appendix A. Due to the lucid explanations also used in Appendix A even non-specialists should comprehend the basic principles and the most important terms.

By the way, what is "dynamics"? A simple explanation would be: Dynamics is a structural analysis under consideration of the time dependence of deformations and forces. This can be made clear by means of the Newtonian equations (2.13): the sum of all forces is equal to mass multiplied by acceleration. And when the accelerations can be neglected the basic static law remains: the sum of all forces acting in equilibrium must add to zero.

$$\sum \underline{F_i}(t) = \underline{M} \cdot \underline{\ddot{x}}(t) \qquad\qquad \sum \underline{F_i}(t) = \underline{0} \qquad (2.13)$$

Dynamics Statics

But, when are accelerations negligible? Any load depends on time! Experience teaches, however, that the term of inertia is negligible with loads being applied "slowly enough". Of course, the standard of comparison can not be an absolute duration of time, but must be a duration that is natural to the system: the natural period, the reciprocal of the natural frequency, as will be explained in the chapter to follow.
Consequently, the terms of inertia may be neglected and thus the systems may be calculated approximately by a static analysis when the following applies:

Period of load change >> Natural period of the system

or

Frequency content of the excitation << Natural frequencies of the structure.

An earthquake with excitation frequencies up to approx. 25 Hz thus leads to vibrations with structures with natural frequencies lower than 25 Hz. If the lowest natural frequency of the structure exceeds 25 Hz the load of inertia can be considered to be quasi-static and the system can be calculated by a static analysis.

Thus, frequency analyses of both the dynamic load and the structure serve to clarify whether and to which extent this load can excite the structure to vibrations. Consequently, also the requirements for a possible analysis model result. With regard to the earthquake this means that an analysis model should describe all natural frequencies up to approx. 25 Hz.

2.3.2 Single-Degree-of-Freedom-Systems (SDOFS)

2.3.2.1 Equation of Motion

Real systems possess an infinite number of degrees of freedom. In many cases however, it is possible to describe the essentials of the dynamic behavior by means of an equivalent model with only one single degree of freedom. This is a system where one motion or force quantity respectively is sufficient to clearly define the behavior.
Single-degree-of-freedom-systems present the basis of dynamics. They serve to study and comprehend the principal behavior and are essential for approximate calculations and checks. Moreover, it is possible to subdivide systems with *N* degrees of freedom into N systems with one single degree of freedom, just by a mathematical trick. Thus, the single-degree-of-freedom-system is an elementary part of big systems. For that reason it is considered and investigated in more detail.

The simple system shown in Figure 2.14 consists of a (rigid) mass *m*, the (massless, linear) spring of the stiffness *k* and a (linear) damper proportional to velocity of the stiffness *r*. All three quantities are equivalent quantities for a real structure, i. e. the result of a model idealization. Within the natural period an energy exchange occurs between mass and spring, with part of the energy being dissipated in the damper and transferred into heat. From the mathematical point of view the (viscous) damper proportional to velocity is especially comfortable. For that reason, other energy dissipation mechanisms are converted into viscous equivalent damping (criterion: identical energy dissipation per cycle) as far as possible.

For a better understanding it is useful to consider a force excitation *F(t)* at the mass and a displacement excitation *z(t)* at the bottom of the spring parallelly. The absolute displacement of the mass be *x*, the relative displacement between masses and base point be $y = x - z$. In addition, exactly two independent mathematical parameters can be derived from the three physical parameters *m, k*, and *r*:

$$\omega = \sqrt{\frac{k}{m}} \left[\frac{1}{s}\right] \qquad\qquad D = \frac{r}{2 \cdot \sqrt{k \cdot m}} [-] \qquad (2.14)$$

Natural (undamped) angular frequency   Damping factor

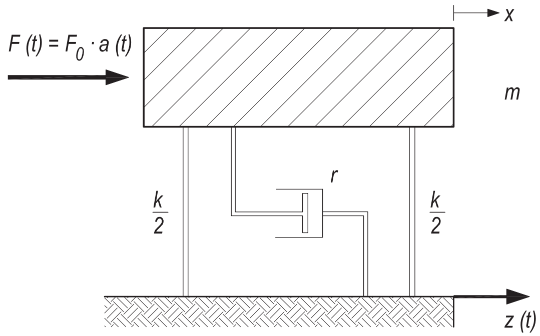


Figure 2.14 Single-degree-of-freedom-system

The Newtonian equations lead to the equations of motion:

Force excitation:

$$m\ddot{x} = F(t) - r \cdot \dot{x} - k \cdot x$$

$$m\ddot{x} + r \cdot \dot{x} + k \cdot x = F(t) = F_0 \cdot \alpha(t)$$

$$\ddot{x} + 2D\omega \cdot \dot{x} + \omega^2 \cdot x = \frac{F_0}{m} \cdot \alpha(t)$$

Base excitation:

$$m \cdot \ddot{x} = -r \cdot (\dot{x} - \dot{z}) - k \cdot (x - z)$$

$$m \cdot \ddot{y} + r \cdot \dot{y} + k \cdot y = -m \cdot \ddot{z}(t) = m \cdot a_o \cdot \beta(t)$$

$$\ddot{y} + 2D\omega \cdot \dot{y} + \omega^2 \cdot y = -a_o \cdot \beta(t) \qquad (2.15)$$

The dual form of the equations for force and base excitation can be recognized and it can be determined that the base excitation can be considered as force excitation with a load “mass multiplied by base acceleration”.

2.3.2.2 Free Vibrations

First it is useful to look at the free vibrations where the right side of the equations of motion (2.15) equals zero:

$$\ddot{x} + 2D\omega \cdot \dot{x} + \omega^2 \cdot x = 0 \qquad (2.16)$$

Of course, the equations for force and base excitation must be identical then. With an initial displacement x_0 (e. g. due to snap-back) and the initial velocity $\dot{x}_0$ (e. g. due to a shock/impulse) the solution results to:

$$x(t) = e^{-D\cdot\omega\cdot t}\left[x_0 \cdot \cos\omega_d \cdot t + \frac{1}{\omega_d} \cdot (\dot{x}_0 + x_0 \cdot D \cdot \omega)\sin\omega_d \cdot t\right]$$

$$\dot{x}(t) = e^{-D\cdot\omega\cdot t}\left[\dot{x}_0 \cdot \cos\omega_d \cdot t - \frac{1}{\omega_d} \cdot (x_0 \cdot \omega^2 - \dot{x}_0 \cdot D \cdot \omega)\sin\omega_d \cdot t\right] \qquad (2.17)$$

Figure 2.15 shows the free vibrations for D = 0.05 following an initial displacement.

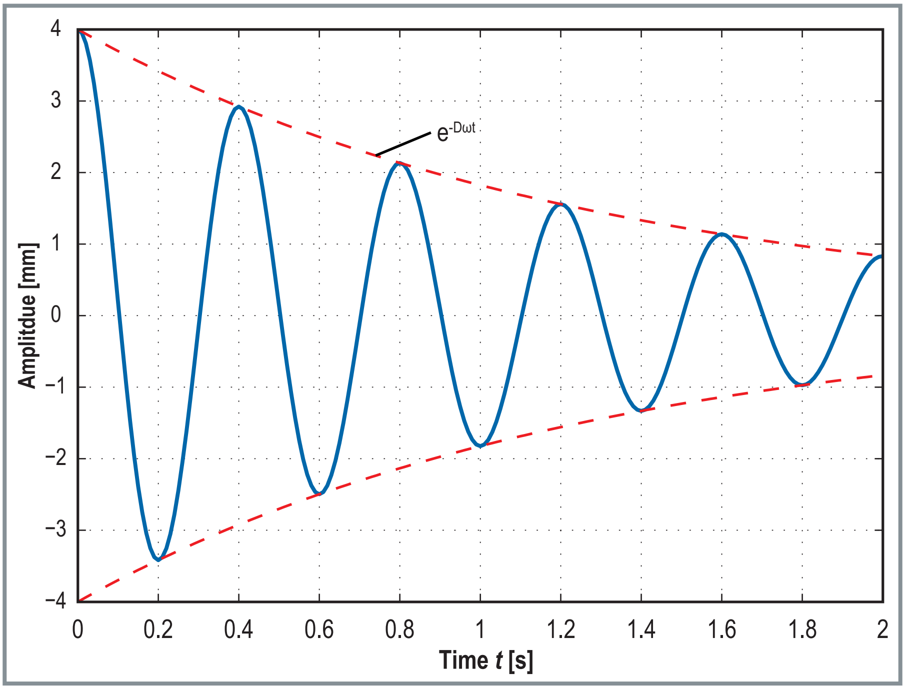


Figure 2.15 Free vibrations, one degree of freedom

These are vibrations whose amplitudes die out with an e-function. In addition to the parameters ω and D as indicated above, further parameters of the oscillator with one degree of freedom are necessary:

$f = \dfrac{\omega}{2 \cdot \pi}$	[Hz]	Natural frequency of the undamped system	
$T = \dfrac{1}{f}$	[s]	Vibration period of the undamped system	
$\omega_d = \omega \cdot \sqrt{1 - D^2}$	[1/s]	natural angular frequency of the damped system	(2.18)
$T_d = \dfrac{1}{f_d} = \dfrac{2\pi}{\omega_d}$	[s]	Vibration period of the damped system	
$\Theta = \dfrac{2\pi D}{\sqrt{1 - D^2}}$	[-]	Logarithmic decrement	

Obviously, ω_d becomes imaginary for $D > 1$; the vibrations change to creep functions without zero-crossings and are described by e-functions. D = 1 separates the vibrations from the creep processes. For that reason D = 1 is also called critical damping and e. g. D = 0.05 is described as 5 % of the critical damping. All the same, equations (2.17) remain real also for $D \geq 1$, for $D > 1$ the circular functions change to hyperbolic functions, a polynomial results for D = 1.

The differences between natural frequency and vibration period of the damped system and that of the undamped system can be neglected for small damping factors D, e. g. for D = 0.20 they are only 2 %. Especially with the application of numerical values taken from literature the difference between D and Θ is important, for small damping factors it amounts to a factor 2π.

Relative to the free vibrations also the interrelation between natural frequency and static deformation is of importance. On the basis of the definition of the frequency $f = \frac{1}{2 \cdot \pi} \cdot \sqrt{\frac{k}{m}}$ and of the consideration that the weight mg - better: the force of inertia from mass m x 1 g - produces the deformation Δ, with g = 981 cm/s² the simple equation

$$f \approx \frac{5}{\sqrt{\Delta[cm]}} \qquad \text{or} \qquad \Delta[cm] \approx \frac{25}{f^2} \tag{2.19}$$

results, which allows for quick approximate calculations and checks of natural frequencies by means of a static load case "unit acceleration 1 g in the respective direction". Thus, it is possible e. g. to get a feeling for the horizontal fundamental frequency of a structure when loading a static system, that has already been analyzed for wind loads, statically with horizontal unit acceleration 1 g. Table 2.3 shows some numerical examples for the interrelation (2.19).

Table 2.3 Interrelation between natural frequency and static deformation

Frequency / Hz	1	3	5	10	50
Deformation / cm	25	2.8	1	0.25	0.01

The numerical examples show that large deformations are related to low natural frequencies and that systems with high natural frequencies must possess a high stiffness. Equation (2.19), by the way, from the formal point of view is identical to that for a pendulum; the denominator shows the root of a length. But compared to the (small) elastic deformation here, with the pendulum it is a question of its (large) length. For that reason elastic frequencies exceed those of pendulums by one to two orders of magnitude.

And still something to add: Equation (2.19) exactly applies exclusively for the single-degree-of-freedom-system. If the mass is distributed continuously, as it is the case in the afore mentioned example of a building, and if the system thus has several degrees of freedom, then the equation needs to be modified:

$$f \approx \frac{5}{\sqrt{\frac{\Delta[cm]}{\Gamma}}} \qquad \text{or} \qquad \Delta[cm] \approx \frac{25 \cdot \Gamma}{f^2} \tag{2.20}$$

Here the value Δ represents the maximum deformation of the system. As will be shown in Appendix A, factor Γ represents the participation factor of the modal analysis. Of course, it is not known exactly in advance. It is known, however, that - with normalization of the eigenvectors to the maximum displacements - it generally lies between 1.3 and 1.7. And for most approximate calculations a medium factor of 1.5 is sufficient.

2.3.2.3 Forced Vibrations

If the right side of the equations (2.15) of motion does not equal zero then these are referred to as forced vibrations.

Although the parallel consideration of force and base excitation would be of further technical interest and could impart additional understanding, only the base excitation as relevant for earthquake shall be pursued in the framework of this book. According to (2.15), the equation of motion is:

$$\ddot{y} + 2 \cdot D \cdot \omega \cdot \dot{y} + \omega^2 \cdot y = -\ddot{z}(t) = -a_0 \cdot \beta(t) \quad \text{with} \quad y = x - z \tag{2.21}$$

with a_0 being the peak value (amplitude) of the ground acceleration and $\beta(t)$ being the time function normalized to 1. As shown in chapter 2, with the load case earthquake this is a transient function of (in Europe) 5 to 30 s of duration, consisting of a starting up phase, a strong motion phase and dying out phase. The main excitation occurs in the frequency range 1 to 25 Hz. The time signal is composed of individual frequency (spectral) components that differ in their phases. With regard to the frequency range of the main excitation the phases of the individual spectral components are stochastic (random) to a large extent.

Here, the analytical solution of the equation of motion (2.21), as it is possible for right sides that can easily be described mathematically (e. g. harmonious excitation, time function as a polygon), shall not be taken into further account. The numerical solution was already addressed in Chapter 2.1.3: When confining oneself to considering the absolute maximum of the responses within the resulting time histories then one gets the so called response spectra, and the result is represented by the **acceleration response spectrum S_a** as follows:

Relative displacement $\quad y_{max} \approx \dfrac{S_a(\omega,D)}{\omega^2}$

Internal force $\quad N_{max} \approx S_a(\omega,D) \cdot m \qquad (2.22)$

Absolute acceleration $\quad \ddot{x}_{max} = S_a(\omega,D)$

So the result is described by the response spectrum which includes the amplitude a_0 of the ground acceleration and which has already been elucidated in Chapter 2.1.3. Normalized to the amplitude 1 such a function is referred to as **Dynamic Load Factor (DLF)** which serves to indicate how much the dynamic result differs from the quasi-static one.

For three characteristic time histories of the base excitation

- Harmonious excitation with frequency Ω
- Transient excitation with an earthquake time history
- Triangular, symmetric time history of duration $t_{E,}$ *(shock excitation)*

Figures 2.16 a to 2.16 c show the acceleration response spectra normalized to 1 (i. e. strictly speaking the DLF). They are plotted for the earthquake time history over the frequency f / Hz, for the harmonious excitation over the ratio from natural frequency and exciter frequency $\xi = \omega / \Omega$ and with regard to the triangular function over the product from natural frequency and loading time $\zeta = f \cdot t_E$.
Obviously, the DLF or the response spectrum of all time functions respectively can be described in one diagram once these differ in one parameter only. This is e. g. the exciter frequency with harmonious excitation and the loading time with the triangular excitation. Such parameter then adds as factor or ratio in the abscissa. With the earthquake time history the same would apply if the different time histories only differed by a uniform extension of shortening of the time axis. This knowledge, of course, is not gained by numerical simulation, but only by analytical calculations or dimensional analyses.

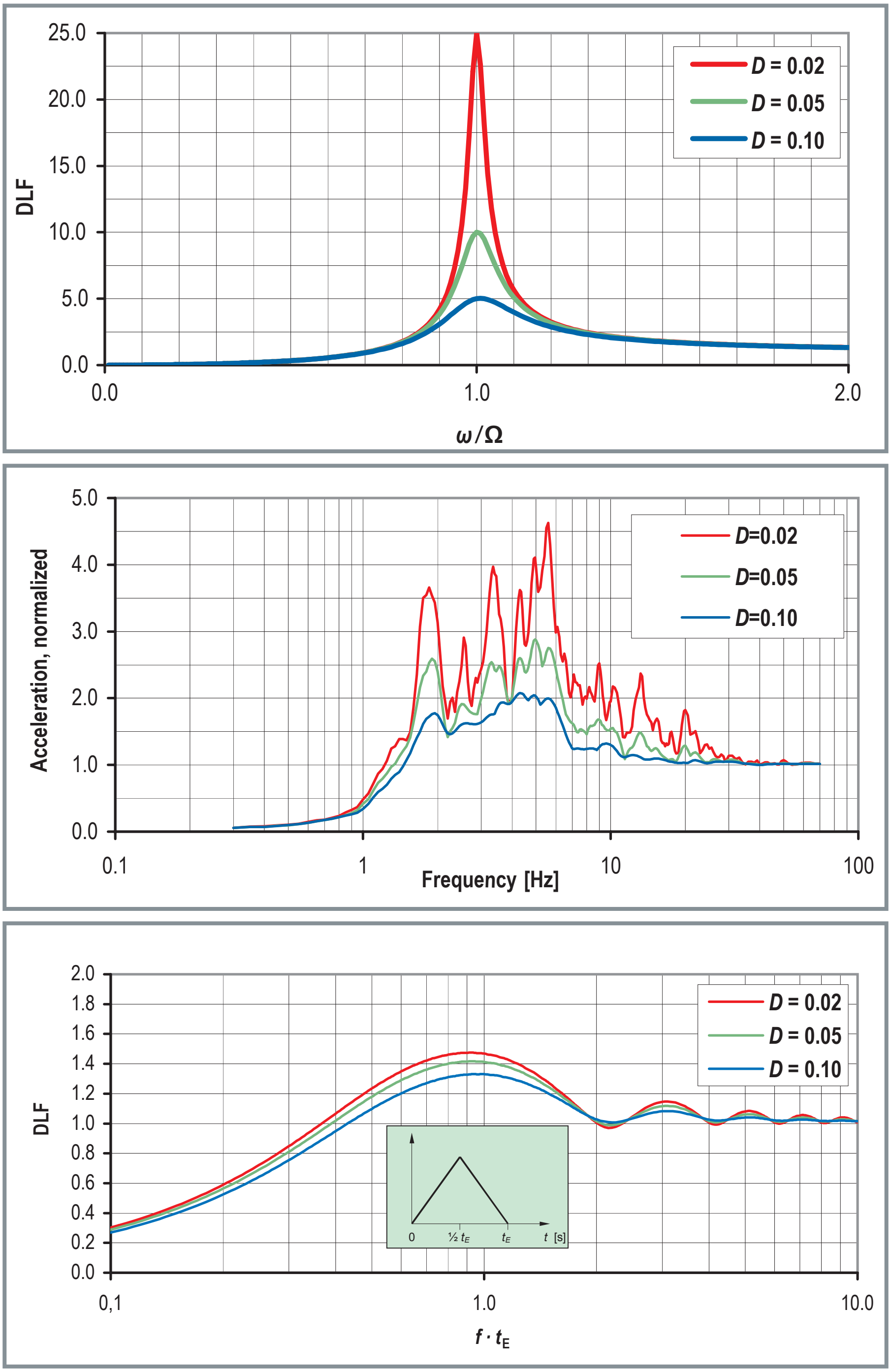


Figure 2.16 a: Acceleration response spectrum (DLF) for harmonious excitation
b: Acceleration response spectrum (normalized), earthquake excitation
c: Acceleration response spectrum (DLF) for shock excitation

The information from the figures can be summarized as follows (cf. Chapter 2.1.3):

- The spectra indicate the absolute maximum acceleration response of a single-degree-of-freedom-system to the relevant base excitation, as function of natural frequency and damping factor of that system and additionally depending on a characteristic parameter of the excitation, if required. (If the spectrum is normalized (DLF) it still must be multiplied by the ground acceleration.)
- In the low frequency range the curves approach 0, the excitation undergoes the flexible oscillator. In the higher frequency range the curves approach 1 (or the ground acceleration); the stiff oscillator follows the ground motion identically.
- In the medium frequency range the oscillator responds by resonances whose volume depends on the damping value. The highest resonance can be recognized with harmonious excitation and with diminishing damping here tends to infinity. The lowest resonance can be recognized with an impulsive excitation such as e. g. the triangular load. The transient excitation can be found in-between. With regard to the two last named the resonance remains finite also for diminishing damping.

For the earthquake engineer it is of main interest to understand and correctly apply the ground response spectra given in standards or by seismologic experts. By the broadening in direction of general structural dynamics this understanding shall be deepened.

2.3.3 Multi-Degree-of-Freedom-Systems (MDOFS)

The little excursion to the basics of structural dynamics come to an end now. In Appendix A, the extension to multi-degree-of-freedom-systems is included and shows that by a mathematical trick a system with N degrees of freedom can be subdivided into N systems with one single degree of freedom, so that the methods that have just been described also serve as basis for the solution of the motions of equation of multi-degree-of-freedom-systems.

Additional considerations, of course, are required as to the superposition of the individual modal solutions.

2.3.4 Equivalent Method, static Load Method

With simple structures such as e. g. buildings that are uniformly stiffened over the height, regular frames or girder grids it is not always necessary to carry out a dynamic analysis under consideration of many eigenmodes, as described in Appendix A. Here the essential information results from the fundamental mode with frequency f_0, the contribution of the higher modes can be registered by a factor ($1 \leq \kappa \leq 1.5$), the exact amount of which depends on the type of the structure. Thus, the real system is approximately reduced to a single-degree-of-freedom-system. By multiplication of the spectral value to the fundamental frequency f_0 by the factor κ an equivalent acceleration

$$a_{ers} = \kappa \cdot S_a(f_0, D) \tag{2.23}$$

develops, which can be assumed quasi-static and which leads to structural stresses of sufficient exactness. The technical background of this method consists in the similarity of the first vibration mode to the static deflection curve under the equivalent load. This is the substance of the simplified method or static load method which is offered by all seismic codes for the normal case. Only when dealing with special structures with responses that are characterized by higher modes more exact, dynamic analysis methods according to Appendix A (RSMA or THMA) are required. The conditions for the use of the static load method are explained in chapter 3.1.

3 Earthquake Design

3.1 Concepts

3.1.1 Basic Principles of the Aseismic Design

As already addressed in chapter 2.2.1, in countries without earthquake events structures primarily are designed to transfer vertical loads. It is only to a little extent (load case wind) that they are designed against horizontal loads. In countries with high seismic zones, however, the horizontal earthquake loads can reach the magnitude of the dead load, so that elements suitable for the systematic transfer of horizontal loads must be available.

As a consequence, the decisive step towards seismic design already consists in thinking of the transfer of horizontal loads from the purely constructional point of view. All further measures are improvements of this thinking by a more exact quantification of the additional loads (cf. Chapter 2.2.3), more exact calculations for the stress determination (cf. Chapter 2.3 and Appendix A), the optimization of the structure to that it will be capable to carry such additional loads (cf. Chapters 2.2.2/2.2.4) and optimization of the local constructional design in order to guarantee for the required ductility.

With regard to stress analyses, the static load method (cf. Chapter 2.3.3) can be used as simplest solution method, based on respective conditions. Dynamic analyses following the response spectrum method (RSMA, cf. Appendix A) or the time history method (THMA) provide higher exactitude. In special cases, but in particular in the field of research and development, non-linear analyses in the time domain are used which, with regard to the behavior during earthquake, attain even more verisimilitude.

The applicability of the simple static load method is always bound to the conditions that the structure shows "favorable" dynamic behavior. The following listing describes essential features of buildings which lead to a favorable dynamic behavior and therefore allow for a simplified analysis and design. In order to minimise the seismic risk these features should be realized in every structure in seismic zones as far as possible, independent of analysis. In any event, compared to the more exact analysis the seismic design always deserves priority attention.

With this in mind the most important features are:

- Choice of a simple, preferably rectangular layout shape in the ground plan, resolution of complicated layouts into individual simple shapes (cf. Figure 3.1).
- As far as possible, foundation on uniform level in identical soil layer, in addition, if necessary, partial deep foundation (cf. Figure 3.2), avoidance of ground with risk of liquefaction.

- As far as possible, foundation on continuous ground slab, at least connection of single and strip foundations by tie beams (cf. Figure 3.3).
- As far as possible, symmetric arrangement of the bracing components (walls, frames) in the layout in both directions (cf. Figure 3.4), so that stiffness and mass centers are as close as possible to each other.
- Choice of bracing components with ductile behavior and consequent constructional design, especially of the plastifying areas.
- Connection of the bracing components per story, by floor slabs acting as plates, as far as possible without level differences, at least arrangements of ring beams per story in which the floor beams are anchored.
- Constant or continuously decreasing stiffness over the structural height, as far as possible no steps or offsets, no soft story, especially with regard to the ground floor (cf. Figures 3.5 and 2.13).
- Constant or continuously decreasing weight per metre over the building height, no heavy roof, no inverted pendulum, i. e. no heavy top mass on a slender substructure (cf. Figure 3.6).
- Avoidance of heavy non-load bearing brick walls that can fall out with transverse accelerations, as far as possible choice of lightweight constructions for the non-load bearing partitions which are stable during earthquake in themselves, especially in the cross direction (cf. Chapter 3.1.2 and particularly chapter 4).
- Avoidance of short columns that can hardly plastify and tend to brittle failure (cf. Figure 3.7), instead arrangement of movement joints at the connections of the infills and/or replacement of the infills by lightweight constructions.
- Anchoring of all heavy building parts (façades, chimneys, gables etc.) and heavy equipment (components, machinery, shelves etc.) at the load bearing main structure (cf. Chapter 3.3.3).
- Planning of a sufficient distance between buildings, as sum of the earthquake deformation of both buildings in order to prevent pounding during earthquake.

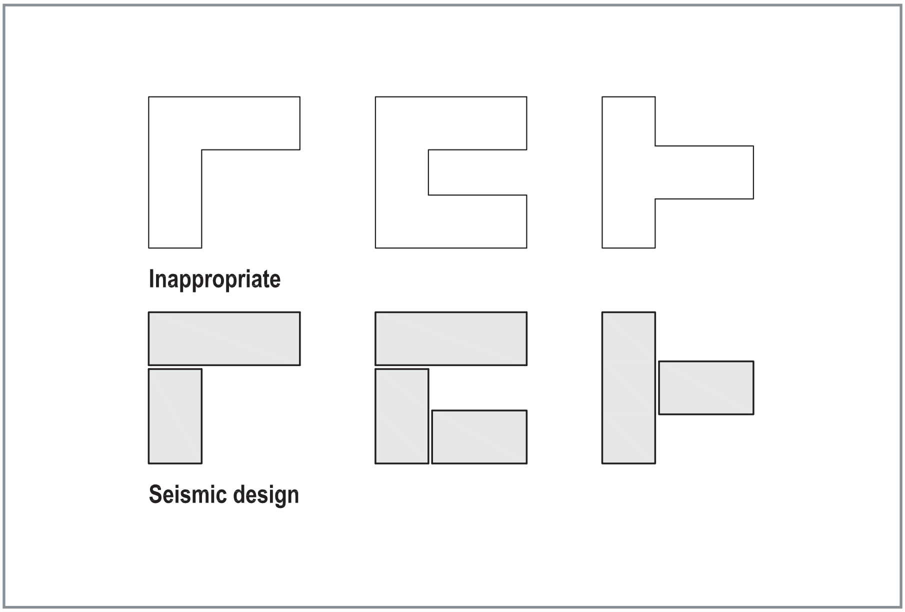


Figure 3.1 Layout shapes ground plan

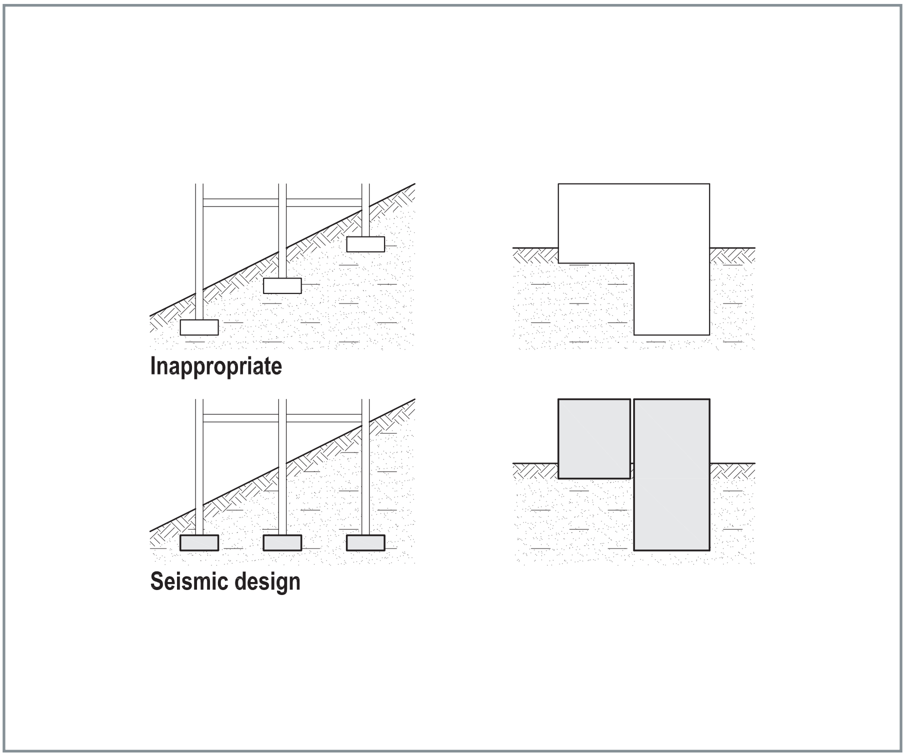


Figure 3.2 Groundings

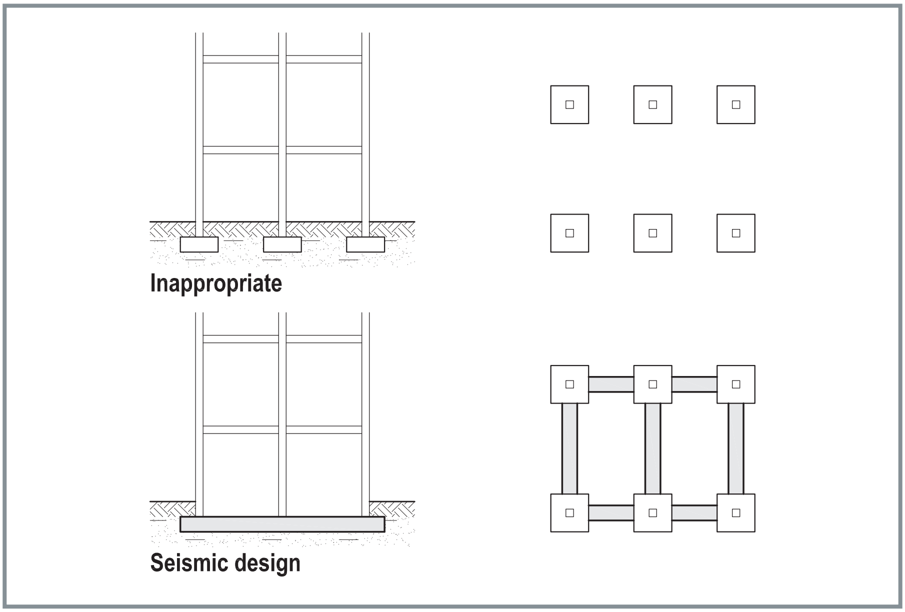


Figure 3.3 Foundations

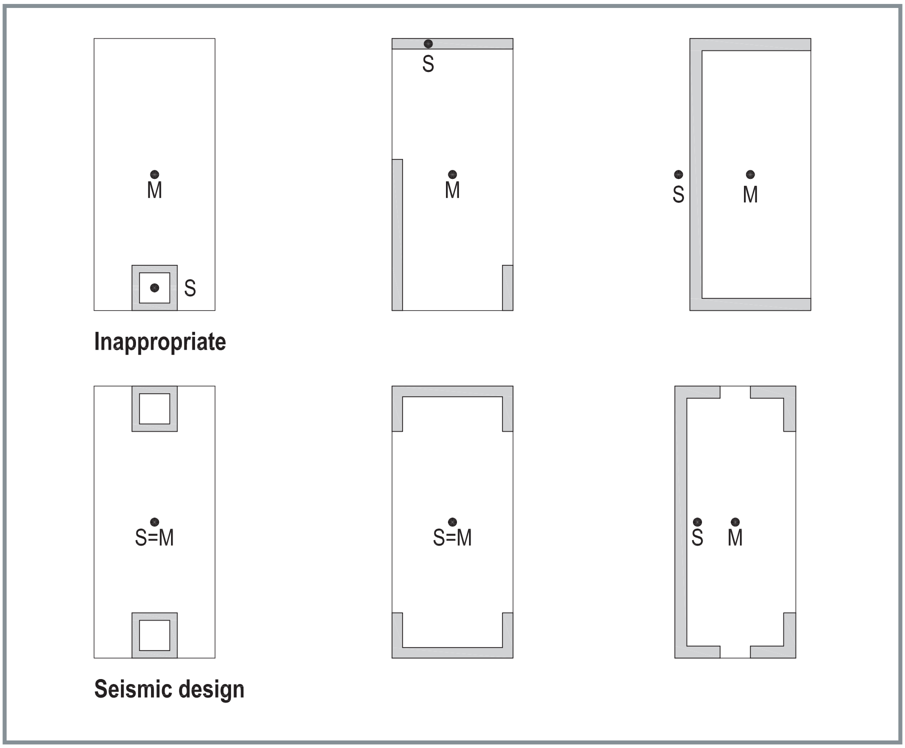


Figure 3.4 Stiffness distribution in the ground plan layout (M = mass center, S = stiffness center)

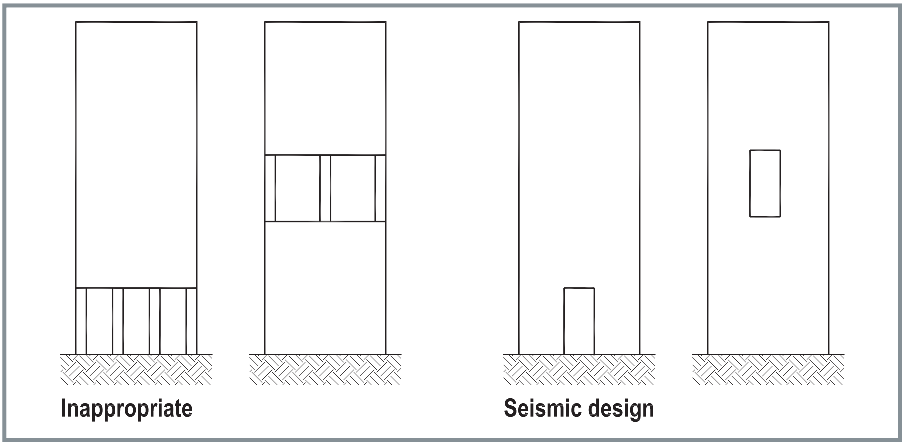


Figure 3.5 Stiffness distribution over the height (soft story)

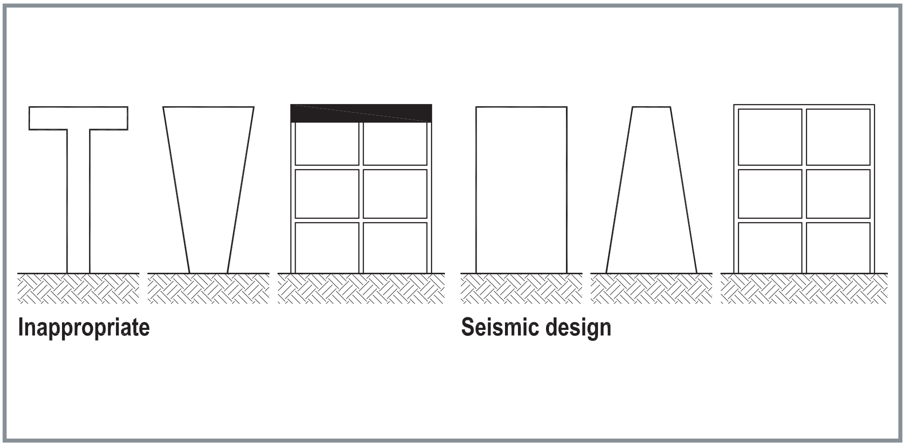


Figure 3.6 Mass distribution over the height

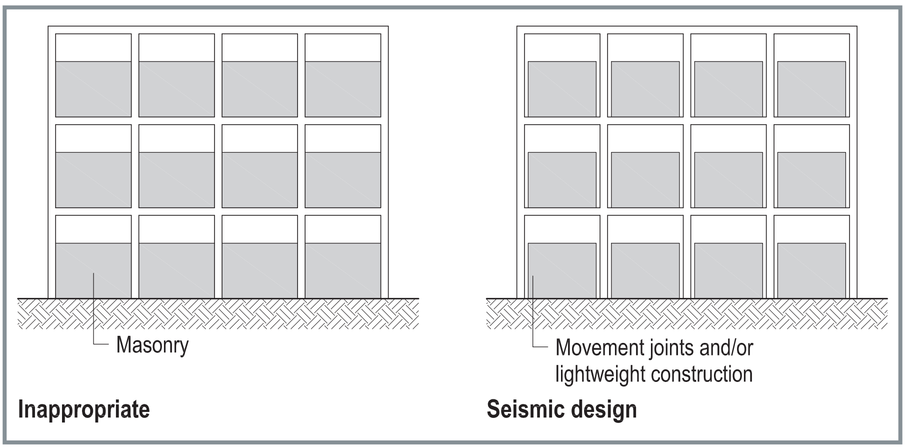


Figure 3.7 Columns and infill

The following photographs show typical earthquake damage (Figures 3.8 to 3.18) and impressively illustrate the cause of such damage: mostly not taking into account the afore mentioned features of a proper seismic design. These photographs only show some examples though which were picked for demonstration purposes; they do not represent a systematic, complete documentation of damage.

Figure 3.8 Lome Prieta Earthquake (USA), 1989 (Source: VHT Darmstadt)

Figure 3.8 shows the typical damage of an irregularly stiffened residential building including extensions with some constructional deficits of course.

Figure 3.9 Earthquake in Adana-Ceyhan (Turkey), 1998 (Source: BWG, Photographer: Pierino Lestuzzi)

In Figure 3.9 the unfavorable effect of an infilling masonry in connection with short columns can be recognized: The masonry is destroyed, the short columns do not possess the necessary ductility and shear off, which is not least due to their insufficient constructional design with too minor and wrongly designed stirrup reinforcement.

Figure 3.10 Earthquake in Bam (Iran), 2003, (Source: archive)

Figure 3.11 Earthquake in Izmit (Turkey), 1999 (Source: archive)

In Figure 3.10 the masonry has fallen out completely. The concrete skeleton remained, but the heavy ruins presented a great danger for the people.
Figure 3.11 shows details of a short column that has sheared off. In evidence: the insufficient constructional design.

Figure 3.12 Earthquake in Kobe (Japan), 1995 (Source: SGEB, Wenk)

The effect of a soft ground story becomes evident in Figure 3.12: Due to the garages, longitudinal stiffening is missing. The building barely remained, but the demolition is inevitable.

Figure 3.13 Lome Prieta Earthquake (USA), 1989 (Source: VHT Darmstadt)

In Figure 3.13, in contrast, it did not suffice any more. The ground storey just collapsed in it - with fatal consequences for people who stayed there.

Figure 3.14 Earthquake in Izmit (Turkey), 1999 (Source: archive)

In Figure 3.14 finally the entire building consisted of soft stories with heavy floor slabs and with unsymmetrical bracing in the layout additionally, resulting in a complete collapse of the building - a fatal trap to all residents.

Figure 3.15 Earthquake in Kobe (Japan), 1995 (Source: SGEB, Wenk)

Soft ground story in connection with unsymmetrical bracing caused the spectacular partial collapse of this commercial and office building in Figure 3.15.

Figure 3.16 Earthquake in Kobe (Japan), 1995 (Source: SGEB, Wenk)

In addition, a look into the building in Figure 3.16 shows the extent of damage done by earthquake to the infrastructure - here a computer center. In respect of the cost situation such damage today can possibly exceed structural damage.

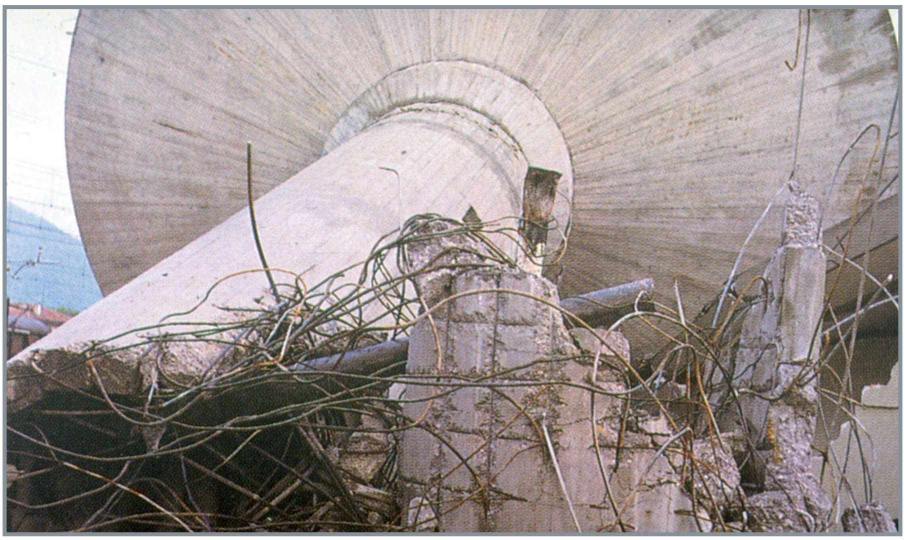


Figure 3.17 Earthquake in Friaul (Italy), 1976 (Source: archive)

As conclusion of this series of photographs there still are two special photos left: In Figure 3.17 there is the phenomenon of the inverted pendulum. The heavy top mass of the water tower would have required a strong and ductile design of the slender tower pole. But this and an appropriate constructional design were missing.

Figure 3.18   Earthquake in Kobe (Japan), 1995 (Source: SGEB, Wenk)

Figure 3.18 finally shows the damage done to a church. As per definition, the masonry construction with masonry vaults possesses infinitesimal ductility only. According to chapter 2.2.1 a high load bearing capacity would have been necessary here so that the church could have stayed stable. However, this was not the case.

3.1.2 Drywalling within Earthquake Engineering

In the area of earthquake engineering the significance of drywalling in substance can take three different effects:

1) Solely mass reduction. Since with earthquake engineering (base excitation) the loads of inertia to be transferred are proportional to the masses, the replacement of heavy non-load bearing walls by lightweight drywall constructions already leads to a valuable release. The loads thus reduced still are transferred exclusively by the skeleton made of reinforced concrete, steel or timber. By respective constructional design of the connecting joints for the lightweight walls, the relatively large deformations of this skeleton must be acceptable unconstrained to a large extent (sliding connections with tolerance), or the tolerated damage from restraints must be compatible with the personal safety protection objective respectively. The loads of inertia transverse to the lightweight walls of course must be carried by these walls and at the wall boundaries be released to the skeleton.
   Due to the small mass these loads are small, however, and with observation of elementary construction rules (cf. Chapter 4.2) easy transferable, needing only little analysis expenditure (cf. Appendix C).

2) Contribution to the bracing. Principally, lightweight walls also possess stiffness and load bearing capacity in direction of the component plane, so that with respective design they contribute to the horizontal bracing of the load bearing structure. With regard to the practical implementation, the stiffness of the various lightweight partitions, their load bearing capacities, the constructional measures as necessary for that purpose at a time, as well as the amount of achievable ductility and thus the behavior factors which can be assumed are required. This topic is still under research, the numerical values available are not yet sufficient.

3) Pure lightweight construction. In this case the entire load bearing system for the transfer of horizontal loads consists of lightweight structures. With regard to the practical realization, here - besides the necessary data of material and construction parts - concepts are required on how the respective analyses can be carried out efficiently and manageable by normal engineers. Research work is necessary either in this regard; adequate concepts for practical implementation are not available yet.

It is for the mentioned reasons that this book deals with the application of lightweight constructions in seismic zones mainly for the first field of application. The other fields of application are implementable as well, though with limitations. However, their potential cannot be fully exploited before the presentation of respective advanced research work.
Substantial part of the benefit is already attained in the first field of application, i. e. by solely mass reduction, whereas the gain achieved in the other fields of application is paid for by extra analysis expenditure.

3.2 Seismic Codes

3.2.1 Structure and Elements of Seismic Codes

Seismic codes vary from country to country. Therefore it is not useful to present seismic codes of individual countries in detail. Nevertheless, the basic structure of all modern seismic codes is comparable. This chapter serves to explain the systematic of a seismic code in order to facilitate its application to the users.

Modern seismic codes worldwide are structured following the same scheme:

- Definition and delimitation of the field of application, at the same time explicitly excluding defined special structures such as plants with increased secondary risk (e. g. structures as parts of nuclear plants, chemical plants, dam walls)
- Formulation of constructional basic requirements for the achievement of a favorable structural behavior during earthquake. In substance these are the favorable features as addressed in chapter 3.1.1, however, with priority grading depending on the individual codes
- Determination of the design parameters, i. e. zoning of the country into seismic zones, determination of the seismic forces in the form of elastic design spectra (cf. Chapter 2.1.3), depending on seismic zone, foundation soil and geological subsoil on site, definition of the analysis methods - simplified (quasi-static) or more exact (dynamic) analysis method, definition of the combinations of actions to be analyzed with definition of the partial safety factors
- Consideration of additional effects such as e. g. torsional effect in case of buildings with stiffness- and mass-centers per storey not coinciding. Influence of second order theory (P-Δ-effect), handling of non-load bearing structures and heavy internals (e. g. machinery, vessels)
- Additional constructional requirements in order to guarantee for the prerequisites for analysis, particularly with regard to the behavior factors assumed (cf. Chapter 2.2.3), i. e. measures for the achievement of the required ductility, general construction rules for usual materials (steel, reinforced concrete, masonry, timber).

The itemization above points out again that constructional measures - basic requirements (cf. Chapter 3.1.1) and additional requirements with regard to the constructional detailing - are at least coequal to the correct determination of seismic forces (cf. Chapter 2.2.4).

The determination of seismic forces will be deepened in the chapter to follow, substantiated by the example of EC 8.

3.2.2 Seismic Forces

The simplified static load method that all seismic codes offer as standard method generally can be described as follows: In a building in the *i-th* storey, the horizontal static equivalent load of the seismic action according to equation (3.1) to be assumed is proportional to the ground acceleration a_0 of the free field dependent on the decisive seismic zone (some codes still build the ground acceleration a_0 formally from ground acceleration and a seismic zone factor) and to the mass mi of the *i-th* storey. In addition, such horizontal static equivalent load is evaluated with a distribution function *f(z)* that depends on the mass distribution in the building and on the assumed shape of the deflection curve of the building over the height in the fundamental mode (cf. Figure 3.19).

The basic action so determined is now modified by up to five factors

$$F_i = a_0 \cdot (\alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5) \cdot m_i \cdot f(z_i) \tag{3.1}$$

with every factor taking a special effect into account. (Since the designations of the individual factors vary from code to code they are uniformly referred to as 'α' here. Also part of the terms varies; sometimes also the reciprocal of the respective factor is defined. These sophistications, however, should be easy to recognize in the individual code.):

- Importance factor α_1: The importance factor considers the importance of a structure to the general public. The lowest values apply to subordinated buildings such as barns and garages, whereas the highest values apply to public buildings (hospitals, emergency buildings) or event buildings (theatres, schools). Ordinary residential buildings are rated in-between. As a consequence of this factor, a structure is designed against a higher/lower and thus rarer/more frequent earthquake in order to compensate the possible larger/lower effects with the risk evaluation (cf. Chapter 1.3). In order to be more specific some few codes define an extra risk factor in addition to the importance factor. Such risk factor rather refers to the potential extent of damage. The effect on the design is the same; it is only a question of the product.
- Soil factor α_2: The soil factor considers both the change of the earthquake excitation caused by subsoil and foundation soil (cf. Chapter 2.1.1) as well as in part the effect of the foundation soil on the vibration responses of the structure. In modern codes the soil factor in this sense does no longer exist or only in part respectively. The second aspect has merged in the dynamic factor, i. e. in the free field spectrum dependent on the subsoil conditions (geological subsoil and local foundation soil). An additional factor for the first aspect can be available.
- Dynamic factor α_3: The dynamic factor considers the vibration behavior of the structure the simplest way possible. It represents the reading value of the free field spec-

trum at the fundamental frequency f_0 or period T_0 of the structure. Modern codes define the free field spectrum depending on the subsoil conditions, which means that the dynamic factor, i. e. the reading value of the free field spectrum as decisive for the respective subsoil conditions, partly includes the influence of the foundation soil.

- Damping factor α_4: The damping factor considers that with specific structures the effective damping may deviate from the standard damping D = 0.05 the free field spectrum is based on (e. g. steel lightweight construction, D < 0.05). The factor is mostly limited to values of 0.7 to 1.4. The damping factor is taken into account only if the elastic response spectrum is applied.
- Behavior factor α_5: The behavior factor considers the capacity of the construction to dissipate the induced seismic energy by non-linear plastic (ductile) behavior and thus sustain higher seismic forces than would result with linear consideration (cf. Chapter 2.2.3). With constructions with brittle failure the value is 1, with ductile constructions it can fall to 0.2 or even less. The terms mentioned above are often assigned also to the reciprocal of the factor defined here; i. e. the behavior factor in this sense then exceeds 1, and can be found in the denominator of equation (3.1)

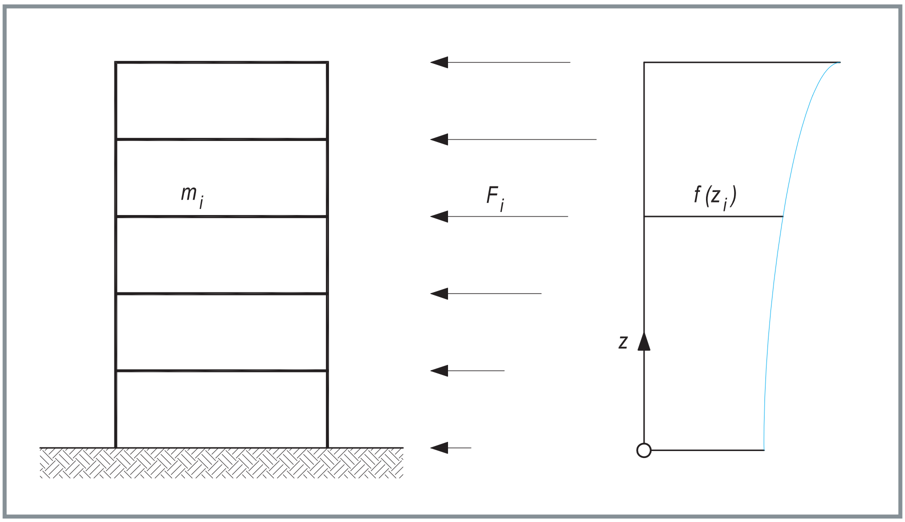


Figure 3.19 Seismic force according to equation (3.1)

The general formula as per equation (3.1) and related explanations shall now be exemplarily substantiated on the basis of Eurocode 8 (EC 8):

EC 8 defines the horizontal elastic acceleration spectrum $S_{ae}(T)$ as a function of the natural period T of a linear oscillator with one degree of freedom by the following formulae (cf. Figure 3.20):

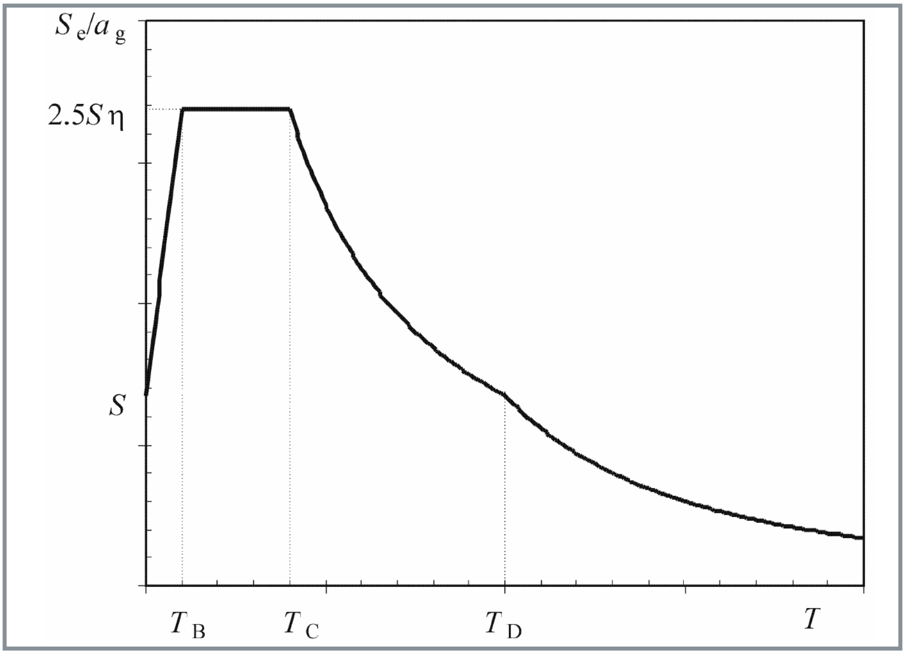


Figure 3.20 Shape of the elastic response spectrum according to EC 8

$$0 \le T \le T_B: \quad S_{ae}(T) = \gamma_I \cdot a_{gR} \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2.5 - 1)\right]$$

$$T_B \le T \le T_C: \ S_{ae}(T) = \gamma_I \cdot a_{gR} \cdot S \cdot \eta \cdot 2.5$$

$$T_C \le T \le T_D: S_{ae}(T) = \gamma_I \cdot a_{gR} \cdot S \cdot \eta \cdot 2.5 \cdot \left[\frac{T_C}{T}\right] \qquad \text{(3.2 a-d)}$$

$$T_D \le T \le 4s: S_{ae}(T) = \gamma_I \cdot a_{gR} \cdot S \cdot \eta \cdot 2.5 \cdot \left[\frac{T_C T_D}{T^2}\right]$$

With:

$S_{ae}(T)$	[m/s²]	Ordinate of the elastic acceleration response spectrum
γ_I	[-]	Importance factor. The structures are classified into four categories of importance, for which values of 0.8, 1.0, 1.2 and 1.4 are suggested. The exact values are defined nationally.
a_{gR}	[m/s²]	Reference ground acceleration for soil class A according to national definition
S	[-]	Soil parameter reflecting the amplification of the ground acceleration depending on the soil structure (cf. Chapter 2.1.1). With regard to soil class A, S = 1.0 applies. With regard to the other soil classes values up to 1.4 (response spectrum type 1; see below) or up to 1.8 (response spectrum type 2) respectively are suggested for S. The exact values must be defined nationally. In a simplified manner Table 3.1 shows the definitions of the soil classes according to EC 8 for common types of soil.
η	[-]	Damping correction coefficient with value 1 for 5 % viscous damping. For other damping values $\eta=\sqrt{10/(5+\xi)}\geq 0.55$ with ξ as viscous damping factor expressed in per cent.
T	[s]	Natural period of a linear oscillator with one degree of freedom as abscissa
T_B	[s]	Lower limit of the range of constant spectral acceleration
T_C	[s]	Upper limit of the range of constant spectral acceleration
T_D	[s]	Beginning of range of constant displacements of the spectrum

Table 3.1 Soil classes according to EC 8 (simplified, abstract)

Soil Class	**Stratigraphic Profile**	**Shear Wave Velocity $v_{s,30}$**
A	Rock or other rock-like geological formation, with a maximum of 5 m of weaker material at the surface	> 800
B	Sediments of over-consolidated sand, gravel or very stiff clay, with a thickness of up to several tens of meters, characterized by a gradual increase of mechanical properties with depth	360 - 800
C	Deep sediments of dense or medium-dense sand, gravel or stiff clay, with thicknesses of several tens to many hundreds of meters	180 - 360
D	Sediments of loose to medium-dense cohesion-less soil (with or without some weak cohesive layers), or of predominant weak to stiff cohesive soil	< 180
E	A soil profile consisting of a surface alluvial layer with v_s-values according to C or D and varying thickness between approx. 5 m and 20 m about stiffer soil material with v_s > 800 m/s	

For the cutoff periods T_B, T_C, T_D EC 8, depending on the soil class, suggests values, again distinguishing between two types of spectra: type 1 - normal case, and type 2 - earthquake with a surface magnitude $M_s \leq 5.5$. As already addressed in Chapter 2.1.3, with an earthquake of small magnitude the excitation maximum is shifted to lower natural periods (higher frequencies) with increasing spectral amplitudes. This fact is taken into account by the differentiation into the afore mentioned two types of spectra. The exact numerical values for the cutoff periods must be defined nationally, so that the respective tables and curves are not quoted herein.

Figure 3.21 shows the elastic response spectrum type 1 for the five soil classes of Table 3.1 with the values as currently recommended in EC 8 for S, T_A, T_B *and* T_C as well as for $\gamma_I \cdot a_{gR} = 1.0$, which - for reason of better reading - are plotted logarithmically over the natural period T.

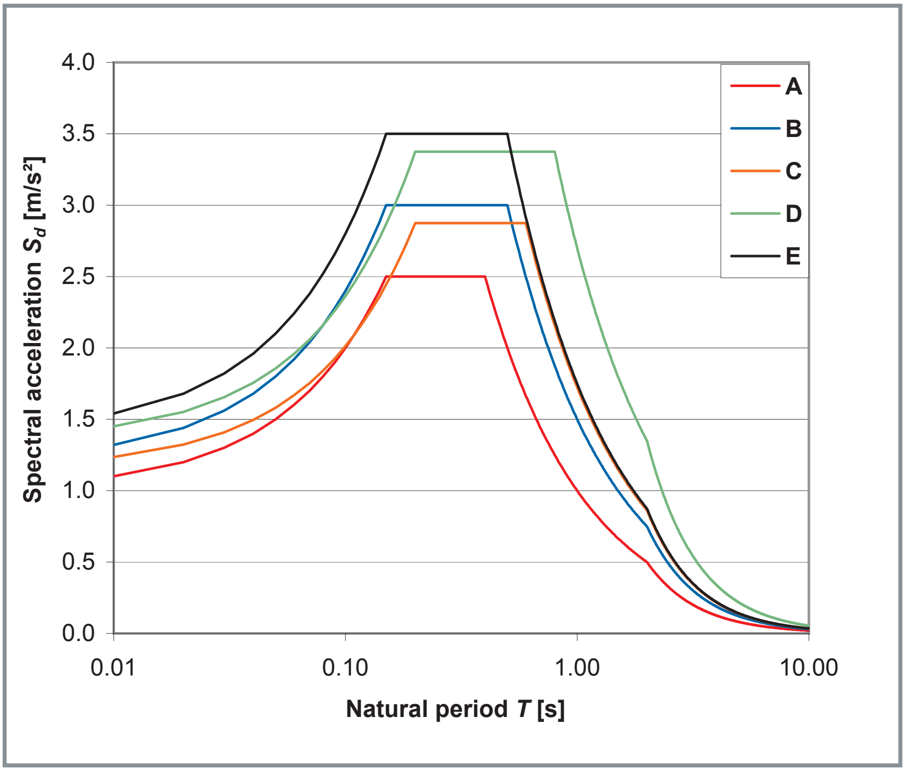


Figure 3.21 Elastic response spectra according to EC 8 for different soil classes

The vertical elastic response spectrum is described by formulae analogous to (3.2 a-d), differing, however, in the soil parameter S being set to 1.0, i. e. the local increase of the amplitudes caused by the layers overlying the rock is neglected. The spectral resonance factor is increased from 2.5 to 3.0 instead. This simplification is due to the minor priority of the vertical excitation (cf. Chapter 2.2.1). Furthermore, the vertical seismic forces must only be pursued when dealing with special constructions (e. g. large spanned constructions or girders bearing columns).

Here the two types of spectra are differentiated as well, whereas EC 8 makes suggestions for the ratio between vertical and horizontal basic acceleration as well as for the cutoff periods, the more exact numerical values are subject to the national definition.

If the non-linear behavior of the structure shall or may be considered, a design spectrum $S_{ad}(T)$ as a function of the natural period T must be generated from the elastic response spectrum:

$$0 \le T \le T_B: \quad S_{ad}(T) = \gamma_I \cdot a_{gR} \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \le T \le T_C: S_{ad}(T) = \gamma_I \cdot a_{gR} \cdot S \cdot \frac{2.5}{q} \qquad (3.3\ a\text{-}d)$$

$$T_C \le T \le T_D: S_{ad}(T) = \gamma_I \cdot a_{gR} \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C}{T} \right] \ge \beta \cdot \gamma_I \cdot a_{gR}$$

$$T_D \le T: \quad S_{ad}(T) = \gamma_I \cdot a_{gR} \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C T_D}{T^2} \right] \ge \beta \cdot \gamma_I \cdot a_{gR}$$

Compared to (3.2 a-d) the modifications first consist in the damping factor η being dropped, since compared to the energy dissipation from non-linear behavior this factor is no longer important. Secondly the amplitudes are reduced by the behavior factor q, as explained in Chapter 2.2.3. The permissible values of q are defined in the building material specific chapters of EC 8, dependent on the type of load bearing structure. They amount to 1.0 with brittle material or particularly unfavorable constructions via 1.5 as normal case without further analyses up to 5.0+ with particularly ductile steel constructions.

The factor β finally shall avoid the number falling below a minimum value. For this purpose EC 8 recommends 0.2, the exact value, however, is subject to national definition.

The summand 2/3 in (3.3 a) that replaces 1 in (3.2 a) is yet to be observed: It leads to a reduction of the rigid body acceleration by a factor 1.5, which can only be explained by the special manner of defining the spectra - linear development below T_B - which results in too high spectral values in the area of small periods.

Since a dissipative behavior in the vertical direction is hardly expectable, the behavior factor q for the vertical seismic excitation shall be assumed to be 1.0 or 1.5 at a maximum. In other respects the modifications apply as described on the basis of the horizontal seismic excitation.

Formulae (3.2) and (3.3) indicate the equivalent acceleration. For the simplified method a multiplication by the masses leads to forces which are distributed to the individual stories of the structure by means of a function $f(z_i)$ according to (3.1). When as a first approximation assuming a linear deformation of the structure over the height this leads to:

$$f(z_i) = \frac{z_i \cdot m_i}{\sum_j z_j \cdot m_j} \tag{3.4}$$

with m_i and m_j representing the masses of the i-th or j-th story respectively and z_i and z_j the altitudes of the individual stories above foundation level. Other assumptions of deformation lead to modified distribution functions.

As can be seen, EC 8 presents a very detailed code which goes beyond the simple description as per (3.1) already in the definition of the earthquake effect, but nonetheless (3.1) is helpful for the basic understanding.

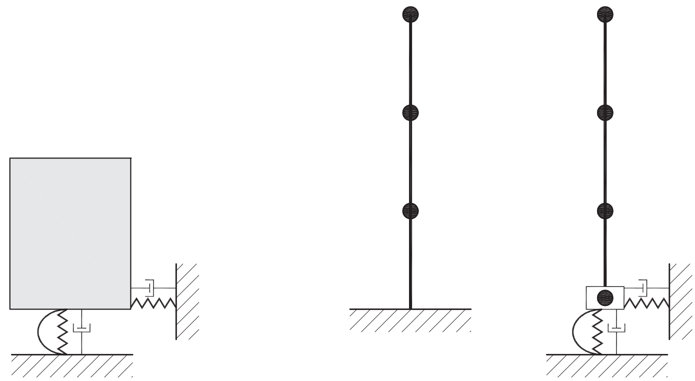
3.3 Analysis and Design of Buildings for Earthquake Loads

3.3.1 Mathematical Models

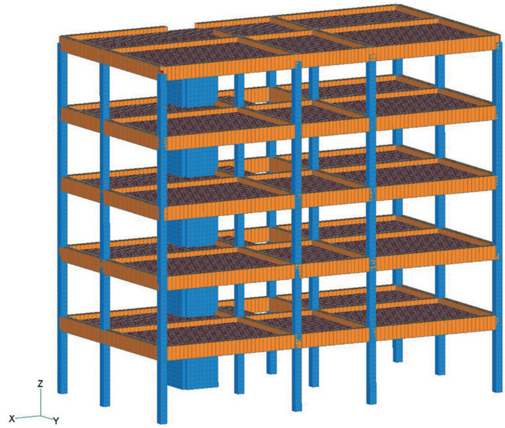
Once the requirements for the application of the simplified static load method (cf. Chapters 3.1.1, 3.2.2) are fulfilled and if this method is used, the determined equivalent forces must be applied as horizontal impacts, and pursued in the load transfer analogous to the wind loads. An upright beam which is supported on the foundation soil, or a frame/plate system on the individual elements of which the loads have to be distributed are used as analysis models. In that case dynamics is reduced to the assessment of a fundamental frequency or period of the structure, so that the value applicable for the dynamic factor (cf. Chapter 3.2.2) can be read from the free field spectrum of the site. When using the maximum value of the spectrum instead, even this residual amount of dynamics is dropped.

If the requirements are not fulfilled or if just a more exact analysis is desired, a dynamic analysis following the spectrum method or the time history method according to Appendix A is to be considered. For this purpose, a dynamic analysis model is necessary, which takes into account the stiffness and mass distribution of the structure and, if necessary, of the foundation soil (cf. Chapter 3.3.2) appropriately. It must allow for the presentation of all relevant natural frequencies of the structure up to the limiting frequency or limiting vibration period of the free field spectrum respectively, i. e. approx. $f \leq 25$ Hz or $T \geq 0.04$ s. From the engineer's point of view one should give preference to the simple and clear model rather than using the more sophisticated one.

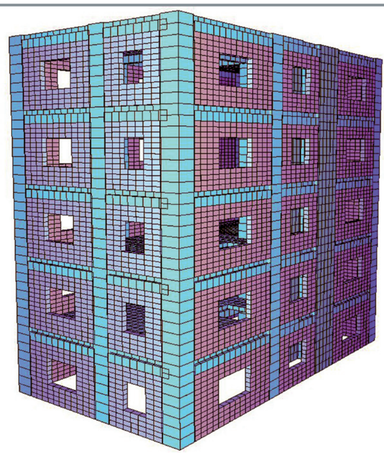
Typical models are shown in Figure 3.22, beginning with the rigid one-mass-oscillator on elastic foundation soil with up to six degrees of freedom in space (a) such as it is considered for compact, bunker-type structures, via plane or spatial beams with rigid of elastic clamping in the ground (b), or respective frame/plate systems (c) until spatial shell models (d) with or without consideration of the soil-structure-interaction (cf. Chapter 3.3.2). An essential facilitation results if the individual stiffening elements per storey are connected by a floor slab which is stiff (rigid) in the plane of the plate. This leads to a reduction of the degrees of freedom per storey to the both horizontal displacements and the torsion around the vertical axis.



a) One-mass-oscillator b) Beam model without/with elastic foundation soil



c)) Spatial frame model



d) Spatial shell model

Figure 3.22 Examples for models for the dynamic analysis of the load case earthquake

The masses are mostly combined by generating a lumped mass model - either by the analyst in the course of the data determination or by the software automatically. The point masses should reflect the spatial mass distribution appropriately. They primarily contain the masses from dead load of the construction and permanent live loads as well as from fixed internals. Impermanent live loads are not considered at all or only to a little percentage in the vibration model, depending on the code.

As a rule, in linear analyses static stiffnesses are applied as dynamic stiffnesses. With regard to reinforced concrete this means the stiffnesses in the uncracked state. The reduction in stiffness caused by crack formation, however, should be taken into account within the scope of sensitivity analyses. This does not only apply to the stiffnesses of the structure, but also of the foundation soil (cf. Chapter 3.2.2): Since dynamics does not imply a safe side readily, regarding the parameters as relevant for the result both an upper and a lower limit should always be considered. In special cases it can also be helpful to use two models, e. g. for sensitivity analysis or quality assurance purposes (see below), with different degree of detailing.

By the design spectrum the damping is defined to $D = 0.05$ on average. For structures with very small damping, e. g. steel lightweight constructions, the codes provide a damping factor > 1, taking this into account. If the soil-structure interaction (cf. Chapter 3.3.2) is explicitly considered, the increased damping from energy radiation can be converted to a damping factor < 1. Fundamental idea in this respect is a weighting of the damping values of foundation soil and structure corresponding to their contribution to the deformation energy of the respective natural mode.

For the generation of analysis models and thus for dynamic structural analyses a large number of program systems is available internationally. These can be general-purpose finite-element programs offering options appropriate for earthquake engineering, or specialized finite-element programs especially developed for earthquake analysis. Within the framework of this book specific software packages are not addressed. Such software may be used only by experts anyway, who must check it in respect of the options as required for the idealization of the relevant national code and for their work at a time - and have the determined results undergo quality assurance.

3.3.2 Soil-Structure Interaction

The foundation soil does not only provide a (static) support for the structure, but - due to the soil-structure interaction - more or less clearly influences the vibration behavior of the same during earthquake as well. It has minor influence with in relation to the foundation soil weak structures as e. g. buildings stiffened by frames. The influence increases with structures stiffened by plates and dominates with compact reinforced concrete structures with cellular design.

In view of the above the foundation soil has minor influence with ordinary residential building. For that reason, with dynamic analyses, the foundation soil is usually neglected and the building or its bracing components respectively are assumed to be rigidly supported in the foundation soil. This leads to natural frequencies that are slightly too high, which is acceptable as long as the significant natural frequencies increase parallel to the free field spectrum or lie in the plateau of the same. In order to estimate the influence of the foundation soil better- above all with regard to special buildings - the following aids are given:

The foundation soil can be approached as layered half-space which is homogeneous in special cases. Its decisive parameters - dynamic modulus of shear G, density ρ and Poisson's ratio ν per layer - have already been defined in Chapter 2.1.1. An exact analysis, which due to the frequency dependence of the foundation soil characteristics must be carried out complexly, requires the specialist. Such exact analysis is required only rarely though - but particularly when dealing with buildings of nuclear plants. With ordinary building it is sufficient - if ever required - to consider the foundation soil approximately. In the 1970s it became possible to idealize the soil below a rigid circular foundation as a first approximation by a spring-damper system, Figure 3.23. This model allows for the reproduction of the two main characteristics of the soil - flexibility or stiffness respectively due to the elasticity of the soil material and energy dissipation due to wave radiation into the infinite half-space. For the six degrees of freedom of the motion Table 3.2 indicates equivalent stiffnesses and equivalent damping factors of the homogeneous half-space below a rigid circular foundation.

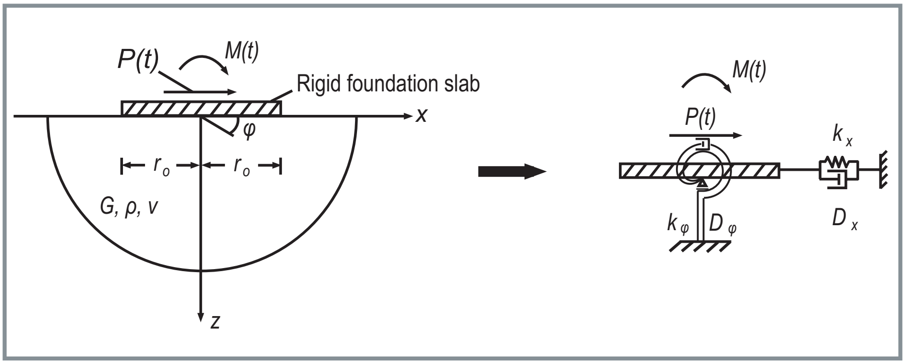


Figure 3.23 Simple solution for the homogeneous half-space

Table 3.2 Equivalent stiffnesses and equivalent damping factors of the homogeneous half-space below rigid circular foundation

Direction of Vibration	Spring Constant	Damping Factor	Mass Ratio
Vertical	$k_z = \frac{4 \cdot G \cdot r_0}{1-\nu}$	$D_z = \frac{0.425}{\sqrt{B_z}}$	$B_z = \frac{(1-\nu)}{4} \cdot \frac{m}{\rho \cdot r_0^3}$
Horizontal	$k_x = \frac{32 \cdot (1-\nu) \cdot G \cdot r_0}{7-8\nu}$	$D_x = \frac{0.288}{\sqrt{B_x}}$	$B_x = \frac{(7-8\nu)}{32 \cdot (1-\nu)} \cdot \frac{m}{\rho \cdot r_0^3}$
Torsion around the horizontal	$k_\varphi = \frac{8 \cdot G \cdot r_0^3}{3 \cdot (1-\nu)}$	$D_\varphi = \frac{0.15}{(1+B_\varphi) \cdot \sqrt{B_\varphi}}$	$B_\varphi = \frac{3 \cdot (1-\nu)}{8} \cdot \frac{I_\varphi}{\rho \cdot r_0^5}$
Torsion around the vertical	$k_T = \frac{16}{3} \cdot G \cdot r_0^3$	$D_T = \frac{0.50}{1+2B_T}$	$B_T = \frac{I_T}{\rho \cdot r_0^5}$

With

G	[kN/m²]	Dynamic modulus of shear of the soil
ν	[-]	Poisson's ratio of the soil
ρ	[t/m³]	Density of the soil
r_0	[m]	Radius of the rigid foundation
m	[t]	Vibrating mass of the building, in case of stiff building the total mass of the building
I	[tm²]	Mass moment of inertia of *m* around the axis of the foundation

Mass ratio *B* controls damping factor *D*. The smaller the vibrating mass of the building in relation to the effective mass of the soil, the greater the damping factor. Since the dynamic modulus of shear of the soil *G* depends on the strain of the soil, the values for earthquake are lower than e. g. those for operating vibrations with small amplitudes.

Most foundations, of course, are not circular. But it is possible to calculate an equivalent circle from the real foundation area. Such an equivalent circle results from area comparison for vertical and horizontal springs and from comparison of the moments of inertia of the foundation around the respective axis for the rotational springs. This provides sufficient exactitude in view of the fact that - even with careful analysis - the most important parameter, namely the dynamic modulus of shear of the soil *G*, holds an uncertainty which at least amounts to a factor 2. Hence an upper and a lower limit must be considered with the analysis anyway, at the same time taking into account the uncertainty regarding the idealization of the soil as well as other parameter uncertainties.

For the sake of completeness, it has yet to be pointed out that every site must be analyzed as to the exposure of the foundation soil to liquefaction. This risk exists with very homogeneous, water-saturated soils that can lose their load bearing capacity under earthquake vibrations, due to the increase of the pore water pressure.

3.3.3 Results and Design

If a dynamic analysis method was chosen the natural frequencies and natural modes as well as the modal parameters of the model are determined primarily. This is also helpful if direct integration or non-linear analyses shall be carried out later on; it facilitates the understanding of the dynamic behavior of the model and guarantees the appropriateness of the same. Furthermore, when using more demanding models it is recommended to carry out a static analysis with horizontal unit acceleration beforehand, in order to check the model generation and later separate dynamic effects from static ones. If the static load method was chosen the static analysis according to Chapter 3.2.2 provides the decisive internal forces directly.

With regard to the dynamic analysis method, the analysis of the forced vibrations, e. g. following the response spectrum modal analysis method (cf. Appendix A as to natural frequencies, modal superposition and determination of the rigid-body term to be considered) leads to the dynamic internal forces of the analysis model. If the model was simplified to a large extent compared to the real structure, the internal forces of the model are yet to be transferred or distributed respectively to the real stiffening elements.

The internal forces from the load case earthquake must then be superpositioned, according to the prevailing code with those from dead load and other live loads, includ-

ing the required partial safety factors, and must be compared to the related ultimate loads depending on the material. The design of construction parts as well as of anchorage and connection devices and with regard to reinforced concrete the amount of reinforcement are derived therefrom.
In order to guarantee the favorable structural behavior desired during earthquake the structural elements that must continue to behave linear during earthquake (e. g. the columns of frames) must be provided with overstrength according to the capacity design method (cf. Chapter 2.2.4) so that only those areas planned will plastify (e. g. in the girders). In connection with an appropriate constructional design according to the code this leads to a favorable mechanism without sudden failure.

In addition to the design the deformations in the individual stories must be determined, aimed at the assessment of

- the remaining degree of serviceability
- the distance to adjacent buildings, in order to prevent pounding
- the effect on non-load bearing walls, especially lightweight walls, in order to avoid incompatible constraints
- the effect on important infrastructure, e. g. supply and waste lines.

If the result is not satisfactory because of technical or economical aspects, it is possible to look for optimizations. These can be done by

- the basic design, i. e. type and arrangement of stiffening elements (cf. Chapter 3.1.1) and choice of the plastic mechanism
- the earthquake analysis, i. e. detailing of analysis model and analysis method (cf. Appendix A)
- the design, e. g. more or less strength with less or more deformation (cf. Chapter 2.2.1).

In any case, rather than the exploitation of the computational feasibilities the seismic design deserves priority.

3.3.4 Components and Equipment

With the exception of plants with increased secondary risk, exterior components at buildings (e. g. chimneys, facade plates, canopies), or equipment in buildings (e. g. vessels, piping systems, pumps) usually do not need to be designed against earthquake themselves, but should be anchored at the building in order to limit the danger caused to people by flying parts during earthquake. This principally applies to components that can come down outside, whereas for components in buildings this normally applies only when a certain mass is exceeded. The Uniform Building Code (UBC) indicates a value of 181 kg (400 lbs), in many codes such information (or the requirement of the anchoring of components respectively) is completely missing. A value of 200 kg seems to be principally reasonable.

For the design of anchorage construction and anchoring devices respective equivalent loads are required. The formulae as applied for this purpose by UBC and EC 8 are given as equations (3.5)/(3.6):

$$\text{UBC:}\quad F_P = \frac{W_P}{g}\cdot\frac{I_P}{R_P}\cdot(C_a\cdot g)\cdot\left(1+3\cdot\frac{h_x}{h_r}\right)\cdot a_p \quad \begin{matrix} \geq \dfrac{W_P}{g}\cdot I_p\cdot(C_a\cdot g)\cdot 0.7 \\ \leq \dfrac{W_P}{g}\cdot I_p\cdot(C_a\cdot g)\cdot 4.0 \end{matrix} \tag{3.5}$$

$$\text{EC 8:}\quad F_a = \frac{W_a}{g}\cdot\frac{\gamma_a}{q_a}\cdot a_g\cdot S\cdot\left[\frac{3\left(1+z/h\right)}{1+\left(1-T_a/T_1\right)^2}-0.5\right] \tag{3.6}$$

The designations have been taken from the prevailing codes, where:

F_a resp. F_p	Equivalent force, to be applied in the center of gravity of the component for the design of their anchorage
W_a/g resp. W_p/g	Mass of component
I_p resp. γ_a	Importance factor of component, in most cases identical to that of the related structure; higher in case of increased secondary hazard (toxic or explosive agents)
R_p resp. q_a	Behavior factor of component
$(C_a \cdot g)$ resp. a_g	Ground acceleration on site
S	Soil factor (Maximum amplification of the free field spectrum compared to ground acceleration)

a_p	Vibration amplification of component compared to structure
h_x/h_r resp. z/H	Erection height of component above foundation level, referring to height of roof
T_a/T_1	Vibration period of component, referring to basic vibration period of the structure

Equation (3.5) (UBC) superproportionately emphasizes the building height, but does not specify any analytical frequency tuning between building and structural element or component respectively that can lead to an increase of their vibration response compared to the building (resonance, cf. Chapter 2.3.2.3). It is only for some types of components that amplification values are defined, independent of their frequency tuning. Equation (3.6) (EC 8) explicitly includes the frequency tuning with a maximum amplification in resonance by approx. a factor 2. Both formulae include a behavior factor of the structural element or component respectively that considers their plastification capacity. As a rule, for the actual anchoring devices (anchors, dowels, bolts) this factor must be applied to 1.0 so that the anchorage will not fail before the structural element or component respectively reach their ductility. Details can be taken from the prevailing codes.

As a first approximation these formulae can also be used for a more detailed design of the structural elements or components respectively by distributing the determined equivalent forces proportionally to the masses on the struc-tural element or component respectively and carrying out respective strength analyses. On that basis and as a first approximation also non-bearing lightweight partitions or suspended ceilings can be analyzed and anchored against earthquake loads.

4 Drywalling

Based on the fact that lightweight drywall constructions can improve earthquake resistance, this chapter aims to explain this construction method with regard to structures and application possibilities in general, and particularly in terms of the earthquake-resistance implementation.

In doing so, areas that are not directly connected with earthquake resistance will also be explained, as understanding the associations and a proper execution of dry construction structures is of considerable importance. Using drywall construction methods to create structures in any case contributes towards improving earthquake resistance. However, when using in areas of seismic activity, to ensure structural stability it is important to observe additional constructional measures and dimensioning approaches, etc. which are dealt with in the respective sections.

4.1 Basics of Drywalling

Drywalling is a state-of-the-art, efficient method of construction which is increasingly replacing solid construction in internal finishing. This method of construction is standard in Great Britain and particularly in the United States of America.

Drywall constructions are applied in areas that place high demands on design, the physical properties of buildings and quality as well as in many special applications:

- Non-load-bearing partitions
- Diaphragm walls (wood frame constructions, prefabricated buildings)
- Ceiling linings and subceilings
- Floor constructions
- Encasements for columns and girders
- Ventilation and cable channels
- Façade constructions
- Lightweight steel construction

A key constituent of drywall construction systems is the cladding, which is generally executed using gypsum boards. Alongside the outstanding technical properties, which will be dealed with in more detail later on, the building material gypsum generates a comfortable living climate. Thanks to its crystalline structure, gypsum can absorb surplus moisture from the air in the room and then emit it again when the air becomes too dry. This allows you to ensure balanced air humidity within the room without additional measures. At the same time, the surfaces of gypsum materials always feel warm to the touch, as gypsum has a slight thermal conductivity.

It must be pointed out that the drywall construction method is a system construction method, which means that the structures are made up of several components which in turn provide the desired properties only when they interact with one another. In doing so, it is important to keep to the manufacturer's instructions with regard to the components that are to be used and the execution details, and to use exclusively their recommended products in the system. Otherwise the properties cannot be guaranteed. The responsibility for this lies with whoever is carrying out the drywall construction work as well as with the project engineers or architects who have a monitoring function.

4.1.1 Application Areas of Drywall Construction Systems

The range of application areas for drywall systems is very broad. A whole host of requirements can be fulfilled with an optimum use of materials. The following basic requirements can be placed (individually or in combination) on drywall systems:

- Stabilty
  A basic requirement that all drywall systems fulfill is the stability for defined load cases (e. g. dead load, additional loads, bracket loads, wind pressure, and earthquakes).
- Serviceability:
  For standard loads, i.e. dead loads, live loads and wind loads, criteria in addition to the stability, for example, maximum deformations and resistance to cracking, must be complied with in order to guarantee a sustained utilization.
- Enclosure of room
  If there are no requirements on sound insulation or fire protection, the enclosure of room is visually and geometrically guaranteed.
- Additional installation levels
  On account of the non-solid design of drywall systems, there are cavities that can also be used as additional installation levels.

Figure 4.1 Installation levels in ceiling, wall and floor constructions

- Fire protection
  Depending on the design, drywall systems ensure fire protection for various fire resistance classes, types of fire exposure and requirements on the fire behavior.

Appendix B1 contains more detailed information on fire protection with drywall systems.

- Sound insulation

  With an appropriate constructional design, drywall systems offer a considerable airborne and structure-borne sound insulation, either alone or in connection with other components. Refer to Appendix B2 for more detailed information on sound insulation in drywall systems.

- Room acoustics

  With apertura boards of various perforation designs, it is possible to regulate the reverberation times of sound in enclosed areas and hence improve the understanding of speech and reduce noise pollution. Appendix B3 contains more detailed information on sound absorption in drywall systems.

- Visual appearance

  Surfaces of drywall constructions can be designed to different quality standards and, with a wealth of possible coatings or linings, offer the foundations for a high-quality visual design. Chapter 4.3.4 includes the basic information for a proper, high-quality design with joint skimming.

- Bracing

  In the area of wood panel constructions, e. g. in prefabricated building construction, panels braced by gypsum or gypsum fiber boards are used to transfer horizontal loads, i.e. from wind or earthquakes.

  In lightweight steel construction this function is also adopted in structures with metal stud frameworks as well as wall and ceiling frames, e. g. in room-in-room systems like Knauf Cubo.

In addition to this, drywall systems can be used to fulfill very special requirements efficiently, so that these can also be applied whilst taking into consideration earthquake resistance:

- Ball impact safety

  In sports halls, schools and childcare facilities, wall and ceiling structures must offer sufficient resistance to ball impact. Some simple constructional measures usually guarantee this for most drywall systems.

- Radiation protection (x-rays)

  Room-enclosing components for rooms in which x-rays are emitted must shield the adjacent rooms. Standard structures made of heavy concrete can be replaced by lighter drywall systems with lead sheet lamination.

■ Burglar-proofing

Banks and similar buildings make demands on burglar-proofing the walls and, with the appropriate design, these can be fulfilled using drywall partitions.

■ Securing escape routes

Escape tunnels, which, in emergencies or catastrophes must ensure an escape route and withstand the impact of objects falling on them, can be manufactured using a light and practical drywall system.

■ Creative design

With drywall constructions made of gypsum boards, the design possibilities are virtually endless. High-quality, creative room design with molding or mitering techniques or even domes and arches can be realized.

Figure 4.2 Wood frame construction

Figure 4.3 Creative ceiling design

4.1.2 Principle Advantages of Drywalling Compared With Solid Construction

Thanks to the optimized concept for the respective requirements, drywalling offers key advantages compared with standard solid construction:

Controllable properties
Due to the optimum use of components, the properties of drywall systems can be controlled practically. By means of the innovative arrangement and design of the components, virtually all requirements can be achieved with minimum expense - right down to surfaces of all quality levels.

Sound insulation
A prerequisite for good sound insulation is usually a high mass of the component. However, drywall systems have a low weight. By using suitable components and a structural isolation in terms of sound insulation, it can be compensated for this apparent disadvantage. Special drywall partitions fulfill even the considerable sound insulation requirements of cinemas. And this at a fraction of the weight compared to solid construction structures.

Fire protection
Drywall systems can also fulfill virtually all the requirements of fire protection. E. g. ceiling structures with fire resistance period of up to 90 minutes, wall structures with fire resistance period of up to 180 minutes and steel girder and column encasements with up to 180 minutes.

Flexibility
Drywall systems can be assembled and also dismantled quickly and practically. This means subsequent changes to the floor layouts, fire protection and sound insulation improvement measures as well as additional partitions and installation levels can be created quickly and simply without any great expense.

Dry construction method
Apart from the filling compound, all the materials used are “dry”, i.e. they do not contain water. Hence, no moisture can enter the structure and there is now waiting necessary until it is dry and the construction process is accelerated. A more significant role is played by the drywall method pursuant to retrofitting, here construction measures can be carried out in separate floor levels (e. g.: attic extensions, converting a floor in an office block), without the risk of endangering the floors below from any penetrating wetness.

Time savings
The high degree of prefabrication of drywall systems and their components, which is possible right down to completely prefabricated wall and ceiling frames, shortens the

construction time considerably. Thanks to the large area dimensions of the cladding units compared to, for example, masonry bricks, large areas can be completed very quickly.

Less transportation

On account of the lower density of the drywall systems and also of the individual components, the transportation expenditure is rapidly reduced compared to that of solid materials. This applies both for the transportation to the construction site as well as any necessary transportation within the building, which must often be executed by muscle power alone.

- *Example:*

  With a 20 t lorry load of lime sandstone you can produce approx. 100 m² of masonry work. With the same load of gypsum boards you can produce 800 m² of single clad or 400 m² of double clad partition.

Less weight

Of course, the reduced mass does not just play a role in transportation. Thanks to the lower dead weight of the drywall systems, the overall load of a structure can be kept down with far-reaching consequences for the load-bearing structure.
Thanks to the low dead weight, the supporting components are much more efficient.

- *Example:*

  In accordance with DIN 1055-1 light partitions can be taken into consideration up to an overall weight of 3 kN/m, simply by a working load allowance of 0.8 kN/m². When taking into consideration the actual load of, for example, 1.25 kN/m for a 2.5 m high, double clad metal stud partition, the dimensioning is even more favorable.

Table 4.1 Comparison of the dead weight of masonry and drywall partitions

1 m² Internal wall as	**Weight per unit area**
■ Masonry *d* = 11.5 cm	Approx. 145 kg/m²
■ Metal stud partition,	
single-layer cladding	Approx. 25 kg/m²
double-layer cladding	Approx. 50 kg/m²
→ **Weight reduction by 65 % to 83 %**	

If the lighter weight as a structural dead weight is of particular economic importance, it also has significant additional safety-improving properties for the seismic load, as already explained in Chapter 3.1.

More information /4.1/ (refer to the bibliography)

4.1.3 Influence of Drywall Systems on the Earthquake Resistance of Buildings

In the event of an earthquake, the appurtenances of a building must ensure their own structural stability and participate with the movement of the flanking load-bearing structure without suffering any significant damage in the process or even collapsing entirely. If necessary, they must absorb the impact loads from objects falling down around them or on top of them.

By contrast with solid construction, drywall systems offer a series of advantages in the area of earthquake resistance. These benefits are of great significance both from an economical point of view as well as – and most importantly - with regard to safety and the risk of damage:

<u>Structural behavior</u>

a) Non-load bearing partitions in reinforced concrete structures:
Due to the great difference in rigidity between the load bearing structure and the drywall components, any undesired redistribution of load to the non-bearing components (in plane direction) and hence the effects described in Chapter 3.1.1 like the brittle collapsing of infilling masonry, are avoided.

b) Non-load bearing partitions in steel structures and lightweight steel constructions:
On account of their ductile structural behavior, in earthquake areas steel load bearing structures are preferred to reinforced concrete structures. Non-load bearing drywall partitions fit in well in these load bearing structures. In the case of dynamic loads, ductility causes the dissipation of energy (the energy is “absorbed”) caused by the movement of the load bearing structure. The significance of ductility has already been explained in Chapter 2.2.2. As essentially illustrated in the previous chapters, this fact is taken into account in the general design method of earthquake standards by the application of behavior factors so as to reduce the assessed seismic loads, enabling a more cost-effective dimensioning.
Current research is focusing on lightweight steel constructions where the drywall components adopt a load-transferring function in their plane direction levels (diaphragm action).
Particularly cost-effective solutions are expected here, provided behavior factors can be established for these structures. Chapter 2.2.3 explains the connections in terms of the behavior factors.
The current status of the suitability of using lightweight steel constructions is illustrated in Chapter 4.2.4.

Less weight

The reduced weight of drywall components has a very positive effect on the earthquake behavior and earthquake dimensioning of a building:

- Fewer loads that are to be transferred through the load bearing structure.
- Less dead load of the component on account of the seismic load in lateral direction to the component's plane.
- Less potential danger in the building with regard to personal injury and consequential damage.

A simple, exemplary comparative analysis shows the effects of using lightweight construction systems in the place of solid components for the internal finishing:

Load bearing structure:	5-story reinforced concrete skeleton structure with U-shaped lift as a reinforcing core
Conditions:	Design acceleration of 2.4 m/s², Soil class B, Design spectrum 1 in accordance with Eurocode 8 (EN 1998)

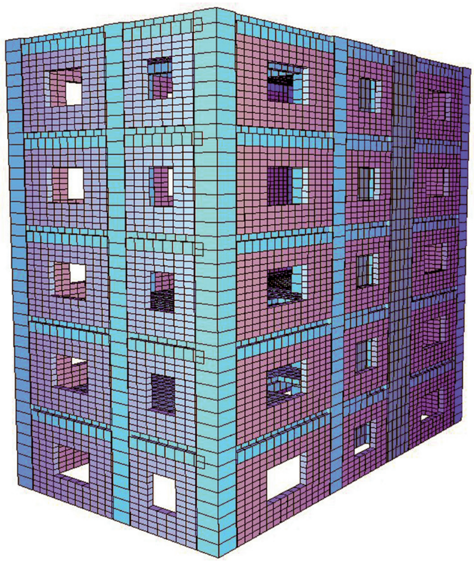


Figure 4.4 Load bearing structure

The following Table 4.2 shows the characteristic values and earthquake loads for the load bearing structure under seismic influence for the variants:

- Non-bracing infilling of the load bearing structure with masonry walls,
- Bracing infilling of the load bearing structure with masonry walls,
- Infilling of the load bearing structure with drywall metal stud partitions

for the specified conditions:

Table 4.2 Comparison of earthquake parameters
Finishing with masonry or drywalling

		Masonry infill non-bracing	**bracing**	**Drywalling infill**
Dead weight [kN]		8107	8107	6160
Basic period [s] of	X-direction	0.42	0.21	0.38
	Y-direction	0.37	0.16	0.33
	Torsion	0.31	0.13	0.28
Spectral acceleration [m/s²]	X-direction	7.2	7.2	7.2
	Y-direction	7.2	7.2	7.2
Behavior factor q		4.0	2.0	4.0
Earthquake load on base point [kN]	X-direction	**1459**	**2919**	**1109**
	Y-direction	**1459**	**2919**	**1109**

The effects of using the infilling variants on the earthquake load are very evident; brought about by a combination of a more favorable behavior factor with the lower weight. The table also illustrates a comparison of the load determination with and without taking into account the bracing effect of the masonry structure. You can see quite clearly the importance of the conformity of load determination and load bearing design. If, as in the majority of cases, the masonry infilling is executed without movement joints, the seismic loads are significantly greater. If this is not taken into consideration in the load determination, the entire load bearing structure, especially the footings and the lowest floor, are underdesigned, which, in the most unfavorable of circumstances, can cause a total collapse.
Even with an appropriate design of the reinforced steel load bearing structure, the masonry is subjected to excessive shear stresses, which could lead to the collapse of the masonry with the respective consequences for the people residing in the building as well as for the entire load bearing structure, i.e. entire collapse on account of a sudden load redistribution. The maximum achieved shearing stress is 1.2 N/mm² (Figure 4.5). In this example, the permissible shearing stress in masonry amounts to just 0.25 N/mm².

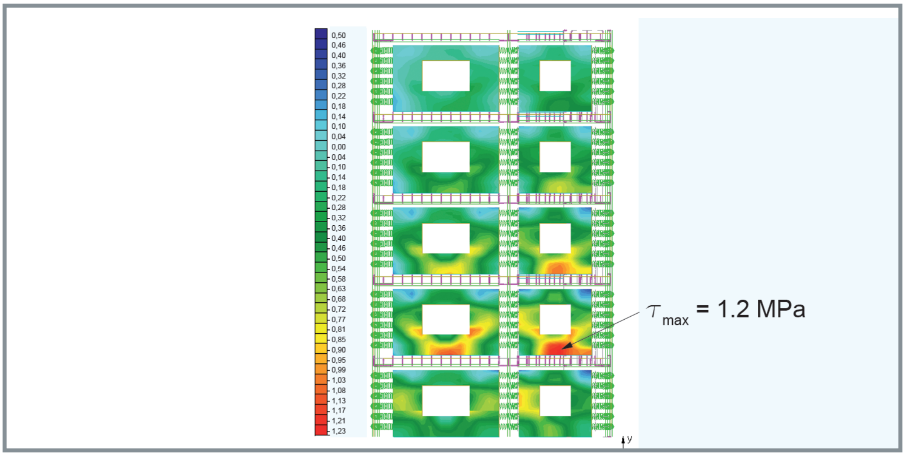


Figure 4.5 Shear stresses in masonry as a result of seismic loads

Leaving aside the effects of the dimensioning, there are even more factors that speak in favor of using drywall systems in earthquake-endangered buildings:

Collapse behavior with less potential danger
Even in the improbable scenario of the drywall system failing, e. g. in association with a strong earthquake, the danger is comparatively slight, as the failure would not be brittle. On account of their many connections with each other, the individual components (especially those in walls) would not suffer an extensive fall.
Also when repairing destroyed or damaged elements, the considerably lighter masses that are to be moved compared with those of solid construction components, play a role both with regard to clearance as well as, in extreme cases, when saving enclosed people.

Simple reconstruction / repair
Depending on the intensity of the earthquake and the design of the component connections, serious or negligible damage could occur to the non-load bearing components. The economic efficiency of the reconstruction is the main focus for this approach. In the case of drywall systems, the extent of the reconstruction requirement could range from simply filling cracks through to replacing entire components after a strong earthquake. If drywall systems are used, the repairs can be carried out efficiently and quickly, i.e. less work and the building will be up and running again more quickly, which can also be of great economic as well as infrastructural significance (schools, hospitals, authorities, etc.).

Figure 4.6 Repairing damaged drywall structures

Furthermore, the secondary damage in the case of damaged drywall systems compared to destroyed solid construction components is considerably less, which in turn reduces the retrofitting expenditure.

4.1.4 Earthquake Resistance of Drywall Systems

While, as illustrated in the previous chapter, drywall systems have a positive influence on the earthquake resistance of the entire construction, they must however also be designed to take into consideration the additional loads. The criteria of structural stability and serviceability (maintaining function and remaining free from damage) play a significant role here.

The structural stability of the structures must be ensured for non-load bearing components exclusively caused by

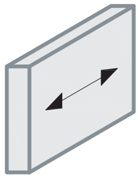
- Dead weight x earthquake excitation

as well as

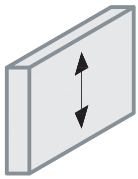
- Movement (deformation) of the flanking components,

in three directions.

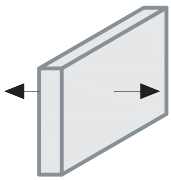
<u>Vertical components (walls, façades, furrings)</u>



Diaphragm action, horizontally
(in plane direction of component)
Cause: Movement of the flanking components
(e. g. storey movement)

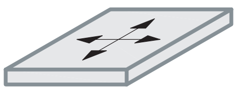


Diaphragm action, vertically
(in plane direction of component)
Cause: Movement of the flanking components
(e. g. ceiling deflection)

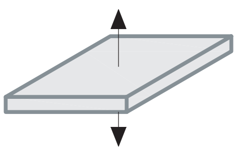


Lateral loads, horizontally
(transverse to plane direction of component)
Cause: Force from
dead weight x horizontal earthquake acceleration

<u>Horizontal components (ceilings, floors)</u>



Diaphragm action, horizontally
(in plane direction of component)
In both main directions
Cause: Force from
dead weight x horizontal earthquake acceleration



Lateral loads, vertically
(transverse to plane direction of component)
Cause: Force from
dead weight x vertical earthquake acceleration

Figure 4.7 Load directions on non-load bearing components from seismic load

Depending on the position of the component, the design, the execution of the connections and the application area, the individual directions can be of differing significance, and occur in combination. Figure 4.7 shows the load directions, in dependence on the position of the component.
When dimensioning and constructionally designing the components, all the load directions must be taken into consideration.

Bracing components as load bearing components, take additional loads from the overall structure.

The serviceability is ensured by constructional measures in the required limitations.

In walls, for example, by means of sliding connections to the flanking components.

Chapter 3.3.3 illustrates the demands on serviceability beyond the dimensioning of the pure load bearing capacity. As, for example, drywall systems compared to solid construction have less stiffness and a greater deformation capability, they can combat greater deformations in the plane direction without failing and, in doing so, maintain their room-enclosing function. In the case of earthquakes of less strength, the damage risk is minimized.
This fact is taken into consideration when establishing the damage restriction conditions of the Eurocode 8 (EN 1998. Design of structures for earthquake resistance).
For high-rising structures, where non-load bearing components made of brittle materials, e. g. masonry walls, are fixed to the construction, the maximum permissible story movement d_r *is restricted to*

$$d_r \cdot v \leq 0.005 \cdot h$$

h ... Height of story
v ... Reduction factor = 0.5 or 0.4, depending on the importance category of the structure

If, however, ductile building materials are used to produce the non-bearing components, e. g. metal stud partitions, the threshold for the story movement d_r is

$$d_r \cdot v \leq 0.0075 \cdot h.$$

The reduction factor takes into account the fact that the damage restriction for earthquakes is established with a greater probability of occurrence than the design spectrum for the load bearing capacity.

During tests at the CSTB (Centre Scientifique et Technique du Bâtiment) in France, the behavior of metal stud partitions was tested under dynamic load in vertical and horizontal in-plane directions for different deformations. The results show that even after considerable deformation under dynamic loads (story movement of 26 mm with 2600 mm wall height, corresponding to 0.01•*h*) this did not endanger the surrounding areas. (See Figures 4.8 and 4.9)

Figure 4.8 External and internal view of the wall after dynamic load with horizontal head point movement in plane direction of 26 mm with 2600 mm wall height (story movement)

Figure 4.9 External view of the wall after dynamic load with vertical head point movement in plane direction of 27 mm (ceiling deflection)

The following chapter explains the individual drywall systems in terms of their functioning with regard to their construction. Then, in each case, the points that are of relevance with regard to structural stability (dimensioning) and serviceability (constructional measures) for earthquake resistance are presented in detail.

4.2 Drywall Constructions

The construction principle of drywalling as an industrialized construction method is based on the optimum use of components in accordance with their properties and hence complying with the requirements on the respective component with a minimum use of materials.

Drywall systems generally consist of a substructure made of thin gauge sheet metal profiles or wood studs or battens, if necessary an insulation material in the wall or ceiling cavities and a cladding made of gypsum boards, the joints of which are sealed using a suitable filling compound.

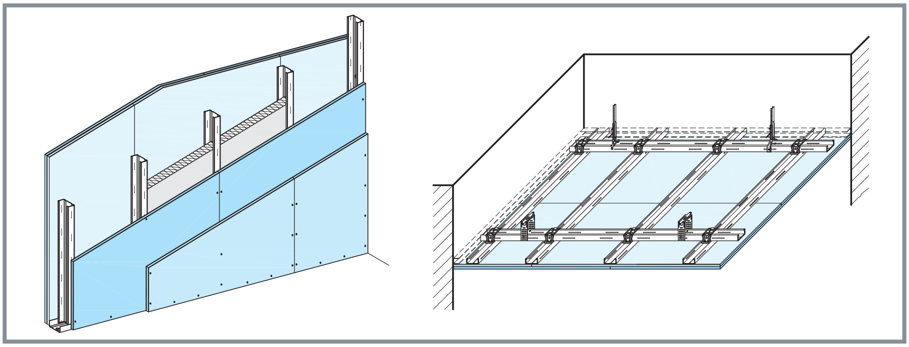


Figure 4.10 Drywall metal stud partition and subceiling

The connection between the substructure and the cladding is made using screws, nails or staples. The cladding fulfills the room-enclosing function while the substructure, in interaction with the cladding, guarantees the structural stability.

4.2.1 Ceiling Linings and Subceilings

4.2.1.1 Difference Between Ceiling Linings and Subceilings

Widely independently of the variant of substructure, drywall ceilings are divided into two categories:

Ceiling linings are connected directly to the fixing base without using interim components, e. g. the cladding of wood joist ceilings or trapezoid sheet ceilings. There is no space between the structure of the ceiling lining and the basic ceiling.

Subceilings have a cavity space between the fixing base and the suspended subceiling, the substructure is not fixed directly to the basic ceiling. It is suspended either using hangers or free-spanning between surrounding walls.

It is therefore clear that the variant without substructure can be assigned only to the ceiling lining.

The free-spanning ceiling again can be considered exclusively a subceiling, in the

same way as a dropped ceiling, which can be merely installed as a subceiling.
All the other variants with substructures can be assigned to both categories depending on how the substructure is anchored to the basic ceiling.

Description of construction

- Ceiling linings:
  Ceiling linings are anchored directly to the fixing base, i.e. on a solid ceiling, wood joist ceiling, the rafters of a roof framework or on a trapezoid sheet ceiling. They are preferably used where a low construction height is required, i.e. as little loss as possible in room height.
  They can be designed with or without substructures. In the case of variants without substructures, gypsum boards are screw attached directly to the fixing base.
  In the case of variants with substructures, these are anchored directly and hence with no gap to the fixing base, the cladding is screwed to the substructure. This has the advantage that any unevenness in the surface can be balanced out, and a better sound-insulating isolation of the cladding from the fixing base is achieved.

- Subceilings:
  Subceilings always consist of a substructure and the cladding fastened to this.
  The substructure is generally suspended from the basic ceiling by means of hangers. But it is also possible to use a free-spanning substructure that is anchored only to the surrounding walls.
  The cavity space achieved between the basic ceiling and the subceiling can be used for insulation or installations.
  The room height can be freely chosen, e. g. in the case of extremely high rooms fitting a subceiling can reduce the heating energy requirement.

4.2.1.2 Construction Types of Ceiling Linings and Subceilings

There are nine construction types for drywall ceilings, and these are briefly described in the following:

1) Without substructure, direct fastening of gypsum boards to the fixing base, e. g. lining of roof rafters, wood joist ceilings or trapezoid sheet ceilings or roofs.

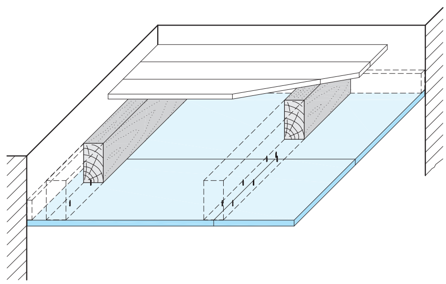


Figure 4.11 Direct fastening of ceiling lining

Advantages: Very low construction height
Disadvantages: The surface must be level as it is not possible to compensate
No sound isolation of the cladding

2) With substructure made of wooden battens with one substructure level, consisting only of furring battens.

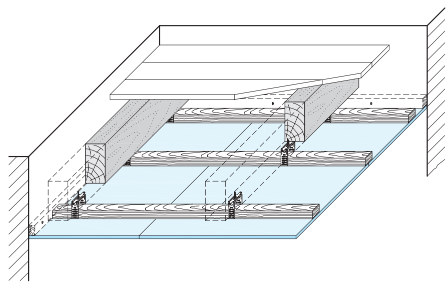


Figure 4.12 Board ceiling with wooden substructure with one substructure level

Advantages: It is possible to compensate for an uneven surface
Good sound isolation
Disadvantages: A slightly greater construction height than with direct fastening

3) With substructure made of sheet metal profiles, with one substructure level, consisting only of furring channels.

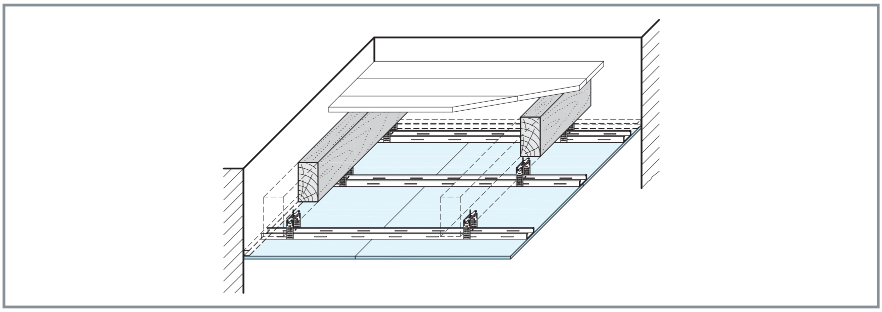


Figure 4.13 Board ceiling with metal substructure with one substructure level

Advantages: It is possible to compensate for an uneven surface
Good sound isolation
Disadvantages: A slightly greater construction height than with direct fastening

4) With substructure made of wooden battens with two substructure levels, which are connected at right-angles. The upper substructure level is called the carrying timber battening, the lower is referred to as the furring timber battening, as this is “furred” with the cladding.

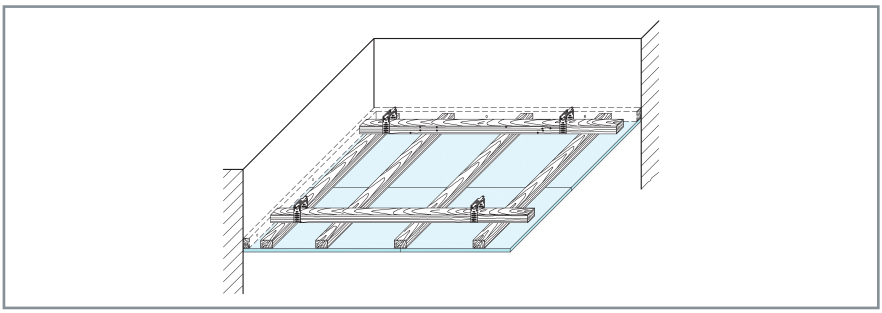


Figure 4.14 Board ceiling with wooden substructure with two substructure levels

Advantages: Installation levels, space for insulation
The room height can be chosen freely
Good sound isolation
Fewer suspension points than with structures that have just one substructure level
Disadvantages: Greater construction height

5) With substructure made of sheet metal profiles, with two substructure levels, which are connected at right-angles. The upper substructure level is called the carrying channels, and the lower is referred to as the furring channels.

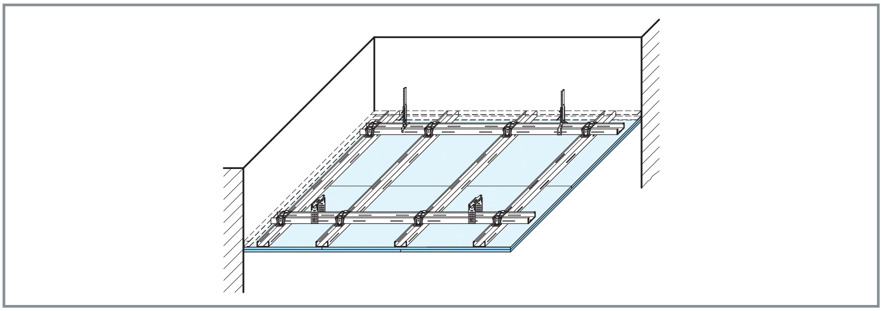


Figure 4.15 Board ceiling with metal substructure with two substructure levels

Advantages: Installation levels, space for insulation
The room height can be chosen freely
Good sound isolation
Fewer suspension points than with structures that have just one substructure level

Disadvantages: Greater construction height

6) As 5), however, the carrying channels consist of UA sheet metal profiles using 2 mm thick sheeting, greater distances between the hangers are possible; "wide-span ceiling".

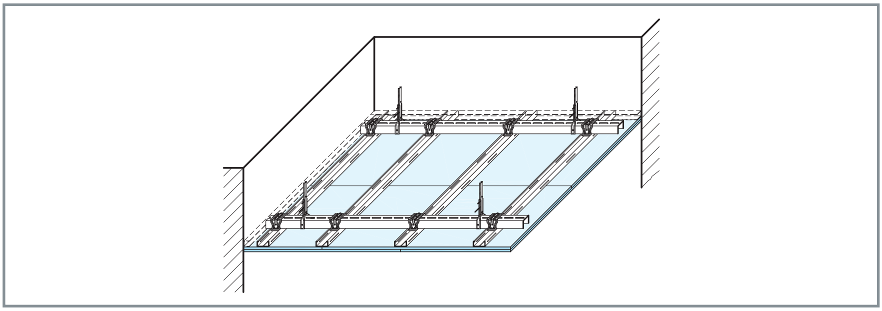


Figure 4.16 Board ceiling with wide-span metal substructure with two substructure levels

Advantages: Greater distances between the hangers possible
Installation levels, space for insulation
The room height can be chosen freely
Good sound isolation

Disadvantages: Greater construction height

7) Flush, carrying and furring channels are at the same level, and the carrying channels are also fitted with cladding.

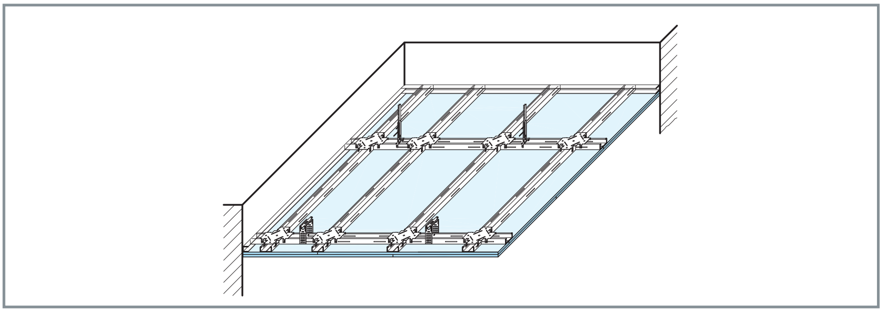


Figure 4.17 Board ceiling with flush metal substructure

Advantages:	Reduced construction height
	Installation levels, space for insulation
	The room height can be chosen freely
	Good sound isolation
Disadvantages:	Because of the greater suspension distances and the method of construction a rather unfavorable behavior in view of dynamic effects (earthquakes)

8) The free-spanning ceiling is an innovative variant without any anchoring to the basic ceiling, anchored only to the surrounding walls.

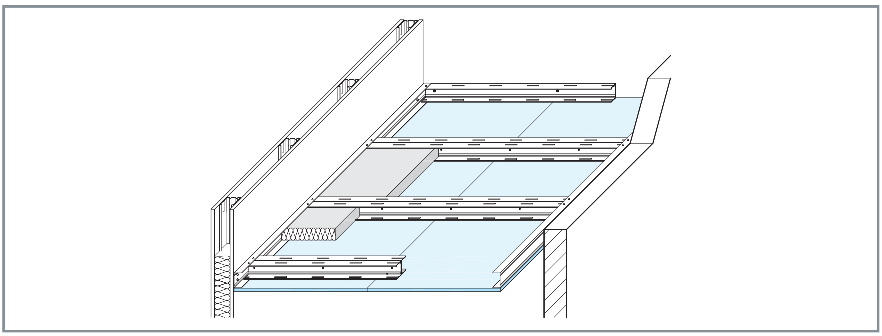


Figure 4.18 Board ceiling with free-spanning metal substructure

Advantages:	Labor-saving (no suspension)
	Basic ceiling is not additionally stressed
	Completely free installation level
	The room height can be chosen freely
	Very good sound isolation from the basic ceiling
Disadvantages:	Restricted room width

9) Dropped ceilings, ceiling boards are placed on the substructure.

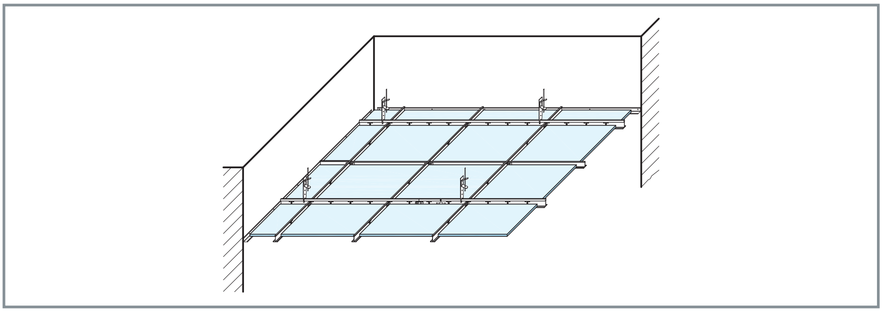


Figure 4.19 Dropped ceiling

Advantages:	Design
	Installation levels, space for insulation
	The room height can be chosen freely
	Good sound isolation
Disadvantages:	Greater construction height
	No diaphragm action (unfavorable with regard to earthquake resistance)
	In earthquake-endangered areas, all the board elements must be secured by means of staples against falling down

The choice of the respective optimum construction type should be made in dependence on the general conditions and requirements that are being placed on the ceiling.

In the case of ceiling systems with a substructure, the boards are generally positioned at right angles to the substructure (furring channels / battens), as their stiffness is greater in the longitudinal direction (production direction). With the exception of dropped ceilings, the cladding is fastened using drywall screws to the substructure or directly to the fixing base. The thickness of the cladding is determined by the requirements. Multi-layer cladding is also possible.
For the purpose of fire protection, sound insulation or thermal protection requirements insert a layer of insulation can be inserted on top of the subceiling.

With the exception of dropped ceilings, the fitted gypsum boards are sealed using a suitable filling compound. Refer to Chapter 4.3.4 for detailed information.
When implementing the ceiling structures you must observe a series of instructions, which are specified in the manufacturer's documentation and in the standards:

- Staggering and arranging the cladding joints
- Selection of and spacings of the fastening materials

- Movement joints
- Design of the connection joints
- Connections for non-load bearing partitions
- Fixing additional loads / single loads

4.2.1.3 Dimensioning of Ceiling Linings and Subceilings

The substructure of gypsum board ceilings must be dimensioned in accordance with their dead weight as well as taking into consideration possible additional loads from added insulation materials, built-in parts, the wind pressure or earthquakes.
The most important parameters of ceiling linings and suspended subceilings are:

- The spacing of the furring channels which is restricted by the maximum span of the cladding
- The spacing of the carrying channels which is restricted by the maximum span of the furring channels
- The spacing of the hangers which is restricted by the load bearing capacity of the hangers as well as the maximum span of the carrying channels

In doing so, the spacing must be chosen in such a way as to ensure a proper application of the boards, i.e. it must always be possible to arrange the cut edge joints with lateral cladding on furring channels / battens. For example, with 2 m long boards the spacing of 333, 400 or 500 mm is possible.

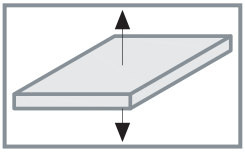
With free-spanning subceilings the maximum room width, i.e. the span, is the main parameter that is influenced by the ceiling load.

For the dimensioning in accordance with the dimensioning diagram specified in this book (Appendix C1) four different load classes are assumed, for which the respective hanger and channel spacing are defined:
$p \leq 0.15$ kN/m²
$0.15 < p \leq 0.30$ kN/m²
$0.30 < p \leq 0.50$ kN/m²
$0.50 < p \leq 0.65$ kN/m²

Dimensioning diagram for gypsum board ceilings **refer to Appendix C1**

4.2.1.4 Consideration of Earthquake Loads for Dimensioning

As horizontal components, as illustrated in Chapter 4.1.4 ceilings must be able to transfer loads in both a vertical as well as a horizontal direction in the event of an earthquake.



The vertical part, which acts at right angles to the plane direction of the component, leads to a greater load on the substructure in the case of subceilings and ceiling linings.

This must be taken into consideration when dimensioning the spacing of channels and hangers for suspended subceilings and ceiling linings or the maximum span of free-spanning subceilings. In doing so, possible additional loads, which also add to the dead weight of the ceiling lining or subceiling, e. g. insulation materials or built-in parts, must also be taken into consideration. By taking the load of the dead weight (structure + additional loads) + earthquake load, you can establish the load class for the further dimensioning of the ceiling.

Table 4.3 shows the earthquake loads that must be considered, based on the equation (3.6) for different earthquake zones (design acceleration $a_g = a_{gR} \cdot \gamma_1$) for the event that the most unfavorable ratio between the natural oscillation behavior of the load bearing structure and the non-load bearing component is $T_a/T_1=1$.

In most cases, this ratio however is dissimilar to 1 and hence the earthquake load that could be applied would be less than this.

The values in the table are however an approach with which earthquake loads can be established for ceiling dimensioning without analyzing the dynamic behavior.

Merely the product from the surface parameter (in accordance with Eurocode 8) and the design acceleration (in dependence on the earthquake zone) must be determined. The base parameter *S* is set at 1.0 for the vertical earthquake loads in accordance with Eurocode 8 so that the design acceleration remains the only variable parameter.

It must be noted that the earthquake load has a downward effect together with the dead weight, as well as an upward effect against the dead weight. If the earthquake load is greater than the dead weight, in the case of suspended subceilings, pressure forces are applied on the hangers through the entire ceiling surface. The hangers must then be capable of resisting these pressure forces. The suspension height must therefore be kept as low as possible.

The earthquake loads shown in Table 4.3 are already included in the dimensioning diagrams in Appendix C1, so this dimensioning diagram deviates from the standard dimensioning diagram for static loads.

Table 4.3 Vertical additional load from earthquakes for ceiling linings / subceilings

a_g [m/s²]	Additional load from earthquakes for ceiling linings / subceilings (kN/m²)							
	Single-layer claddingg Thickness in mm			Double-layer cladding Thickness in mm			Additional load (e. g. mineral wool)	
	12.5	18	20	2x12.5	18+15	2x20	2 kg/m²	5 kg/m²
0.4	0.01	0.02	0.02	0.03	0.04	0.05	0.00	0.01
0.6	0.02	0.03	0.03	0.04	0.06	0.07	0.00	0.01
0.8	0.03	0.04	0.05	0.06	0.07	0.09	0.01	0.01
1.0	0.04	0.05	0.06	0.07	0.09	0.11	0.01	0.02
1.5	0.06	0.08	0.09	0.11	0.14	0.17	0.01	0.02
2.0	0.07	0.10	0.12	0.14	0.19	0.23	0.02	0.03
2.5	0.09	0.13	0.14	0.18	0.23	0.28	0.02	0.04
3.0	0.11	0.16	0.17	0.21	0.28	0.34	0.02	0.05
3.5	0.13*	0.18	0.20	0.25	0.33	0.39	0.03*	0.06*
4.0	0.15*	0.21*	0.23*	0.29*	0.37*	0.45*	0.03*	0.07*
4.5	0.17*	0.24*	0.26*	0.32*	0.42*	0.51*	0.04*	0.07*
5.0	0.19*	0.26*	0.29*	0.36*	0.47*	0.56*	0.04*	0.08*

* Take the pressure force on brackets into account

Example for taking into consideration the vertical earthquake load on ceilings:

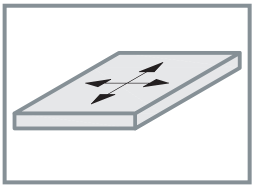
- Public buildings: Importance factor acc. to EN 1998 γ_1 = 1.2
- Seismic zone III (Greece):
  Design acceleration $a_g = a_{gR} \cdot \gamma_1$ = 3.6 m/s² · 1.2 = 4.3 m/s²
- Subceiling with 18 mm GKF cladding (gypsum board, fire-resistant), single-layer
- 40 mm mineral wool with 40 kg/m³ density,
  corresponding to a weight of 1.6 ≈ 2 kg/m²
- Ceiling surface 10 x 10 m

Determination of the respective load class:

- Ceiling dead weight: 0.21 kN/m²
- Weight of the insulation: 0.02 kN/m²
- Additional load from earthquakes - ceiling from table: 0.24 kN/m²
- Additional load from earthquakes – insulation from table: 0.04 kN/m²

Total 0.51 kN/m²

→ **Load class for the subceiling dimensioning: 0.50-0.65 kN/m²**
(take the arising pressure force: 0.21 + 0.02 - 0.24 - 0.04 = -0.05 kN/m² into account)



With suspended subceilings, the horizontal part of the earthquake load leads to compression stresses in the perimeter area, which must be absorbed by means of the compression strength of the boards as well as a connection that is able to transfer this load.

The compression strength of gypsum boards varies depending on the type of board and manufacturer and valid standards, however, as a characteristic value it generally is at least 3.5 N/mm².
Depending on the weight of the cladding, the cladding thickness, the ceiling surface and the horizontal acceleration, the compression stress can be calculated on the edge supports for both main directions.

If you summarize the respective equations, it shows that only the length of the floor layout perpendicular to the proven connection edge (b_1) as well as the ordinates of the dimensioning spectrum are of relevance when establishing the compression stress:

- Compression stress σ_{cd} across the board edge for horizontal acceleration:

$$\sigma_{cd} = \frac{F_a}{b_2 \cdot t}$$

b_2 ... Floor layout length along the edge that is to be proven
t ... Board thickness
F_a ... Earthquake load

- Earthquake load F_a, determined acc. to (3.6) ´with
$z/H = 1$
$T_a/T_1 = 1$
$\gamma_a = 1$ acc. to EN 1998 Chapter 4.3.5.3 (2)
$q_a = 2$ acc. to EN 1998, Table 4.4:

$$F_a = \frac{S \cdot \alpha \cdot 5.5 \cdot W_a \cdot \gamma_a}{q_a} = \frac{S \cdot a_g \cdot 5.5 \cdot W_a \cdot 1}{2 \cdot g}$$

- Weight of the component $W_a = \rho \cdot g \cdot t \cdot b_1 \cdot b_2$

- Hence the earthquake load is

$$F_a = \frac{S \cdot a_g \cdot 5.5 \cdot \rho \cdot g \cdot t \cdot b_1 \cdot b_2}{2 \cdot g} = \frac{S \cdot a_g \cdot 5.5 \cdot \rho \cdot t \cdot b_1 \cdot b_2}{2}$$

- The resulting compression stress on the board edge is

$$\sigma_{cd} = \frac{S \cdot a_g \cdot 5.5 \cdot \rho \cdot b_1}{2}$$

If you now calculate the maximum permissible length of floor layout $b_{1,max}$ for a design earthquake, which lies at the top end of the scale in terms of occurring earthquake loads, and take into consideration that the thickness of the boards is proportionate to the compression resistance, you can establish the maximum floor layout length by using the following starting parameters:

- Density in raw state of gypsum boards, e. g.

  $$\rho = 680 \frac{kg}{m^3}$$

- Maximum soil factor for spectrum type 1

  $$S = 1.4$$

- Characteristic compression strength GKB boards (refer to chapter 4.3.1)

  $$\sigma_{cd} = 3.5 \frac{N}{mm^2}$$

- Horizontal design acceleration for earthquake zone III, Greece,
  Importance category IV

  $$a_g = \gamma_1 \cdot a_g = 1.4 \cdot 0.36g = 5.0 \frac{m}{s^2}$$

- Calculation of $b_{1.max}$:

  $$3.5 \frac{N}{mm^2} = \frac{1.4 \cdot 5.0 \frac{m}{s^2} \cdot 5.5 \cdot 680 \frac{kg}{m^3} \cdot b_{1,max}}{2 \cdot 10^9 \frac{mm^3}{m^3}}$$

  $$b_{1,max} = \frac{2 \cdot 3.5 \frac{N}{mm^2} \cdot 10^9 \frac{mm^3}{m^3}}{1.4 \cdot 5,0 \frac{m}{s^2} \cdot 5.5 \cdot 680 \frac{kg}{m^3}} = \frac{2 \cdot 3.5 \cdot 10^9}{1.4 \cdot 5.0 \cdot 5.5 \cdot 680} mm$$

  $$b_{1,max} = 267379.7\,mm = \mathbf{267.4\,m}$$

Such a floor layout length for a subceiling is however incredibly unrealistic, so it is clear that the horizontal load from earthquakes (even if you apply the highest possible earthquake load and the lowest possible board compression strength) does not endanger the load bearing capacity of suspended ceilings. However, a strong edge connection is imperative, otherwise this could cause uncontrollable pendulum movements of the subceiling and undesirable load redistribution (e. g. as a lateral force in the hangers).

In the case of ceiling linings, the horizontal earthquake load is discharged as a lateral force in the anchoring of the fixings (hangers). The earthquake load is determined analogous to the suspended subceiling and distributed to the number of anchoring points:

- Earthquake load per ceiling area:

$$F_a = \frac{1.4 \cdot 5{,}0\,\frac{m}{s^2} \cdot 5.5 \cdot 680\,\frac{kg}{m^3} \cdot 0.018\,m}{2} = 235.6\,\frac{N}{m^2}$$

- Number of anchoring points with a hanger spacing of 600 mm, furring channel spacing of 625 mm and layout area 10 m x 10 m: approx. 290 pieces
- Lateral force for each anchoring point

$$F_Q = \frac{235.6\,\frac{N}{m^2} \cdot 10\,m \cdot 10\,m}{290} = 81.2\ N \approx 0.08\ kN\ per\ anchor$$

- Permissible lateral force, e. g. Knauf ceiling steel dowel: 0.5 kN
  (note the possible varying load bearing capacity of the anchors being used)

Hence also for ceiling linings it shows that the horizontal components of the earthquake load are not decisive in a normal scenario.

In the case of free-spanning subceilings, the horizontal earthquake force plays no role in the dimensioning, as all the edges are connected to the flanking components in a manner that is tension-proof and compression-resistant.
However, in individual cases it must be checked whether the movement of the flanking walls under the effects of an earthquake can destroy the load bearing connections of the free-spanning ceilings

4.2.1.5 Constructional Measures for Ceiling Linings and Subceilings in Earthquake areas

If ceiling linings or subceilings are used in areas of considerable seismic activity, there are some additional constructional measures that must be taken into consideration, beyond the incorporation of the dimensioning.
In order to minimize the risk of the connections “shaking out”, friction-based connections are not recommended.
Particularly using Nonius hangers or Direct hangers can be recommended, if necessary in connection with drywall profiles (refer to Figure 4.85).The hangers and profile connectors must always be screwed to the substructure.
The steel dowels that are used as anchorage in solid ceilings, must also be authorized for use for earthquake loads and in cracked concrete tension zone.

The use of through dowels or spring dowels, which also maintain their full load bearing capabilities even in the case of large concrete cracks, are also favored. However, it is essential to ensure sufficient concrete quality.
To anchor the substructure to the basic ceiling or to the surrounding walls, use only anchors whose load bearing capacity has also been proven by means of cyclic loads. Suspended subceilings should be implemented only with a substructure as a two level grid. The suspension height should be kept as small as possible.

Computer simulations for suspended ceilings under earthquake loads have shown that the narrower the hanger spacing, the better the structural behavior, which is similar to a flexibly supported board. At the same time, the load on the individual hangers is reduced, resulting in increased safety reserves.

Use only hangers with a load bearing capacity of 400 N or greater. Wire hangers aren't always suitable. The load bearing capacity must be established for all the brackets used in accordance with the pertinent standards, e. g. EN 13964 or DIN 18168, and certified by a respective test certificate from an authorized testing institute.

All built-in elements, installation pipes, etc. whose weights have not been taken into account for dimensioning must be anchored independently on the basic ceiling.

There are two methods of connecting suspended ceilings to the surrounding walls. They can be connected in such a way that the connection has a supporting function, i.e. works as a furring channel and helps to transfer the dead weight. The second possibility is the design as a constructional connection without a supporting function. The choice of the connection type depends on the distance at which the first furring channel and carrying channel as well as hangers must be arranged from the edge of the ceiling.
In the case of bearing ceiling connections to walls, it is necessary to use an edge profile which is anchored to the surrounding walls. In the case of constructional (non bearing) ceiling connections to walls, it is also recommended to build in the edge profile as an assembly aid or this may even be a necessity for fire protection requirements. The design as a constructional connection is recommended for earthquake-resistant ceiling constructions. In the case of contrary movements in view of the surrounding walls opposite to each other, unpredictable loads can occur on the connections. Local damage caused in this way can cause a bearing edge connection to become ineffective, with a potential chain reaction which results in the ceiling failing.

In any case, the connection should ensure the transfer of pressure forces, but not transfer any tensile forces into the subceiling.

Under the following conditions, diagonal bracings must be integrated at specific spacings in the substructure of suspended ceilings:
- Use of hangers made of wire
and
- Suspension height ≥ 30 cm
and
- Ceiling area ≥ 25 m²

In the area of the spacings, compression struts should be built in instead of the hangers. The bracing must be effective in two directions and at an angle of max. 45° to the ceiling surface. The bracings must be arranged at a spacing of max. 4 m in both directions, and max. 2 m away from the flanking walls. Figure 4.20 shows an exemplary design of such a bracing. The diagonal tension struts can, for example, be designed with metal slotted strips.

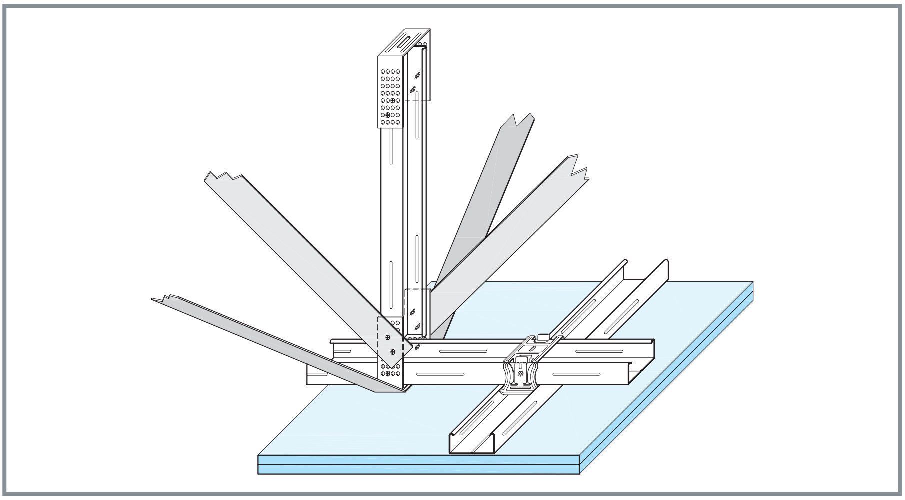


Figure 4.20 Diagonal bracing of suspended ceilings with compression strut

In principle, in earthquake regions the weight of the cladding must be kept as low as possible. Therefore, in the case of requirements on sound insulation or fire protection, if necessary use the more efficient gypsum boards in place of additional cladding layers. Manufacturers of gypsum boards offer special boards for this purpose, e. g. Knauf PIANO Sound Shields (good sound insulation) or Fireboard (good fire protection while being of low weight).

More information /4.2/, /4.3/, /4.4/, /4.5/, /4.6/ (refer to the bibliography)

4.2.2 Non-Load Bearing Partitions

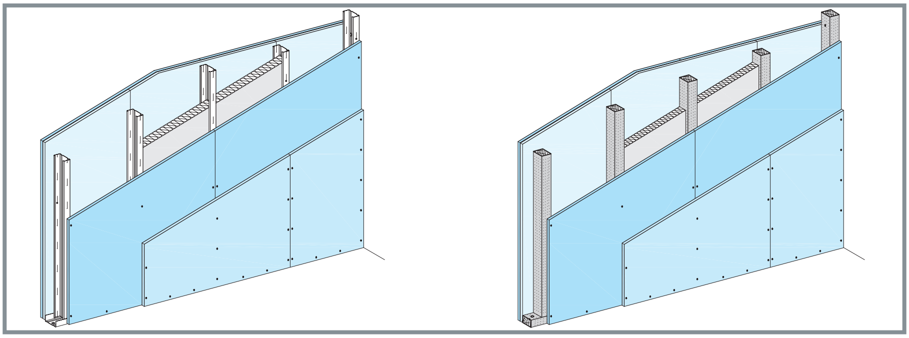


Figure 4.21 Metal stud partition and wood frame partition

4.2.2.1 Description of Construction

Non-load bearing inner partitions essentially consist of a substructure made of sheet metal studs or wooden studs, a cladding made of gypsum boards and, if necessary, an insulation in the cavity of the partition.

The substructure of metal stud partitions consists of circumferential perimeter runners which are anchored to the adjacent components like the ceiling, floor and flanking walls. An acoustical sealant compound or sealing tape is applied to the rear of these perimeter runners in order to seal the edge connection of the wall. The studs are placed in the floor and ceiling perimeter runners with a spacing of usually 600 mm or 625 mm, depending on the width of the gypsum boards.
The substructure can be installed as a single metal stud frame, double metal stud frame or isolated double stud frame or it can be executed as a crossbar construction. The most common variant is the single metal stud frame.

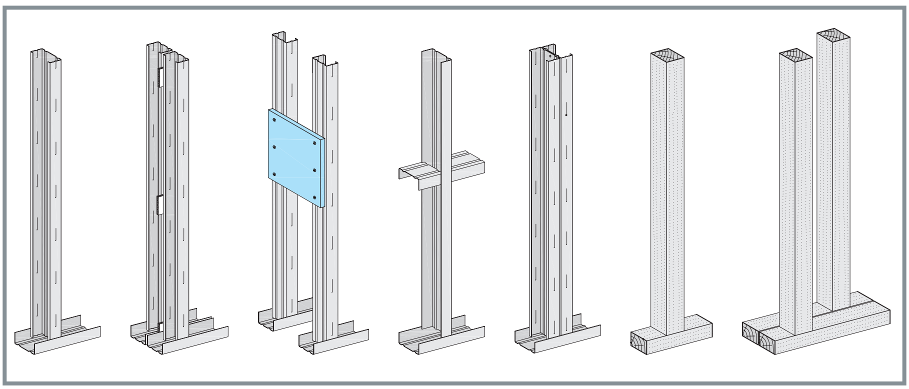


Figure 4.22 Variants of the substructure of metal stud partitions and wood frame partitions

The substructure wood frame partitions is very similar to that of metal stud partitions, however, wooden studs, ties and beams, are used in the place of sheet metal studs and runners.
The gypsum boards are then fastened to this substructure by means of drywall screws or, in the case of wooden substructures, also with staples or nails. The width of the boards generally amounts to 1200 mm or 1250 mm, so that in each case the joints are arranged concentrically on a stand. The boards are fastened to the studs at the edges and in the middle of the boards.
Depending on the requirements, it may be necessary to use a multi-layer cladding. Additional cladding layers increase the sound insulation, the fire protection, the permissible bracket loads and the permissible wall height. The latter applies only partially in the case of earthquake loads, as additional cladding layers signify a greater dead weight.

A range of board types with various properties are available for cladding purposes, as explained in Chapter 4.3.1 in more detail.

If an insulation layer is necessary for reasons of sound insulation, fire protection or thermal protection requirements, this is inserted throughout between the stands of the partition cavity. In doing so, in dependence on these requirements, different insulation materials can be used – refer to Chapter 4.3.5 for more information in this regard.

The fitted gypsum boards are filled and skim coated using a suitable filling compound. Refer to Chapter 4.3.4 for more detailed information.
When building non-load bearing internal partitions you must observe a series of instructions that are specified in the manufacturer's documentation and in the standards:

- Staggering the cladding joints
- Choice and spacing of the fastening materials
- Control joints
- Profile joints (extensions)
- Door openings
- Fitting power sockets
- Special requirements for tile cladding
- Design of the connection joints

4.2.2.2 Load Bearing Capacity of Non-Load Bearing Drywall Partitions

Even non-load bearing partitions can absorb certain loads. The term "non-load bearing" refers to the load transfer within the entire load bearing structure; the partitions are not included here. They can however take bracket loads, dynamic pressure from

wind loads, horizontal impulsive loads and, if necessary, earthquake loads from dead weight.
Bracket loads can be fixed without any additional further measures up to 0.4 kN per meter wall length for single-layer clad walls and up to 0.7 kN per meter wall length for multi-layer clad walls. However, permissible loads for each fixing point, in dependence on the fixing materials, must be complied with.

For specifications on permissible bracket loads **refer to Appendix C2.1**

Greater loads up to 1.5 kN per m wall length will be placed on the substructure by means of sanistands or traverses.
Examples for thes are wall-anchored toilet pans, wall-anchored fold-down seats, wall-suspended boilers, etc.
There are a number of different sanitary accessories for this application area.

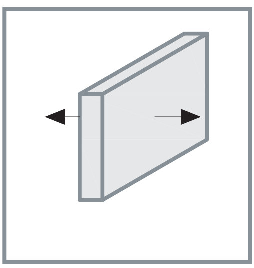
The load bearing capacity and hence the permissible wall height depend essentially on the spacing of the studs, the cladding and the dimensions of the cavity in the partition.

For specifications on permissible wall heights **refer to Appendix C2.2**

4.2.2.3 Earthquake Resistance of Non-Load Bearing Partitions

In principle it can be assumed that non-load bearing partitions are earthquake resistant. There are various expert testimonials to this effect.

As already stated, the earthquake resistance must be guaranteed for partitions in three directions:



First of all, it must be possible for the partition to transfer its dead weight, which has been accelerated in the horizontal movement, at right angles to the plane direction of the partition. For this purpose, the structure itself must offer sufficient resistance and the connections on the adjacent components must be able to transfer the arising forces in the load bearing main structure.

As shown in chapter 3.3.3, Eurocode 8 (EN 1998, design provisions for earthquake resistant structures) makes a proof procedure for non-load bearing components available. Table 4.4 shows the maximum permissible wall heights for non-load bearing metal stud partitions for various earthquake zones and surface conditions
(soil factor *S* • design acceleration a_g) for the event that the unfavorable relationship between the natural oscillation behavior of the load bearing structure and of the non-load bearing component is $T_a/T_1=1$.

In the majority of cases, this relationship however is not equal to 1 and hence the earthquake load is less.

However, the values in the table are an approach with which approximate values for wall heights can be established without analyzing the dynamic behavior. The product of the soil factor (in accordance with Eurocode 8) and the design acceleration (in dependence on the earthquake zone) is the only figure that needs to be determined.

Table 4.4 Permissible wall heights of metal stud partitions in dependence on the earthquake load

<table>
<tr><th rowspan="3">$S \cdot a_g$
[m/s²]</th><th colspan="9">Permissible wall heights in m, spacing of studs 60 cm</th></tr>
<tr><th colspan="3">W111.
single metal stud frame,
sinlge-layer cladding
1x12.5 mm</th><th colspan="3">W112.
single metal stud frame,
double-layer cladding
2x12.5 mm</th><th colspan="3">W113.
single metal stud frame,
triple-layer cladding
3x12.5 mm</th></tr>
<tr><th>CW50</th><th>CW75</th><th>CW100</th><th>CW50</th><th>CW75</th><th>CW100</th><th>CW50</th><th>CW75</th><th>CW100</th></tr>
<tr><td>1.8</td><td rowspan="12">2.75</td><td rowspan="12">3.75</td><td rowspan="12">4.25</td><td rowspan="4">3.5</td><td rowspan="3">5</td><td rowspan="3">5.75</td><td rowspan="2">4</td><td>5.5</td><td>6.5*</td></tr>
<tr><td>2.3</td><td>5.25*</td><td>6.5**</td></tr>
<tr><td>2.7</td><td>3.75</td><td>5*</td><td>6**</td></tr>
<tr><td>3.2</td><td>4.75</td><td>5.75*</td><td>3.75*</td><td>4.5**</td><td>5.5**</td></tr>
<tr><td>3.6</td><td>3.25</td><td>4.5*</td><td>5.75**</td><td>3.5*</td><td>4.25**</td><td>5.25**</td></tr>
<tr><td>4.0</td><td rowspan="2">3</td><td>4.25*</td><td>5.5**</td><td>3.25*</td><td>4**</td><td>5**</td></tr>
<tr><td>4.5</td><td>4*</td><td>5**</td><td rowspan="2">3**</td><td>3.75**</td><td>4.25**</td></tr>
<tr><td>5.0</td><td rowspan="3">2.75</td><td rowspan="2">3.75*</td><td rowspan="2">4.75**</td><td rowspan="3">3.5**</td><td>4**</td></tr>
<tr><td>5.4</td><td rowspan="2">2.75**</td><td>3.75**</td></tr>
<tr><td>5.6</td><td>3.5*</td><td>4.5**</td><td>3.5**</td></tr>
<tr><td>6.3</td><td>2.5</td><td>3.25*</td><td>4.25**</td><td rowspan="2">2.5**</td><td>3**</td><td>3.25**</td></tr>
<tr><td>7.2</td><td>2.5*</td><td>3.25**</td><td>4**</td><td>2.75**</td><td>2.75**</td></tr>
</table>

* Reduction of the anchoring distance of the circumferential perimeter runners to 0.75 m

** Reduction of the anchoring distance of the circumferential perimeter runners to 0.50 m

The standard anchoring distance of the circumferential perimeter runners amounts to 1 m.

For more information on permissible wall heights with other profiles and other stud spacing **refer to Appendix C2.2.**

Example:

Conditions:

- Soil class E: Soil factor S = 1.4 (in accordance with Eurocode 8, EN 1998-1)
- Earthquake zone III (Greece), public building:
  Design acceleration $a_g = a_{gR} \cdot \gamma_1$ = 3.6 m/s² · 1.2 = 4.3 m/s²
- Partition type:
  Partition type: W112 (single metal stud frame
  with double-layer cladding 2 x 12.5 mm)
  and CW75x50x0.6 studs

Determination of the permissible wall height:

- $S \cdot a_g$ = 1.4 • 4.3 m/s² = 6.0 m/s²

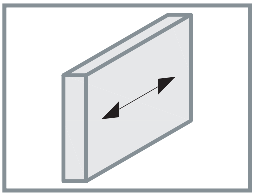
Max. permissible wall height W112 with CW75, from Table 4.4:
→ **3.25 m** (spacing of anchorings of the circumferential perimeter runners: 0.75 m)

It shows that even in the area of the non-load bearing components, the rule of thumb "less weight = less earthquake load" applies. If, without taking into consideration the earthquake load, the permissible wall height in the case of thicker cladding is greater, under earthquake loads this will have a counter-rotating effect. The cladding thickness must therefore be kept as slight as possible. Nevertheless, in order to comply with specific requirements in terms of the physical properties of buildings, you may need to use a higher quality of gypsum board, which guarantees the same performance that you would have from additional cladding layers with standard boards, but is less thick or less heavy.
Without analyzing the dynamic characteristic values, the specifications in Table 4.4 are based on the most disadvantageous ratio of parameters between the wall and the construction. The vibration behavior can, under some circumstances, also be favorably influenced by additional cladding layers. In this way, with a more precise dimensioning, the negative tendency of a greater cladding thickness is not as serious.

Table 4.5 shows the natural oscillation periods of metal stud partitions as per /4.26/ in dependence on the wall height for proof in accordance with e. g. Eurocode 8 or DIN 4149. With these values a precise calculation can be carried out, which generally leads to greater permissible wall heights, especially in the area of greater earthquake loads, greater cladding thickness, and the arrangement of the walls in the lower stories of higher buildings.

Table 4.5 Natural periods of metal stud partitions for earthquake dimensioning acc. to /4.26/

Partition height in m	Natural period of metal stud partitions in s					
	W111. single metal stud frame, sinlge-layer cladding 1x12.5 mm			W112. single metal stud frame, double-layer cladding 2x12.5 mm		
	CW50	CW75	CW100	CW50	CW75	CW100
2.75	0.15	0.10	0.07	0.18	0.12	0.09
3.00	0.17	0.12	0.09	0.21	0.15	0.11
3.25	-	0.14	0.10	0.25	0.17	0.13
3.5	-	0.16	0.12	0.28	0.20	0.15
3.75	-	0.18	0.14	0.33	0.23	0.18
4	-	0.21	0.16	0.37	0.26	0.20
4.25	-	0.23	0.18	-	0.30	0.23
4.5	-	0.26	0.20	-	0.33	0.25
4.75	-	-	0.22	-	0.37	0.28
5	-	-	0.24	-	0.41	0.31
5.25	-	-	-	-	0.45	0.34
5.5	-	-	-	-	0.50	0.38
5.75	-	-	-	-	-	0.41
6	-	-	-	-	-	0.45
6.25	-	-	-	-	-	0.49
6.5	-	-	-	-	-	0.53
6.75	-	-	-	-	-	-
7	-	-	-	-	-	-



Furthermore, the movement of the adjacent components in the direction of the plane of the partition, in particular the horizontal wall movement and the vertical ceiling deflection, must take place as non-destructively as possible. As already explained in Chapter 4.1.4, drywall systems are capable of taking very large movements with virtually no destruction. It is in this area that the planner or investor must decide what damage extent he is prepared to accept for the various degrees of earthquake intensity.

If greater movement of the adjacent components is to be taken without damage, then it is possible to fit sliding connections.

In sensitive areas in which serviceabilty must also be maintained even after a strong earthquake (e. g. operating rooms, communication centers, etc.) you can generally minimize the damage risk by using sliding connections. The following chapter includes examplary construction details.

Permissible wall heights, bracket loads, etc. **refer to Appendix C2**

More information /4.5/, /4.6/, /4.7/, /4.8/ (refer to the bibliography)

4.2.2.4 Constructional Information on the Earthquake Resistance of Non-Load Bearing Partitions

As already mentioned in the previous chapters, non-load bearing partitions can be connected sliding to ceilings and flanking walls in order to minimize the extent of the damage.

In doing so, the calculated story movement gives the required freedom of movement for the wall connections; the calculated ceiling deflection gives the required freedom of movement for the ceiling connections.

The connections can be executed as follows:

■ Sliding ceiling connections (deflection heads)

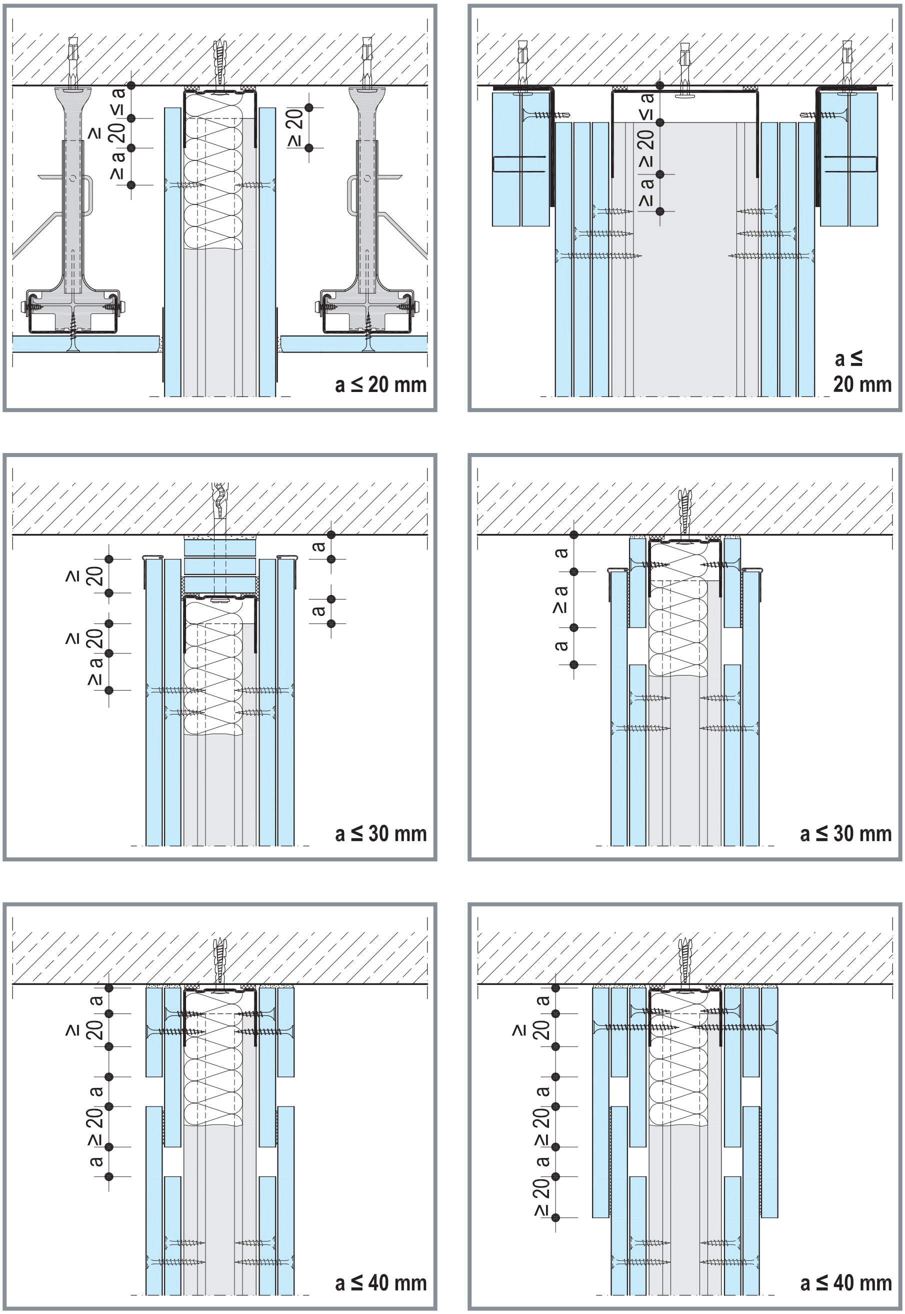


Figure 4.23 Sliding ceiling connections, room for movement 10 - 40 mm

■ Sliding wall connections

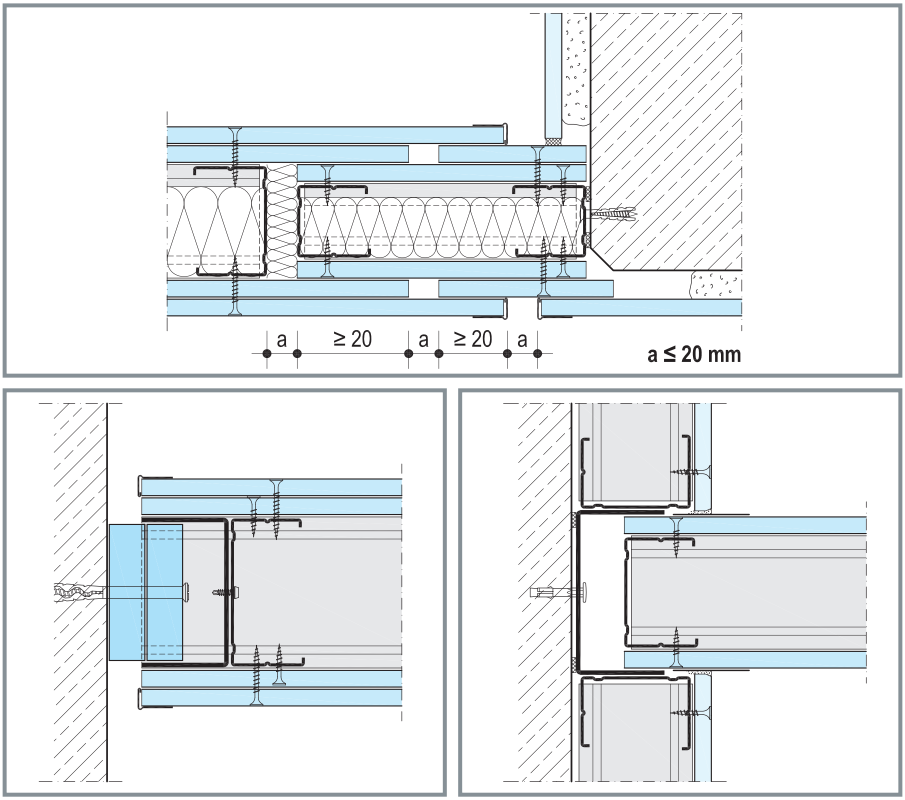


Figure 4.24 Sliding wall connections

Usually the boards are fastened not only to the metal studs but also to the perimeter runners. In earthquake areas the screws should not be attached to the floor connection runners, as additional stresses can occur here, which in turn can cause cracks.

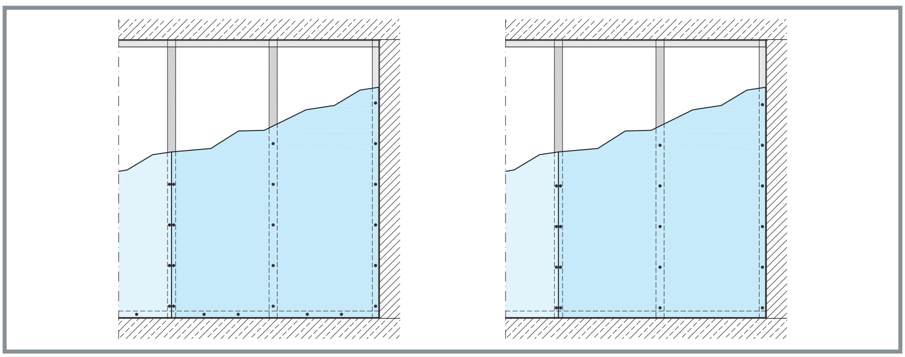


Figure 4.25 Fastening of cladding with and without screw attaching into the floor connection profiles

4.2.3 Structural Wood Frame Panels

Structural wood frame wall panels are walls that have a bracing function for horizontal and partially also vertical loads in wall levels. In doing so, the gypsum board cladding braces the frame for horizontal loads; the vertical loads are solely supported by the substructure. Structural wood frame wall panels are particularly used in prefabricated buildings. The structure of a wood frame is similar to that of a non-load bearing partition with a wooden substructure. You must however observe deviating design details like special requirements on fastening materials, fastening spacings and connections. The cladding can also be used on just one wall side in order to transfer the forces. The wood studs of the wood frame are called ribs. The gypsum boards are arranged in rows at right angles or parallel to the ribs. The edges of the boards are connected in a rigid manner on all sides with the substructure. While lowering the permissible horizontal force, the cladding may be applied with one horizontal joint if the board edges are connected shear-rigid on all sides.

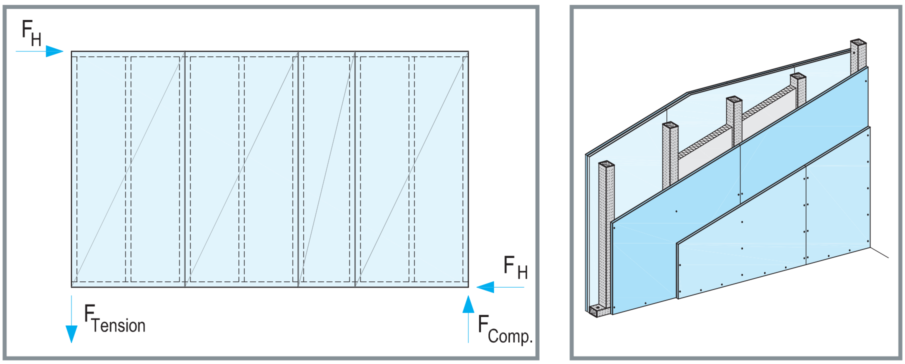


Figure 4.26 Structural wood frame wall panels, load diagram and construction

As structural wood frame wall panels are load bearing components, they must be certified in accordance with the constructional supervisory regulations. Up until now, these certifications have been covered via special permissions of the manufacturer's of the gypsum boards for this application area.

With the introduction of the DIN 1052:2004-08 in Germany, there is now a computational proof procedure whereby a considerably greater flexibility is given in the structural design of structural wood frame wall panels. The gypsum board manufacturer, Knauf, also offers the user a practical brochure giving dimensioning tables based on the proof procedure of the DIN 1052.

Proof of load bearing capacity **refer to Appendix C2**

More information /4.9/, /4.10/ (refer to the bibliography)

4.2.4 Lightweight Steel Constructions

The lightweight steel construction is similar to that of wood frame panel constructions. Buildings made of lightweight steel constructions consist of a steel skeleton to transfer vertical loads and bracing wall and ceiling panels which deflect horizontal forces. These wall and ceiling panels are made up of thin sheet metal profiles and a bracing cladding comprising gypsum or gypsum fiber boards.
In doing so, it is made use of the fact that the cladding protects the stability-endangered thin-walled sheet metal profiles from failing in terms of stability.

Depending on the system, lightweight steel constructions offer a multitude of advantages:

- Very low weight
- Outstanding strength – dead weight ratio
- Dimensional consistency
- Good physical acoustics properties
- The possibility of a speedy construction site assembly
- Considerable recycling and reuse potential of all the materials used in the system
- Depending on the board material, non-combustible (building material class A), no increase in the fire loads via the structure
- Sophisticated joint and connection technology
- Suitable for prefabrication
- Suitable for building extensions (adding floors) with limited load bearing capacity

Lightweight steel constructions are used in buildings with a low number of floors, usually up to four stories, as well as façade components and room-in-room systems.
Such a room-in-room system, which is installed within existing buildings, can be used to explain the functionality of the lightweight steel construction in a very simple way. Figure 4.27 illustrates the basic load bearing system with columns to transfer the vertical forces and circumferential perimeter profiles to accommodate the ceiling structure and to transfer the ceiling loads in the columns as well as for the purpose of fixing the wall frames.
This load bearing system can deflect the vertical loads, but it is unstable in terms of horizontal loads and does require bracing.

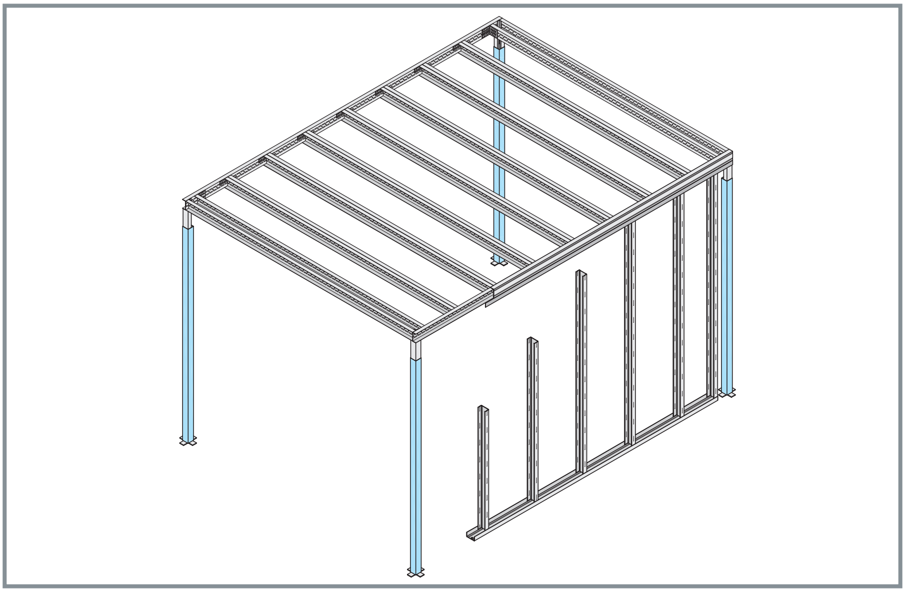


Figure 4.27 Load bearing system – room-in-room system

The bracing is ensured from the gypsum board cladding on the walls and ceilings. The ceiling is hence free-spanning and must be dimensioned for the respective moveable loads in dependence on the use.
In the case of room-in-room systems, it is also possible to rule out a load on the ceiling, so this is designated not accessible.

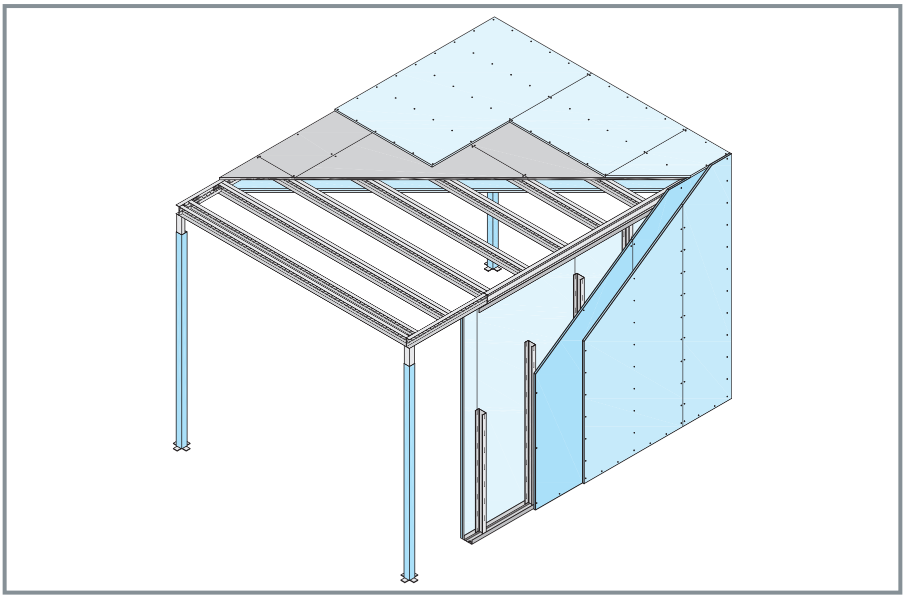


Figure 4.28 Room-in-room system with bracing ceiling and wall structure

According to the same basic principle, complete buildings are also designed in a lightweight steel construction method.

Figure 4.29 Lightweight steel construction

In doing so, the ceiling panels can be installed with a bracing layer at the top. This layer is made up of trapezoid sheet metal or very strong gypsum fiber elements, the underside cladding is made up of gypsum boards.

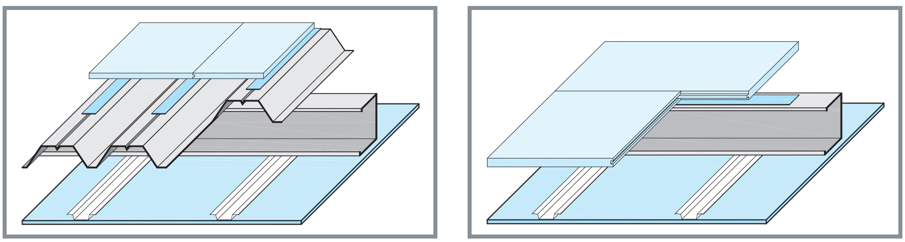


Figure 4.30 Load bearing ceiling structures of lightweight steel buildings

The cladding on external walls is executed using cement boards, gypsum boards that are suitable for exterior walls or plywood boards. If required, a heat insulation compound system or plaster can be used.

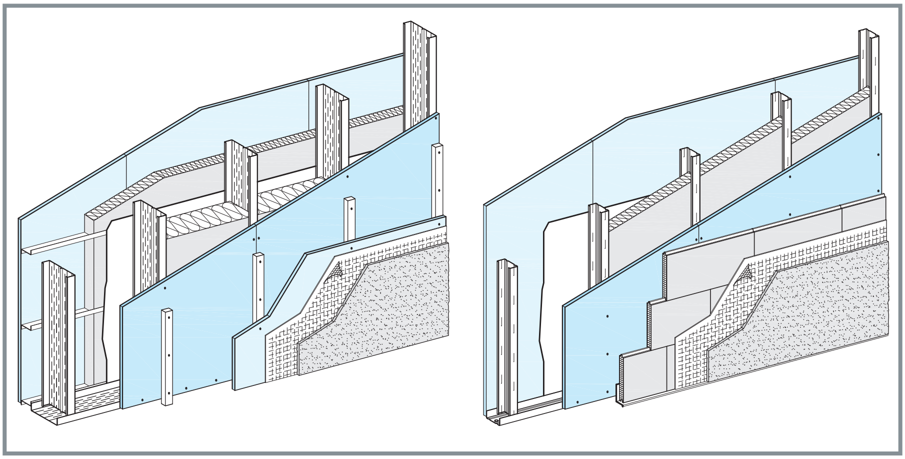
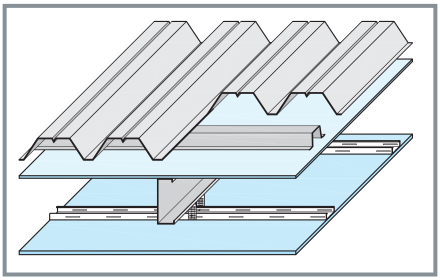


Figure 4.31 External wall constructions of lightweight steel buildings



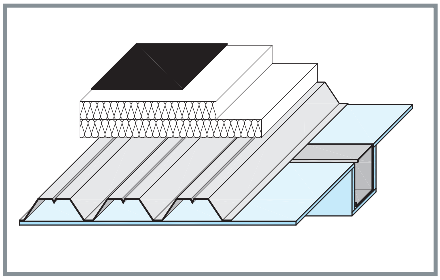


Figure 4.32 Roof constructions of lightweight steel buildings

Trapezoid sheet roofs with or without additional insulation or sealing layers or totally normal rafter roofs are used for roof structures.

Earthquake resistance of lightweight steel structures

On account of the low weight and, in relation to the weight reduction, a not so heavily reduced load bearing capacity compared to solid construction structures, and the considerable ductility thanks to the building materials and connections used, the earthquake resistance is very high. However, particular attention must be paid to the design of the connections of those individual components that are attached to each other, as it is here where the load transfer has to be ensured in accordance with the assumptions used for computational load bearing structure model.

At the moment, for all thin gauge profiles (cross-section Class 4 as defined by Eurocode 3, EN 1993), the proof of the stabilization against the stability failing must be guaranteed, as, in the case of the stability failing, no energy dissipation takes place and hence the dynamic behavior is significantly less favorable. In the computational proof, this fact is taken into account by an unfavorable behavior factor q, which creates greater earthquake loads. According to Eurocode 8 (EN 1998) you can calculate only with a behavior factor $q = 1.5$, if the essential seismic components belong to cross-section Class 4.

However, this doesn't have to be disadvantageous, as, in the case of a special load earthquake, a safety factor of 1.0 must be applied, and the reduced weight of lightweight steel structures signifies less earthquake load. Compared to the wind loads that are to be applied, which have a greater safety factor, therefore the earthquake load case is generally not decisive, at least in the regions of lower and medium seismic activity.

As long as the research results specified in 3.1.2 are not given, the classic calculation with $q = 1.5$ and spectral values of the plateau (maximum) are recommended. Thanks to the low weight, the earthquake loads can still be handled.

It may however be necessary to take into consideration deviating national regulations or manufacturer's instructions.

More information /4.11/, /4.12/, /4.13/ (refer to the bibliography)

4.2.5 External Walls and Ceilings

Light construction can also effectively and securely replace solid construction methods in the area of the envelope of the structure. Of course, here the climatic conditions and loads, which have an effect on the component, are more sophisticated than those experienced in the interior area. The use of gypsum boards in the external area is therefore possible but only in a very restricted manner. In some countries, special gypsum boards are produced and used for external walls, however, it is much more favorable to use cement wallboards, like for example, AQUAPANEL® Cement Board Outdoor.

External wall systems made using cement wallboards like AQUAPANEL® Cement Board Outdoor securely protect buildings from wind and weather, noise and fire. The application possibilities extend from the wooden frame through metal stud constructions and right through to the encasement of steel and solid construction components. Cement boards can also be used to create ceilings in the external area, their earthquake resistance is the subject matter of current research.

Similar to internal partitions, external walls can be produced with a substructure made of metal stud profiles or wood frames. The external cladding is carried out using cement wallboards, the inner cladding usually consists of gypsum boards.

It is also possible to install curtain walls. This does assume a solid wall is being used as a fixing base. As, in connection with the earthquake resistance, with lightweight constructions it is more practical to produce the entire external wall as drywall construction, information is given only on external wall constructions in the following.

More information on curtain walls with cement wallboards is available from the manufacturer.

The construction of an external wall can be done as single shell type, i. e. similar to internal metal stud or wood frame partitions or as double shell type, i. e. two independent furrings are erected opposite each other.

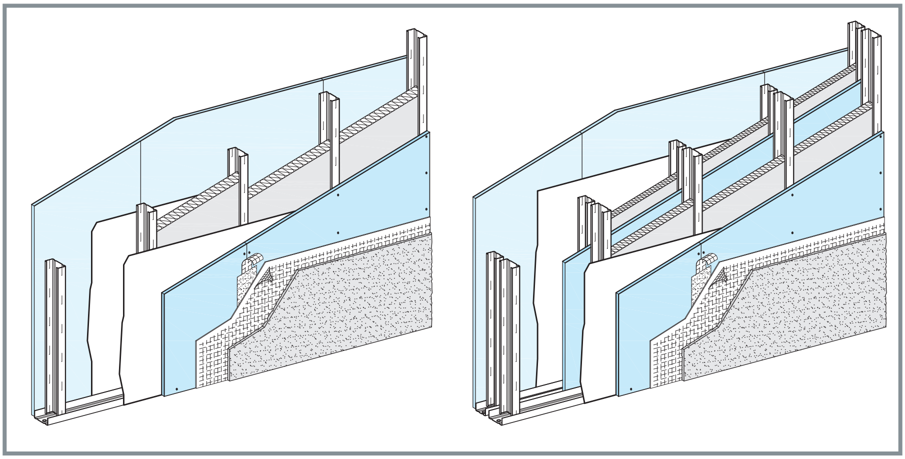


Figure 4.33 External walls with façade cladding made of AQUAPANEL® Cement Board Outdoor, single shell and double shell

These external wall constructions can be installed between floors or in front of floors in a so-called set assembly or preset assembly. "Between floors" assembly means that the circumferential edge connections are anchored directly to the supporting structure. In the case of "in front of floors" assembly, the wall construction is fixed to angle profiles that are fitted to the outside of the supporting structure.

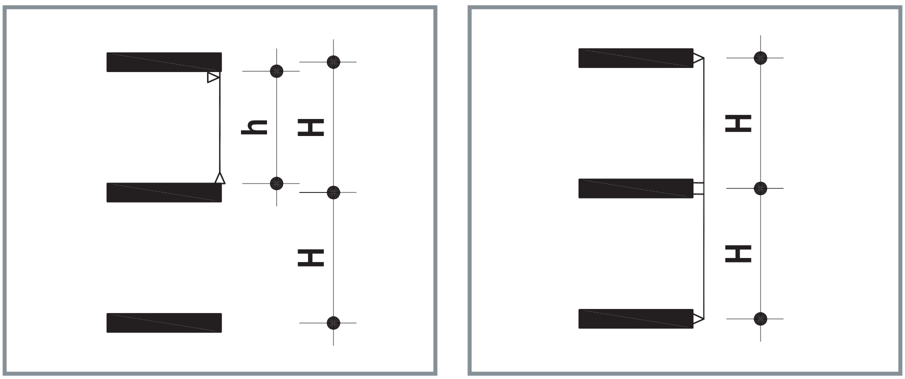


Figure 4.34 Installation between floors or in front of floors

For the area of earthquake resistance, again only the assembly between floors is recommended as the vertical stimulation can be transferred without any problem to the supporting structure.

To create a water barrier, a membrane that is not impervious to water vapor is inserted under the cladding made of cement wallboards, such as, for example, AQUAPANEL® Tyvek StuccoWrapTM, in order to avoid the penetration of water to the substructure and to deflect the water that has penetrated as a water-carrying layer.

For the metal profiles being used, you must take into consideration the increased corrosion protection requirements, in accordance with the local climatic conditions and constructional supervisory requirements.
After fastening, the board joints are filled and then the boards are entirely covered with an exterior plaster system, e. g. AQUAPANEL® plaster system (refer also to Chapter 4.3.4.3). You could also use a thermal insulation compound system or cladding. However, in this case, ensure that these claddings have been permitted by the manufacturer for use on façades made of cement wallboards in earthquake areas.

Load bearing capacity
Alongside the dead weight, walls and ceilings in the external area, in dependence on the height and geometry of the structure, are mainly stressed by the effects of wind and, in earthquake areas, by earthquakes.
As a result, the substructure in terms of its dimensions and cross-sectional values, must be adapted to cope with the effects of wind pressure, wind suction, and if necessary, earthquakes. In some cases, it is necessary for the structure to cope with snow and ice loads.
The individual elements of the substructure must be proven in their function in terms of construction. The proof of the serviceability has to be ensured by restricting the deformation to *max.* $f = l / 500$. The wall heights listed in Appendix C4 have been established accordingly for the specified constructions.
For the external walls, a general proof of structural stability and serviceability must be provided. In doing so, the structural engineer takes into account the dead weight of the components made up of board weight and plaster weight, the wind loads, e. g. in accordance with DIN 1055-4 and, if necessary, the earthquake loads, e. g. in accordance with Eurocode 8 (DIN EN 1998). Wind loads differ depending on the structural shape, position and height. In addition, the planner establishes the width of the border area in which the façade is subjected to considerable wind suction. In the case of construction heights in excess of 8 m, the distance of the metal or wood stands must be reduced in the corner or edge area from 600 to 300 mm. The width of the corner or edge area lies between 1 and 2 meters.
Special loads resulting from, for example, advertising signs, external wall planting or sunblinds, must be transferred independently from the cladding made of cement wallboards to the load bearing substructure and primary structure and, if necessary, must be taken into account in the proof of structural stability. Light loads, e. g. from the dead weight of decorative elements, decorative profiles and lighting, can be anchored using at least two metal cavity dowels in the cement wallboards.
The distance of the dowels to one another should amount to at least 75 mm. For wall

constructions, the size of the light load must be restricted to maximum 25 kg. This refers to individual, non-extensive single loads.

Earthquake resistance of façade systems

In the case of an earthquake, the users within the building instinctively tend to flee to the outside. Parts of the façade or external wall elements that fall down represent a considerable potential danger.

For this reason, with regard to earthquake resistance, the earthquake behavior of external walls is of particularly considerable significance. By means of careful and individual planning it must be ensured that parts cannot fall down.

The weight advantage of light construction methods is also of an advantage here. External masonry walls represent an enormous potential danger, their failure and hence the falling down of a great number of bricks is very probable in the event of an earthquake. Light construction structures have a reduced probability of failing and, should they fail, they will cause less damage.

However, to comply with these advantages, special characteristics must be observed for the use of light external wall structures in earthquake areas.

In doing so, it is particularly a case of detailed solutions like connections and anchorings for which the expected vibration and deformation behavior of the building must be taken into consideration.

The dimensioning tables with the permissible wall heights and substructure spacings by no means replace the careful planning of connection and anchoring details.

These details should be portrayed in such a way that, in the event of deformations caused by earthquakes, the load bearing structure transfers as little as possible of the load to the external wall structure.

This is enabled by arranging horizontal movement joints between all the floors. In doing so, a positive side effect is that the risk of cracking under temperature or wind loads is reduced.

Furthermore, the dimensions of the individual façade areas are minimized, which also contributes to reducing the risk of damage. For example, the German standard DIN 18516 requires a restriction of the part areas to maximum 50 m².

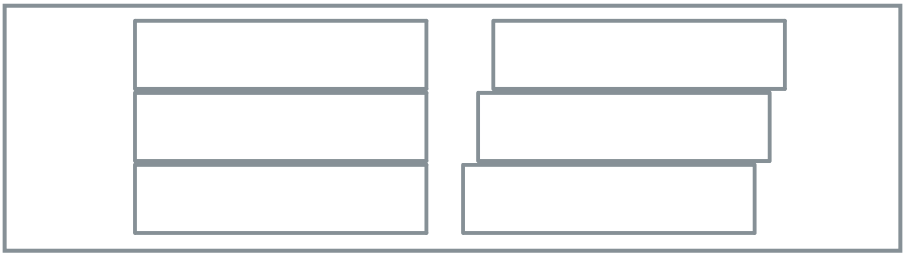


Figure 4.35 Displacement behavior of façade elements with horizontal movement joints

In order to keep the external wall in a load-free state if the load bearing structure deforms, the first studs should be positioned alongside a column at a distance in accordance with the expected floor displacement. The boards are not screw attached into the upper perimeter runner so the ceiling is able to slide across the upper connections and the column can deform in the direction of the plane of the wall, without transferring loads to the light construction.

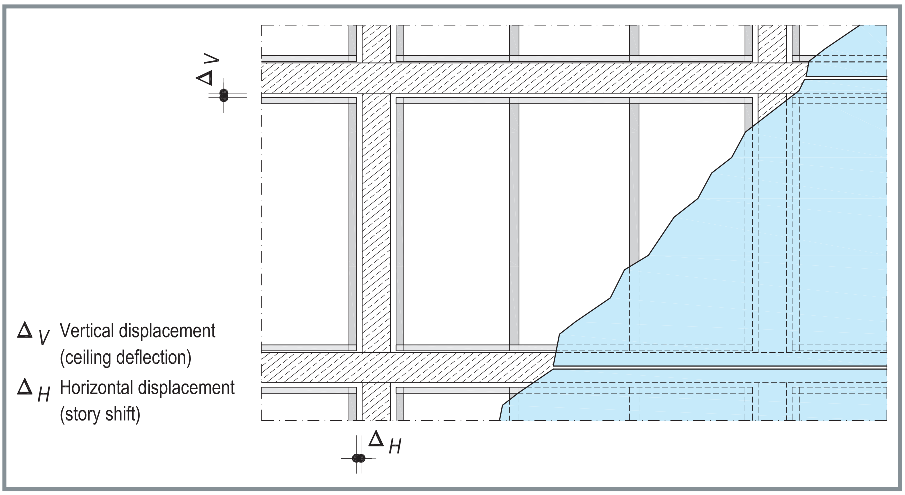


Figure 4.36 Design of the circumferential connections

Examples of horizontal separation joints connected to a sliding ceiling are shown in the details in Figure 4.37. The maximum cantilever of 150 mm of the cement wallboard must be observed. In the case of a double shell structure, additional reinforcement is required on the inside of the substructure in the plane direction of the wall. This can, for example, be ensured by additional cladding on the inside or a crosswise tensioning by means of slotted strips.

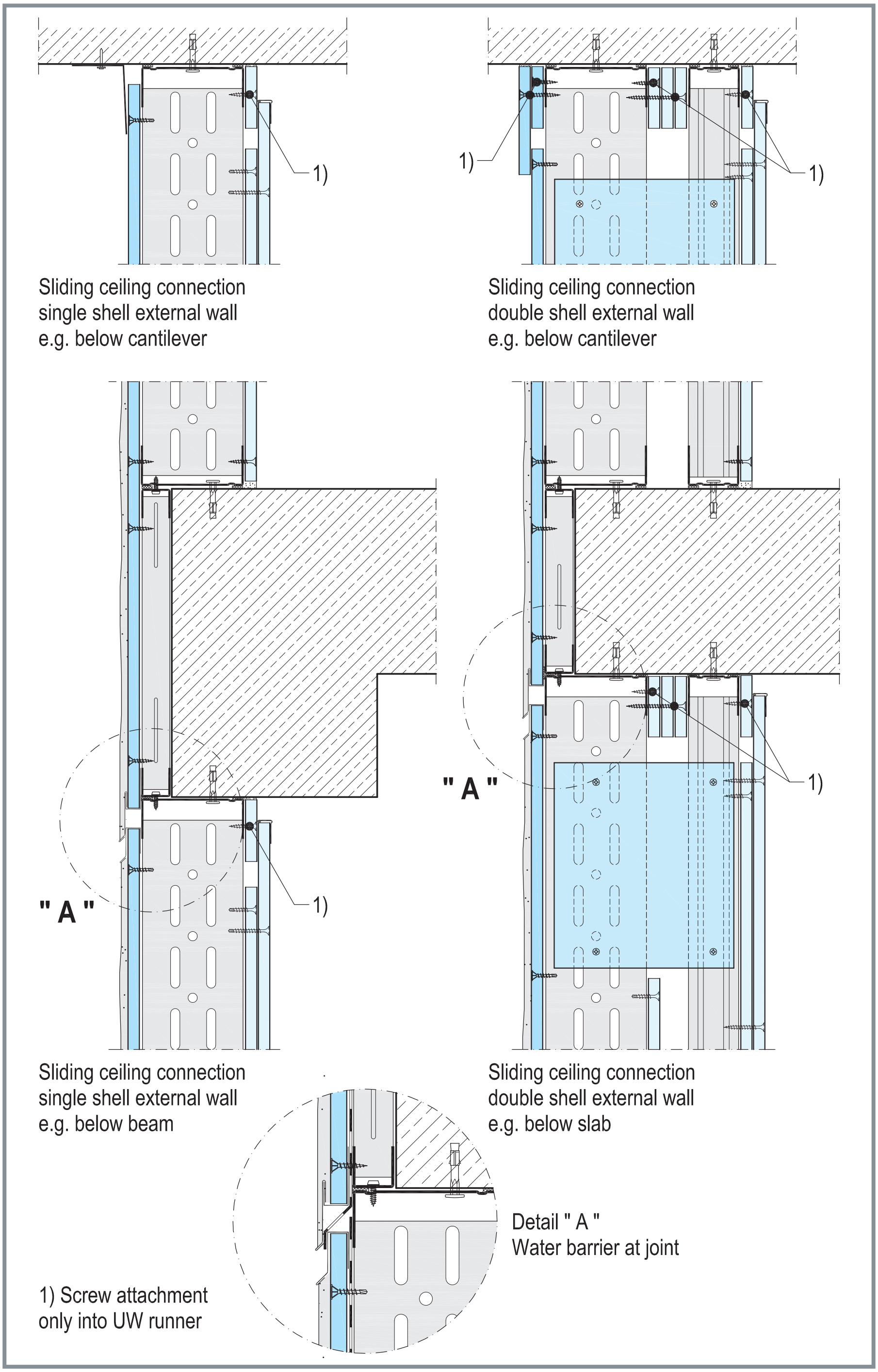


Figure 4.37 Horizontal movement joints of external walls, examples

More information /4.14/, /4.15/ (refer to the bibliography)

4.2.6 Drywall Floor Constructions

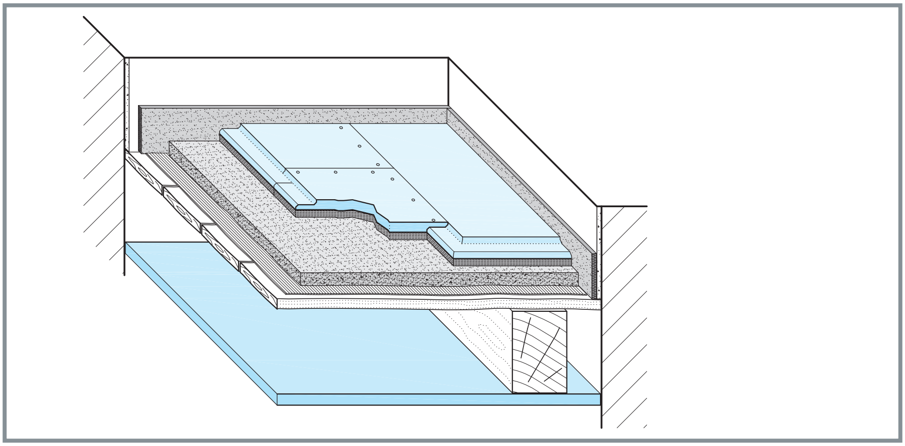


Figure 4.38 Floor construction, example: Pre-fab screed

Floor constructions made of gypsum boards can be designed in three different ways: Prefabricated screeds, hollow floors and raised access floors.

While prefabricated screeds are executed as dry floor compositions with or without insulation or as heating floor screeds; hollow floors and raised access floors have an additional installation level. Raised access floors must be individually dimensioned for use in earthquake areas and design details must be planned especially for transfer of load. Therefore, pursuant to this book, only the variants prefabricated screed and hollow floors will be considered.

Description of Construction

1) Pre-fab screed

Prefabricated screed consists of gypsum fiber panels, gypsum or cement boards, which are placed directly on a foundation made of, for example, bulk leveler or insulation.

Gypsum fiber prefabricated screed units consist of, for example, two gypsum fiber boards that are bonded to each other. The slight offset gives a tier edge on which the units can be connected to each other.

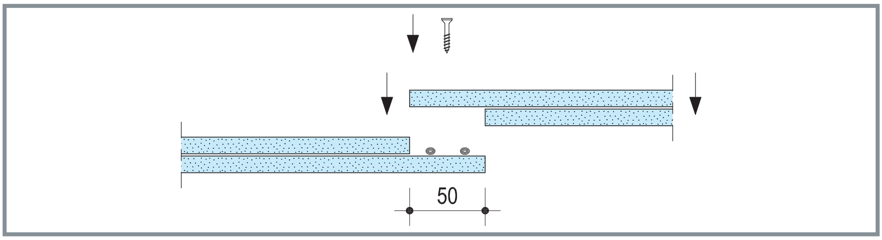


Figure 4.39 Prefabricated screed made of agglutinated gypsum fiber units

With modern manufacturing technologies, the gypsum fiber or cement boards are monolithic, i.e. fabricated from one cast, the tier edge is milled.

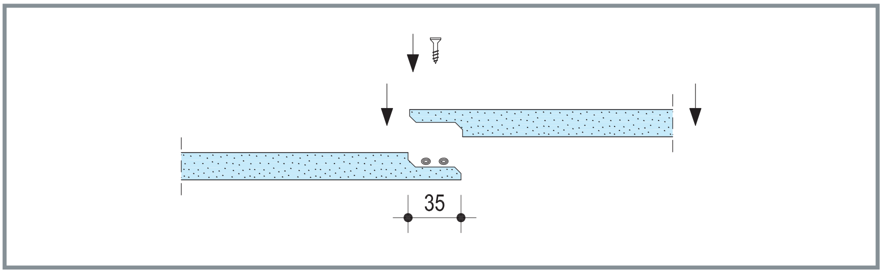


Figure 4.40 Prefabricated screed made from monolithic gypsum fiber unit

The third possibility of producing prefabricated screeds is the multi-layer application of special gypsum boards. The board layers (usually 2) are connected and stapled to each other at the construction site.

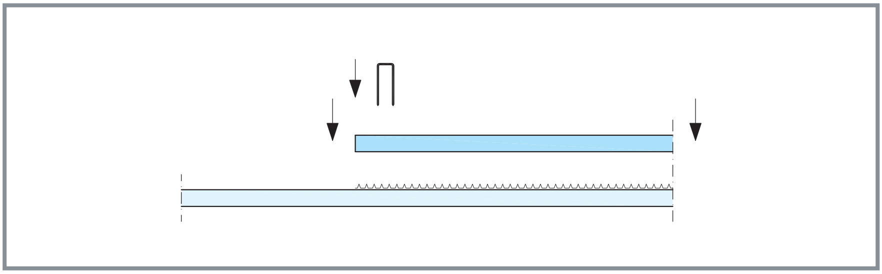


Figure 4.41 Prefabricated screed made of two layers of special gypsum board

Prefabricated screeds are particularly advantageous if used in reconstruction work or for change of use, as with a dry building method and low weight with low installation heights you can quickly and practically carry out floor restoration of the highest quality. In doing so, the sound insulation and fire protection of the respective overall ceiling structure in connection with the basic ceiling is considerably improved.
It is also possible to implement this as a heating screed.

2) Hollow floors

Hollow floors consist of shuttering panels made of gypsum or gypsum fiber boards, which are directly supported on height-adjustable pedestals. The shuttering panels serve as formwork for the actual flooring, e. g. a self-leveling screed. With Knauf GIFAfloor you have at your fingertips a gypsum fiber material that represents the shuttering panels and the flooring in one. Hence the surface covering can be applied directly to the gypsum fiber boards.
Access to the cavity space is only possible via a service hatches or channels.

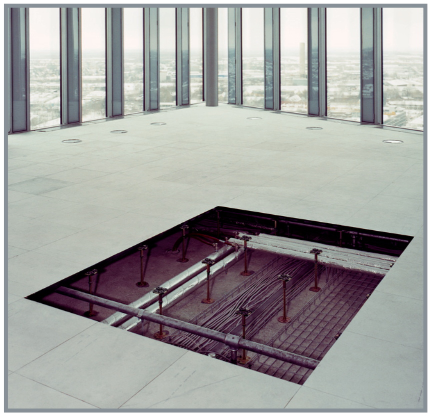


Figure 4.42 Hollow floors

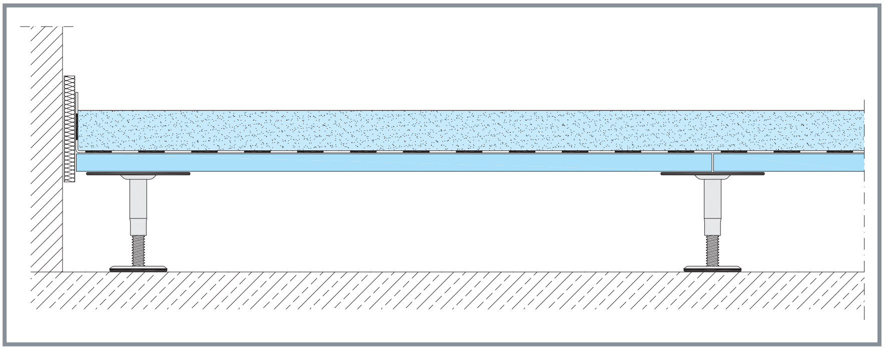


Figure 4.43 Hollow floors with shuttering panels made of gypsum boards and a flooring made of self-leveling screed

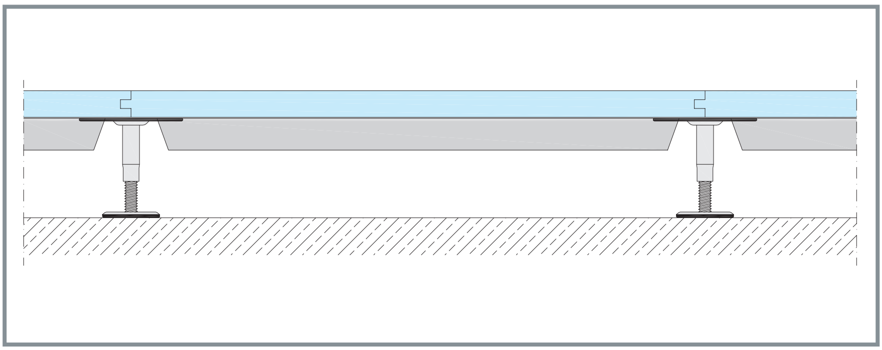


Figure 4.44 Hollow floors made using Knauf Integral GIFAfloor with service hatch F30/F60

Floor structures must always be separated from the vertical components, like for example, adjacent walls, columns, etc. in order to avoid impact sound transference. This applies both to wet screeds as well as all dry floor structures.

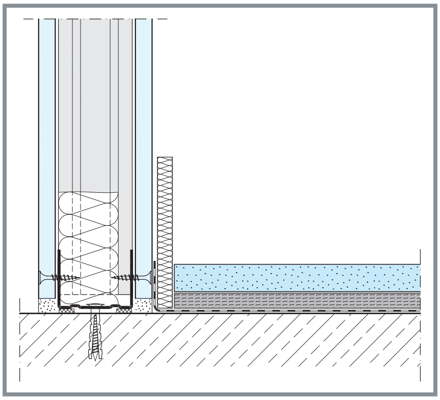


Figure 4.45 Edge detail for prefabricated screeds

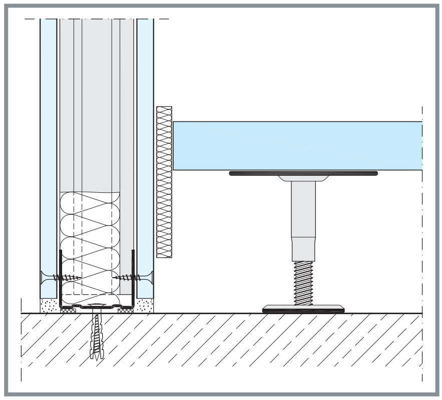


Figure 4.46 Edge detail hollow floor

If floor structures are used in damp areas, an all-over sealant must be applied so as to seal against splash water, and the respective wall connections must be sealed using sealing tape. You should also use a sealant when laying tiles in a bed of mortar directly on the gypsum fiber boards, as considerable moisture is applied with the mortar. Appendix B5 includes more information on use in damp areas.

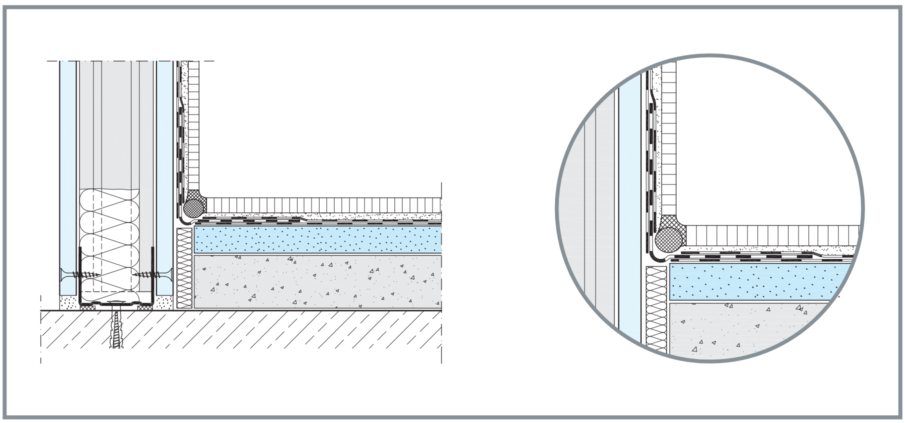


Figure 4.47 Floor sealing in damp areas, example prefabricated screeds

To compensate for any unevenness of the substrate and to ensure a continuous bedding of the prefabricated screed units, you can insert a load bearing dry bulk leveler, generally consisting of expanded perlite, under the prefabricated screeds. This also improves footfall sound insulation. The necessary load bearing capacity must be proven.

Load bearing capacity

The loads on floor constructions are working loads as a variable load or live load. Depending on the purpose of the floor areas, the pertinent standards, e. g. Eurocode 1 (EN 1991) and standards DIN 1055 “Action on structures” specify area loads and single loads, according to which the floor constructions should be dimensioned.
In the case of prefabricated screeds, the load bearing capacity is determined by the thickness of the prefabricated screed units as well as the substructure (insulation, bulk leveler, additional layers of gypsum boards).

In the case of hollow floors, the spacings of the pedestal grid, the load bearing capacity of the individual pedestals as well as the load bearing capacity of the gypsum or gypsum fiber boards, in dependence on their thickness, are the decisive parameters for the load bearing capacity of the floor construction. Testing and classification of the load bearing capacity is carried out in accordance with EN 13213 (hollow floors).
Note: The basic ceiling must be able to support the loads from the hollow floor pedestals.

Earthquake resistance of drywall floor constructions

Floor constructions with prefabricated screeds are not subjected to any particular loads in the event of earthquakes, so it is not necessary to dimension for earthquakes. They provide sufficient load bearing capacity against falling objects. There is no potential danger of these structures failing, which is very unlikely anyway.
However, with their low weight they make a positive contribution to earthquake resistance.
This also conditionally applies to hollow floor constructions as long as these are only stressed with standard live loads for office or living areas. Greater single loads should be transferred directly to the basic floor. In principle, it is recommended to obtain proof of structural stability from a structural engineer.

More information /4.16/, /4.17/, /4.18/ (refer to the bibliography)

4.2.7 Special Applications

Alongside requirements like fire protection and sound insulation, which are commonly placed on drywall systems, there are also a series of special application areas.
These partially have no direct reference to earthquake resistance, however, the aim here is to illustrate the scale of requirements that can be fulfilled using earthquake-resistant drywall structures.

Fire walls
The significance of fire protection also in association with earthquakes has already been explained. Fire walls separate fire compartments and, in addition to a fire resistance class, have a resistance capability against mechanical loads, i.e. in this case, among others, against impacts of high energy (3000 Nm impact activity on 400 cm² surface) as a protection against parts that fall down or around in the event of a fire. This resistance is ensured by a layer of steel sheeting between each cladding layer on both sides of the wall as well as a reduced spacing of studs of 30 cm. Drywall fire walls are characterized by their slender structure.

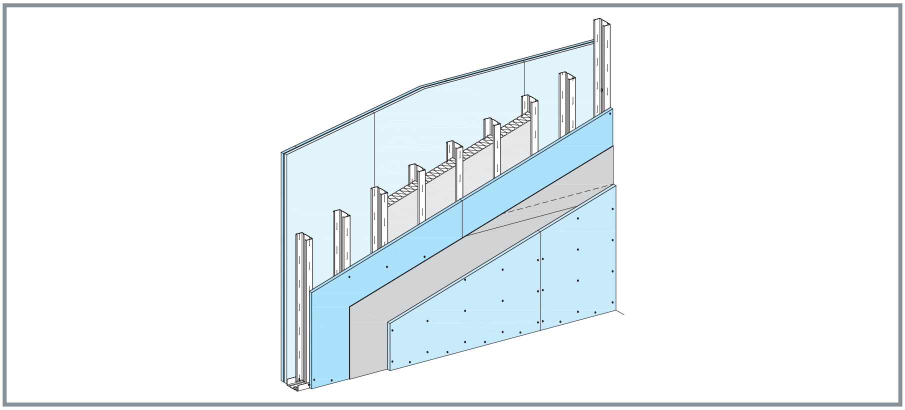


Figure 4.48 Fire wall

More information /4.19/ (refer to the bibliography)

Encasements for columns and girders
Load bearing components made of steel, like columns and girders, have a high load bearing capacity and, on account of their plasticity, are particularly suitable for the load bearing structures of higher buildings in earthquake areas.
However, one of the material's disadvantages is that when temperatures increase it quickly loses strength and rigidity, so an effective fire protection of these components must be ensured. This is why it is particularly suitable to use cladding made of gypsum boards - on account of its fire protection properties.

Appendix B1.2 explains the temperature-insulating effect of the evaporated gypsum water in the event of a fire stress.
This cladding is of low weight and simple to fit. A positive “side effect” is the look of this component in the range of visibility and the high-quality drywall surfaces.

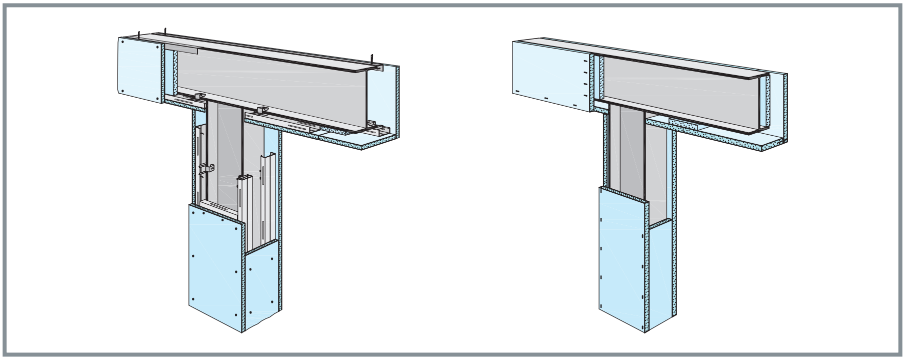


Figure 4.49 Fire protection encasement of steel columns and girders

The sensitivity of steel profiles to heat stress depends on the relationship between their surfaces and their cross-sectional area, the *U/A* value.
It is conceivable that a reduced cross-sectional surface, which at the same time offers the thermal effects a greater area of attack, quickly reaches critical temperatures in the overall cross-sectional area.
Correspondingly, the required fire protection properties of the cladding depend on this *U/A* value.
The documentation of the manufacturers include calculation aids for the *U/A* value for many profile geometries as well as dimensioning aids for the respective required cladding thickness.
In doing so, the length of time the respective component must be protected before it fails in the event of a fire is also of importance. This duration is specified by the fire resistance class, which is specified by the planner, whilst taking into account constructional supervisory guidelines for the respective application area of the component.
Refer to Appendix B1 for more information on the topic of fire protection.

On account of their combustibility, load bearing components made of wood are sensitive to fire stress and can, analogous to the encasement of steel columns and steel girders, be protected against fire load by means of gypsum boards for specified fire protection duration (fire resistance class).
However, in the case of wood components, the required cladding does not depend on the geometry of the component, as the temperature effect does not play a role in the supporting behavior, it is the combustible wood that must invariably be protected against fire.

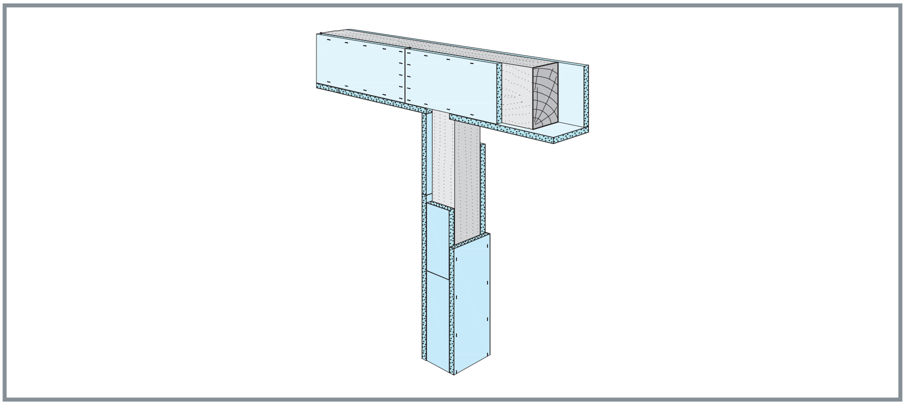


Figure 4.50 Fire protection encasement of wood columns and beams

More information /4.20/ (refer to the bibliography)

<u>Fire protection in engineering terms</u>

It is not only individual components, but also entire load bearing structure parts that can be protected against fire by cladding made of gypsum boards. This makes it possible to use the preferred materials for earthquake areas of steel and wood even in the case of fire protection requirements.

As, in these cases, an individual dimensioning of the board thickness as well as the specification of load bearing details must be carried out by the planning engineer and not carried out on the basis of the standard specifications of the gypsum board manufacturers or the standards, it is referred to this as “fire protection in engineering terms”.

Because of its fire protection advantages, in this application area predominantly gypsum boards with fleece reinforcement (fireboard) are used.

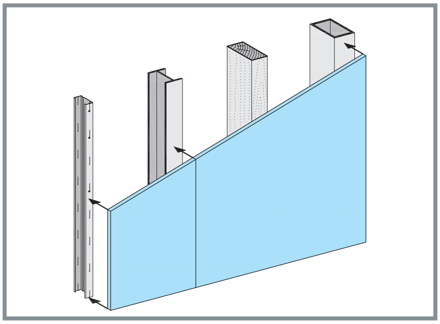


Figure 4.51 Verticale fire protection cladding

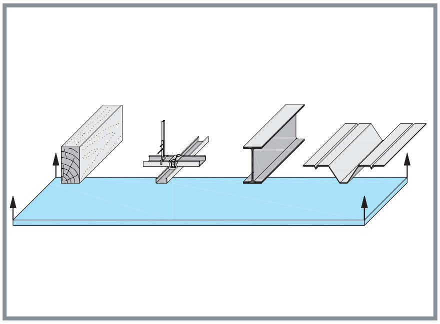


Figure 4.52 Horizontal fire protection cladding

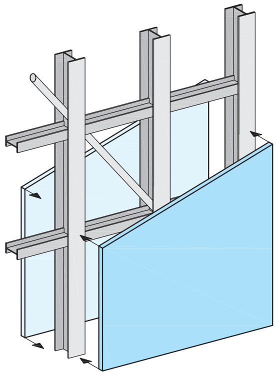


Figure 4.53 Fire protection cladding of steel load bearing structures

In doing so, the respective fire resistance class is guaranteed by the cladding of one side for the clad component that is to be protected, which means greater demands on the cladding than it would in the case of the room-enclosing fire protection of non-load bearing walls, which guarantees the fire protection through the entire structure between two rooms.

Table 4.6 Fire resistance duration of fireboard cladding

30 Min.	60 Min.	90 Min.	120 Min.	180 Min.
20 mm Fireboard	30 mm Fireboard	40 mm Fireboard (double-layer)	60 mm Fireboard (double-layer)	70 mm Fireboard (triple layer)

A common application is the fire protection cladding of steel load bearing structures. In doing so, the steel structure must be executed with a cladding on both sides made of Knauf Fireboard (fleece-reinforced gypsum boards) with a total board thickness on each side in accordance with Table 4.6.

- Without substructure: Fastening the Fireboards directly to the steel load bearing structure using drywall screws (sheet thickness up to 2.25 mm) – note the permissible span of the Fireboard.
- With substructure: Fastening the Fireboard to the substructure (e. g. CD 60x27, hat-shaped channels, resilient channels), which are fixed to the steel load bearing structure.

Such individual constructions must be determined with the manufacturer of the gypsum boards as well as the construction supervisory board.

More information /4.21/ (refer to the bibliography)

Ducts

Earthquakes usually result in fires. Especially in sensitive buildings like schools, hospitals, buildings with lots of people, etc. it is then important to design installations like electrical cables and ventilation in a manner that protects them against fire and so that the function of the installations (lighting, ventilation, electrical devices in hospitals, etc.) is guaranteed in the event of a fire and the fire is prevented from spreading via the installation ducts. Ventilation and cable ducts can be made of gypsum boards to guarantee the fire protection.

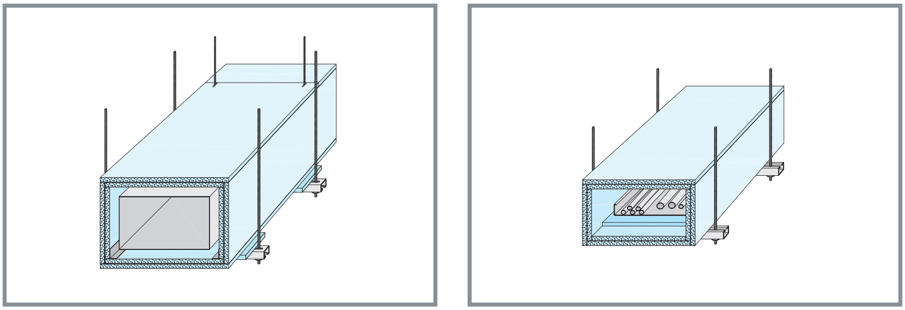


Figure 4.54 Cable and ventilation ducts made of gypsum boards

In doing so, the various principles of fire protection, which decide on the design of the construction, have to be taken into account.

In the case of cable ducts which, in dependence on the load, have the power and media cables inside them running on cable trays or directly on the duct bottom, there are two fire cases.

1) The fire originates in the cable duct, the cable therefore burns and the surrounding room must be protected from fire. In this case, the ducts are called “I ducts”, whose fire class is specified with I 30, I 60, etc.
2) The fire originates in the room through which the cable duct is running and the cable should be protected from the fire in order to maintain the function of the cable, e. g. for emergency lighting, emergency warning systems, medical devices, etc. despite the fire. These ducts are called “E ducts”, whose fire resistance class is specified with E 30, E 60, etc.

For ventilation ducts on whose insides no fire is initially expected, as no flammable materials and no fire sources are located in the duct and the function also plays no security-relevant role, the task is to avoid fire spreading from one room to the neighbouring room via the duct. Their fire resistance classes are be determined with L 30, L 60, etc.

More information /4.22/ (refer to the bibliography)

Escape tunnel

If escape routes run through expansive areas in which it is possible for the fire to spread, the escape routes can be secured by means of free-standing escape tunnels. These escape tunnels have a respective fire resistance duration and can resist the impact loads of rubble.

The structure consists of a load bearing structure to deflect vertical forces which are braced by a wall structure made of a metal studs and gypsum boards as well as a free-spanning ceiling.

The structure is additionally braced with external diagonal struts.

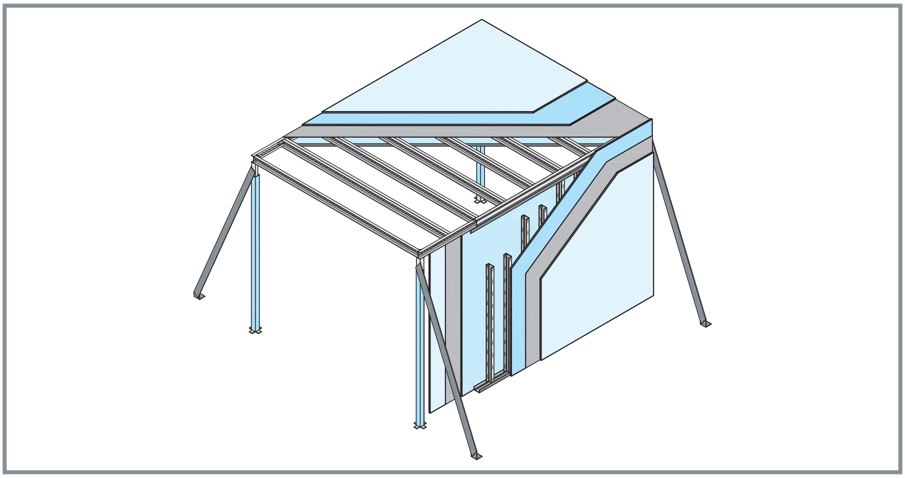


Figure 4.55 Escape tunnel

More information /4.11/ (refer to the bibliography)

Radiation protection

Room-enclosing components of rooms in which X-rays are emitted, must shield the adjacent rooms from these X-rays.

The heavy concrete components that were used for the purposes of radiation protection in hospitals and doctors´ consulting rooms before can today be easily, practically and flexibly replaced by drywall radiation protection systems using lead sheet laminated gypsum boards, which also signify a considerable weight reduction in terms of earthquake security.

Drywall radiation protection systems are used in the area of low power X-ray diagnostics and X-ray therapy. The radiation protection is ensured in the form of shielded room-enclosing components with specific lead equivalent values of the materials being used. The lead equivalent value as a reference value specifies the relationship between the shielding effect of the material with the equivalent lead sheet thickness. For

Knauf radiation protection systems, the required lead equivalent value corresponds to the thickness of the lead sheet, e. g. lead equivalent value 2 = 2 mm lead sheet.
The basics of all constructional radiation protection measures are compiled in the radiation protection plan that must be drawn up by the manufacturer of the X-ray equipment. The specification of lead equivalent values of various building materials is listed, for example, in DIN 6812, Table 16. When executing the radiation protection constructions, ensure that the protection is seamless.
Drywall radiation protection systems are available as ceiling linings or subceilings as well as wall furrings and metal stud partitions, and offer the advantages explained in the previous chapters with regard to earthquake resistance - particularly for sensitive buildings like hospitals.
However, you must take the increased dead weight due to of the lead sheet lamination and, therewith reduced permissible wall heights or increased ceiling loads into account. Appendix C2 includes tables with permissible wall heights and the ceiling loads that must be taken into consideration in dependence on the earthquake load and thickness of the lead sheet lamination.

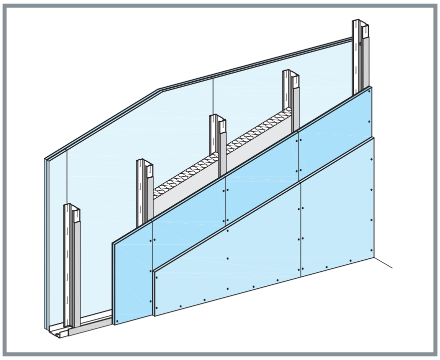


Figure 4.56 Radiation protection wall

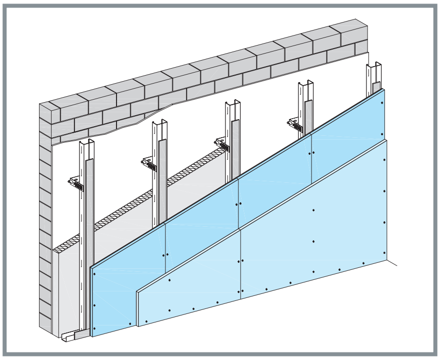


Figure 4.57 Radiation protection furring

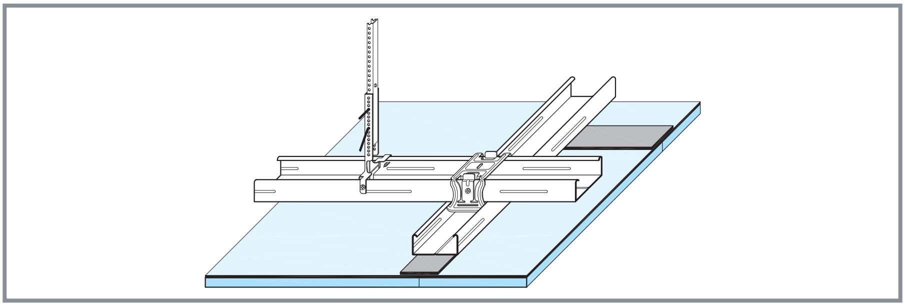


Figure 4.58 Radiation protection ceiling

More information /4.23/ (refer to the bibliography)

4.3 Components of Drywall Systems

As the constructions have been explained in the previous sections, in this chapter the individual components of the drywall systems and their application areas are described, with which the required system properties can be ensured and the efficiency of the construction method can be used to the full.

As only the overall construction and not the individual components can be earthquake resistant, it would seem the topic of earthquake resistance takes more of a back seat in the following pages.

However, this chapter is essential as you use this construction method to improve earthquake resistance and, in doing so, comply with all the other requirements on the component.

Drywall systems are relatively complex composite units, some of which act very sensitively to changes in the properties of their components. Hence, it is therefore necessary to approach the individual components and their application very specifically.

4.3.1 Boards

The key components of drywall systems are the boards, which as cladding or lining on walls, ceilings or façades or as a load bearing layer for floor structures, take on most of the task of ensuring stability, fire protection and sound insulation. Depending on the requirements, different types of boards made of various materials are available. These are described in the following with regard to their material structure, production and properties.

Table 4.7 Overview of boards for drywall constructions

Boards for drywall constructions	Regulated, for example, in
a) Gypsum boards with paper board liner	EN 520, DIN 18180
b) Gypsum boards from reprocessing (Perforated boards, panels, boards with thin laminates)	EN 14190, DIN 18180
c) Composite panels	EN 13950, DIN 18184
d) Gypsum boards with fleece reinforcement	EN 15283-1
e) Gypsum fiber boards	EN 15283-2
f) Cement wallboards	Constructional supervisory permissions

4.3.1.1 Gypsum Boards **EN 520 and DIN 18180**

Gypsum boards consist of a gypsum core that is lined on the front and back of the board and on the longitudinal edges of the boards in a special paper. Depending on the required properties, the gypsum core can have different compositions. It is differentiated between gypsum boards that are used after the production without any factory reprocessing and those that have been reprocessed.
Gypsum boards that have not been reprocessed have longitudinal edges lined with paper, while the production of reprocessed boards like factory board cuts, molded parts, apertura boards, etc., generally do not have edges lined with paper.
Gypsum boards are produced in production lines and have different load properties in the longitudinal and cross-wise directions. Therefore, it is important to ensure the required load bearing properties by considering the specifications of the manufacturer or those of standards and design guidelines for the laying directions of the boards.

Table 4.8 Mechanical properties of gypsum boards acc. to DIN 1052 and EN 520

Action	**Parallel to direction of production line**			**Cross-wise to direction of production line**		
Nominal thickness t in mm	12.5	15.0	t in mm	12.5	15.0	t in mm
Ultimate flexural load in N						
■ Board types A, D, E, F, H, I or combinations	≥ 550	≥ 650	≥ 43 $\cdot t$	≥ 210	≥ 250	≥ 16,8 $\cdot t$
■ Board type R	≥ 870	≥ 725	≥ 58 $\cdot t$	≥ 300	≥ 360	≥ 24 $\cdot t$
Calculation values for the characteristic strength and rigidity values for types A and F						
Board action (transverse to board)						
■ Bending strength $f_{m,k}$ in N/mm²	6.5	5.4	3.6	2.0	1.7	1.1
■ Compressive strength $f_{c,k}$ in N/mm²	3.5 (Type F: 5.5)					
■ E modulus E_{mean} in N/mm²	≥ 2800			≥ 2200		
Diaphragm action						
■ Bending strength $f_{m,k}$ in N/mm²	4	3.8	3.6	2.0	1.7	1.1
■ Compressive strength $f_{c,k}$ in N/mm²	3.5 (5.5)					
■ E modulus E_{mean} in N/mm²	≥ 1200			≥ 1000		
■ Shear modulus G_{mean} in N/mm²	≥ 1200			≥ 1000		

Gypsum boards have different thicknesses, the most common is 12.5 mm. However, 15, 18, 20, 25, 30 mm thickness are also standard. For special applications there are boards with 6.5 mm (mold boards for low bending radii), 9.5 mm or 10 mm (Thermoboards) thickness. The thickness influences the structural stability, the sound insulation and the fire protection. For these areas, a greater board thickness signifies an improvement. With regard to earthquake resistance, however, the greater weight of thicker boards must be considered, whereby even thick boards still have a great weight advantage compared with solid construction components.

Table 4.9 shows the different board types as they are classified in the German standard DIN 18180 “Gypsum boards – types and requirements“ in accordance with their primary properties and are used in many countries.

Table 4.9 Types of gypsum boards in accordance with DIN 18180

Board type DIN 18180	Application area / Properties
GKB Wallboard	Construction boards for cladding non-load bearing drywall constructions and structural wood frame panels, as wall dry lining and to produce gypsum composite boards. Demands on fire protection are only fulfilled with limitations.
GKBI Impregnated wallboard	The application areas of the impregnated wallboards are analogously to the wallboard, however, these boards can be used in damp areas as they have a delayed absorption of water. Appendix B5 contains detailed information on the use of gypsum boards in damp areas. These gypsum boards possess a specially impregnated gypsum core and usually a green-coloured paper.
GKF Fire-resistant board	Fire-resistant boards are used in the same way as wallboards as cladding for drywall constructions, but requirements are placed on the fire resistance duration. The application should be done, e. g. in accordance with DIN 4102-4 or respective test certificates of a testing institue which is authorized by the constructional supervisory board and in accordance with the manufacturer’s specifications. The gypsum core of these boards is additionally reinforced and strengthened by the admixture of glass fibers so as to improve structural cohesion under fire load.
GKFI Impregnated fire-resistant board	The application areas of impregnated fire-resistant boards are analogous to that of fire-resistant boards, however, these boards can be used where a delayed absorption of water is as required in damp areas. Appendix B5 includes detailed information on using gypsum boards in damp areas. These gypsum boards possess a specially impregnated gypsum core and usually a green-coloured paper.
GKP Plaster baseboards	Plaster baseboards are predominantly used as plaster bases on substructures.

With the introduction of the European product standard for gypsum boards, EN 520, European provisions and designations were additionally introduced in November 2004.

Table 4.10 Types of gypsum boards in accordance with EN 520

Board type EN 520	Application area / Properties
A Gypsum board	Gypsum boards for cladding of non-load bearing drywall constructions and structural wood frame panels or as wall dry lining, with a face to which suitable gypsum plasters or decoration coats may be applied. Requirements on fire protection can be fulfilled only with restrictions.
H Gypsum board with reduced water absorption rate	Types of boards which have additives to reduce the water absorption rate. They may be suitable for special applications in which reduced water absorption properties are required to improve the performance of the board. For the purposes of identification, these boards are designated Type H1, H2 and H3, with different water absorption performance. These boards can be used in damp areas. Appendix B5 contains detailed information on the use of gypsum boards in damp areas.
E Gypsum sheathing board	Boards specially manufactured to be used as sheathing boards in external walls. They are not intended to receive decoration. They are not designed to be permanently exposed to external weather conditions. This type of wallboard has reduced water absorption rate. They shall have a minimum water vapor permeability.
F Gypsum board with improved core adhesion at high temperature	Like type A, however these boards have mineral fibers and/or other additives in the gypsum core to improve core cohesion at high temperatures (fire).
D Gypsum board with controlled density	Like type A, however these boards have a controlled density. This enables improved performance in certain applications to be obtained.
R Gypsum board with enhanced strength	These boards for special applications where higher strength is required have both increased longitudinal and transverse breaking loads. They have a face to which suitable gypsum plasters or decoration may be applied.
I Gypsum board with enhanced surface hardness	Like Type A, however these boards are used for applications where higher surface hardness is required.
P Plaster baseboard	Gypsum boards which have a face intended to receive gypsum plaster.

With the exception of the Types A and P, whose properties cannot be combined with other types, gypsum boards can comply with several types according to EN 520. The designation of the type of board then includes all the type designations in combination. Among other things, the application of the CE certificate (CE stands for Communauté Européennes) is also new. The basis of this new development is the resolution passed by the Council of Ministers of the EU of May 1985, to harmonize technical rules and standards for building products, with the aim of minimizing trade barriers. Table 4.10 lists the board types in accordance with EN 520 "Gypsum boards – definitions, requirements and test methods", the allocation to the board types in accordance with DIN 18180 is shown in Table 4.11.

Table 4.11 Types of gypsum board in accordance with DIN 18180, allocation to board types in accordance with EN 520

Board type DIN 18180	Board type EN 520
GKB (wallboard)	Type **A, D, R, I**
GKBI (impregnated wallboard)	Type **H2, DH2, H2R, H2I**
GKF (fire-resistant board)	Type **DF, DFR, DFI, DFIR**
GKFI (impregnated fire-resistant board)	Type **DFH2, DF, H2R, DFIH2, DFH2IR**
GKP (plaster baseboard)	Type **P**

The boards that are most commonly used are wallboards (GKB or GKBI as defined by DIN 18180) and fire-resistant boards (GKF or GKFI as defined by DIN 18180). These boards are at least sufficient to fulfill the minimum requirements.

However, manufacturers of gypsum boards offer a series of high-quality board types that provide additional properties, and that are particularly significant for weight optimization with regard to earthquake resistance.

The following gives some examples of such high-quality boards:

- Knauf Piano Sound Shields
  Knauf Piano Sound Shields are produced both as wallboards (GKB or D) and also as fire-resistant boards (GKF or DF and GKFI or DFH2) and are labeled accordingly. With their flexible nature they have a positive effect on the sound-insulation properties of drywall constructions.
- Knauf Diamant Hard Gypsum Boards
  Knauf Diamant is the fire-resistant board for utmost demands with a large band width (GKFI or DFH2I / DFH2IR). It is the ideal board for impact-resistant, robust cladding in public buildings like schools or hospitals, where robustness, longevity and quality are particularly important.

- LaVita Shielding Boards
  - Shielding low-frequency electrical alternating fields:
    LaVita Shielding Boards considerably reduce the radiation from electrical cables, which generate electrical alternating fields on account of the voltages, e. g. standard voltages at home 230 V and 400 V (three-phase current), in open-wire lines 20 kV to 380 kV.
  - Shielding high-frequency electromagnetic waves:
    LaVita Shielding Boards as a cladding of external walls bring about a very considerable reduction in the waves from outside, like mobile radio equipment, radars and directional radio and radio direction finding, CB radio, satellites and television transmitters.
    As a wall cladding inside a building, these shielding boards reduce the waves from televisions, monitors and transmitter units, wireless telephones in order to protect, for example, children's rooms or bedrooms or for use in the external walls of terraced houses.

  The further properties of LaVita Shielding Boards correspond to DIN 18180 fire-resistant boards GKF or according to EN 520 for board type DF.
- Thermoboards
  Thermoboards are gypsum boards that are 10 mm thick and have improved thermal conductivity for use on cooling and heating ceiling systems.

Gypsum boards can have different edge designs, depending on the thickness, application area, joint filling procedure and in dependence on any possible reprocessing. In doing so, front edges (edges at right angles to the production direction) are generally formed as cutting edges, the boards are cut-to-length from the production line and in some cases they are also beveled.

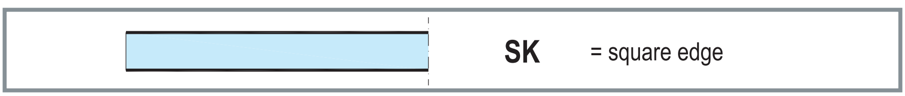


Figure 4.60 Cut edge

Cutting edges, also those that are made up from cuttings on the construction site, should be beveled so as to better accommodate the filling compound.

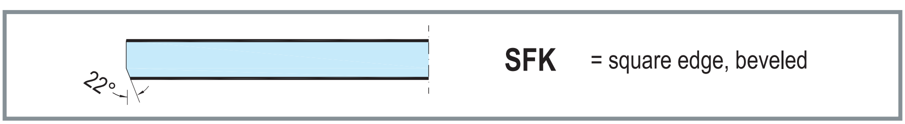


Figure 4.61 Beveled cut edge

The longitudinal edges are usually sheathed in paper and can have the following shapes:

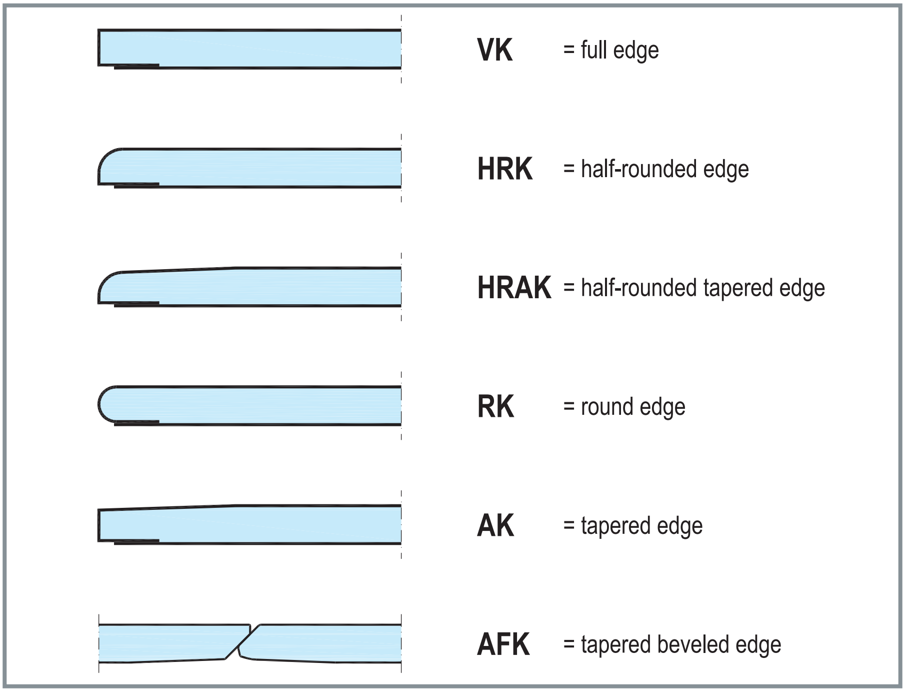


Figure 4.62 Longitudinal edge types

4.3.1.2 Gypsum Boards From Reprocessing EN 14190

Gypsum boards that are reprocessed at the factory after production so as to produce additional functions and properties.

- Special edge designs

  By means of a special procedure, the front edges can also be produced as tapered edges (AK), named 4 AK boards, which are particularly suitable for producing high-quality ceiling surfaces. For more information refer to Chapter 4.3.4 Joint filling.

  Customized cut boards or units have cut edges on all edges, e. g. apertura boards with continuous perforation or tiles for dropped ceilings.
  If the joints remain unfilled, e. g. in case of module ceilings, all edges are beveled.

Figure 4.63 Beveled edge

■ Apertura boards with various perforation designs and edge types
(Design, sound absorption)

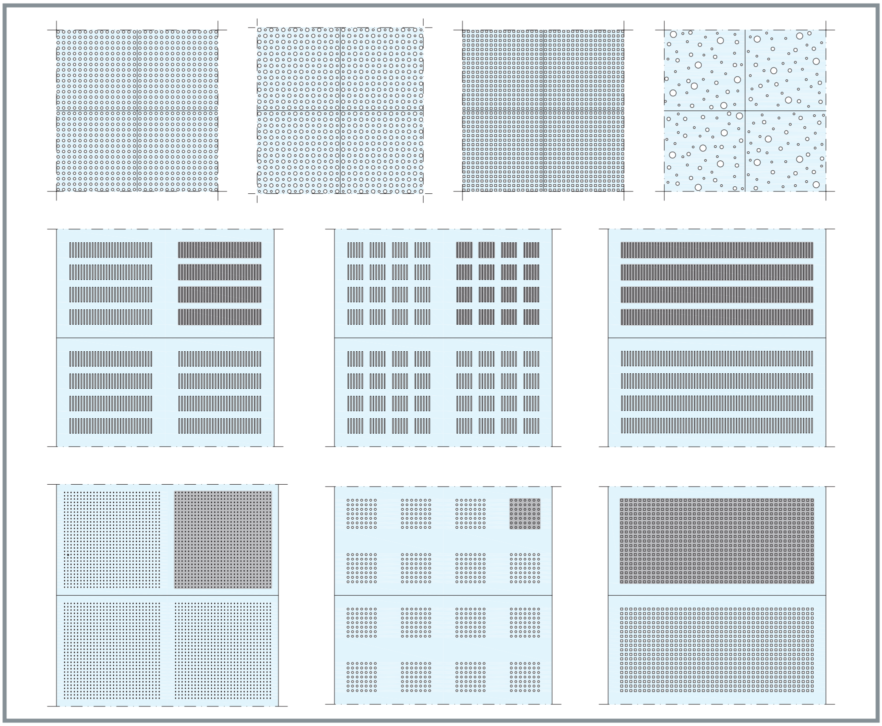


Figure 4.64 Perforation designs of gypsum boards: continuous perforation, block slots, bloc perforation

■ Molded or mitered units (design)

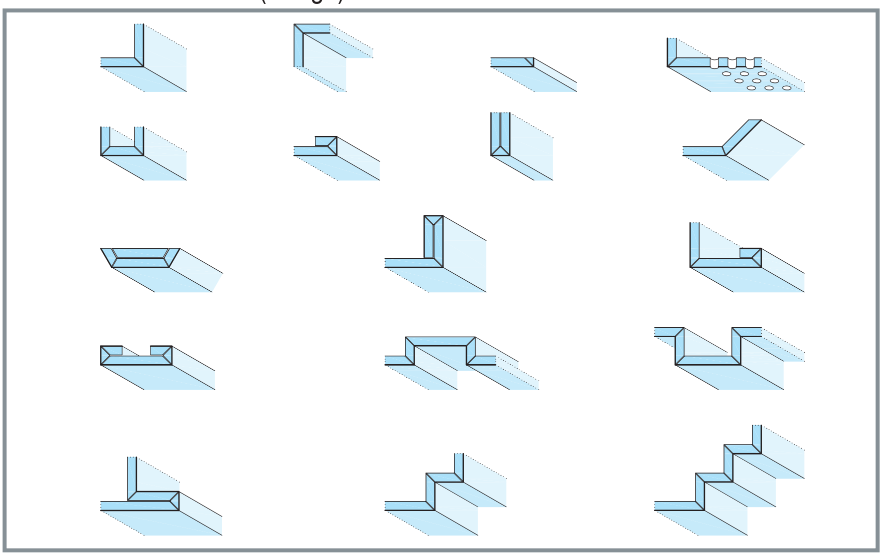


Figure 4.65 Mitered units (prefabricated)

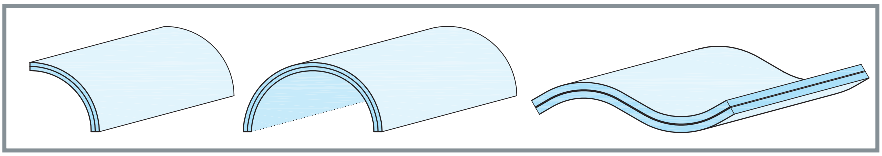


Figure 4.66 Molded units (prefabricated)

- Gypsum boards with thin laminates (e. g. lead sheet, cellulose or aluminum laminating)

Gypsum boards that are reprocessed after production, like, for instance, apertura boards or gypsum boards for dropped ceilings, do not usually have paper edges and are, e. g. regulated in EN 14190.

4.3.1.3 Composite Boards EN 13950 and DIN 18184

These boards have an insulating layer laminated to the rear of the board and this layer is made up of mineral wool, EPS, XPS or PUR.
The main application areas for these composite boards are dry linings, i.e. the boards are glued to the existing wall by means of adhesive mortar. In doing so, mineral wool can ensure sound and thermal insulation; EPS-, XPS- and PUR insulating layers serve exclusively for thermal insulation.
As dry lining is attached to the basic wall in a non-mechanical manner, its suitability for earthquake resistance is restricted to areas with low seismic activity.

4.3.1.4 Gypsum Boards With Fleece Reinforcement EN 15283-1

Gypsum boards can also be sheathed in fiber glass fleecing instead of paper. These boards are particularly suitable for the high demands on fire protection and on fire behavior. These boards rank among the building material Class A1 in accordance with DIN 4102.
In combination with a fiber-reinforced, highly refined special gypsum core, for example, Fireboard is suitable for utmost fire stresses.
Special fire protection systems, ventilation and cable ducts as well as fire protection cladding are the main application areas for these boards.
These boards are available in thickness of 6.5 / 8 / 12.5 /15 / 20 / 25 and 30 mm.

4.3.1.5 Gypsum Fiber Boards EN 15283-1

Gypsum fiber boards consist of gypsum and cellulose fibers which hold together the gypsum core as a “reinforcement”. Gypsum fiber boards have no surface sheathing, they are produced as raw material boards and then processed depending on the application purpose, e. g. cut to size or sanded.

By varying the densities you can achieve different load bearing capacities.
On account of their good load bearing capacity, gypsum fiber boards are preferred as a bracing cladding of structural wood frame panels and for floor constructions.
The weight is greater compared to gypsum boards, so, in terms of earthquake resistance, the greater weight has to be weighed up against the greater load bearing capacity, especially if using them as a wall cladding.
The edge design of gypsum fiber boards are cutting edges. Depending on the filling or joining technique, the board formats are designed for different joints widths. Gypsum fiber boards for prefabricated screeds have an edge formation either as a milled tier, or cut level for a staggered bonding of two boards.

4.3.1.6 Cement Wallboards **Use by permission**

Cement wallboards for use in drywall systems, like, for example, AQUAPANEL®, consist of a core made of Portland cement and additives, which is reinforced on both sides with a glass gauze fabric. The front edges are cut and the longitudinal edges reinforced.
Because of their moisture resistance, these boards can be used without restriction in damp or wet areas on ceilings, walls or floors or in the external area as façade cladding. The fulfilment of fire protection and sound insulation requirements is on a par with that of gypsum boards.
Depending on the application area, three different types of cement wallboards are available:

- AQUAPANEL® Cement Board Indoor with a board thickness of 12.5 mm for the internal finishing, as cladding for ceiling or wall systems
- AQUAPANEL® Cement Board Outdoor with a board thickness of 12.5 mm for external applications, as a façade cladding or for ceilings in the external area
- AQUAPANEL® Cement Board Floor with a board thickness of 22 mm or as a composite unit with mineral wool 33 mm for interior finishing, as cement-based prefabricated screed

4.3.2 Profiles **e. g. EN 14195 and DIN 18182-1**

Though the boards are the key components of drywall systems, yet they usually require a substructure.
These substructures consist of thin-walled, rolled sheet metal profiles with a thickness of around 0.6 to 2 mm. In the area of load bearing systems, profile thickness of 3 mm are also used, the standard plate thickness for non-load bearing structures is 0.6 mm. Lower plate thicknesses could cause problems in the screw connections and hence bring about a reduction in the load bearing capacity.

Depending on the application area, the profiles have different cross-sectional shapes.

4.3.2.1 Wall Profiles

Profiles for walls are divided into studs and perimeter runners. Wall profiles are offered in various web widths from 48 – 150 mm. The web width has an influence both on the sound insulation property of the wall and on the load bearing capacity and hence the permissible wall height. However, the broader profile has less influence on the load bearing capacity than the increased distance of the two-sided wall cladding.

Permissible wall heights for different profiles **refer to Appendix C**

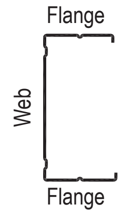


Figure 4.67 Cross-section of profile

The perimeter runners form the connection between the flanking components and the wall, and ensure, together with their anchorage, the load transfer from the drywall partition to the adjacent wall, ceiling and the floor.
Perimeter runners for ceiling and floor connections are UW profiles. They are anchored to ceilings or floors at the required spacing by means of suitable dowels. This anchoring spacing usually amounts to 1 m, in the case of greater wall loads, like for example, from earthquakes or in the case of high fire protection requirements, you can also use smaller spacings, refer to Chapter 4.2.2 and Appendix C2. Chapter 4.3.3.2 explains which dowels are suitable for the different fixing surfaces.
The flange width of the perimeter runners usually amounts to between 30 and 40 mm. Greater flange widths may be required in the case of earthquake loads and the wall connections that need to be used for this purpose.

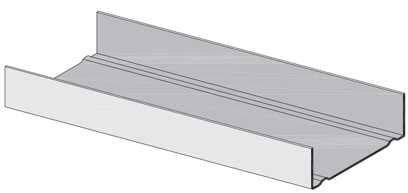


Figure 4.68 UW profile

Edge connections on flanking walls are implemented using studs, which are anchored via the web to the adjacent wall.

Studs are usually C or CW profiles. These are aligned vertically at equal spacings complying with the grid dimensions of the boards and in dependence on the required load bearing capacity and installed into the perimeter runners. With a board width of 1.20 m, the axial stud spacing could amount, for example, to 600, 400 or 300 mm.
The boards are screw attached into the studs.
The flange width of the studs usually amounts to between 34 and 50 mm, whereby with regard to the earthquake load, greater flange widths (50 mm) may be better suited, as greater flange widths enable the boards to be screw attached at greater edge distances, hence reducing the danger of the gypsum boards failing on the board edges. To carry out cabling in the cavity of the partition in the horizontal direction, studs have hole perforations or H perforations in which, if required, you can create a hole by bending upwards.

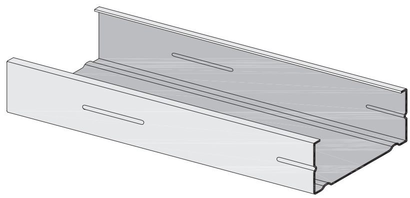


Figure 4.69 Stud profile CW

Studs have a great influence on the sound insulation, which is why there are special profiles for greater sound insulation requirements, like for example, MW studs, which, on account of their special shape, extend the transmission path of the sound and absorb the sound energy.

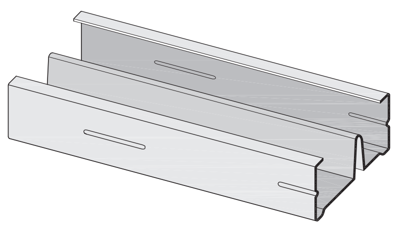


Figure 4.70 Stud profile MW

4.3.2.2 Ceiling Channels

Ceiling channels (profiles), which are used as furring or carrying channels of ceiling linings or subceilings, differ to wall profiles as they are subjected to different loads. While wall profiles are mainly subjected to loads in the direction of the web level, ceiling profiles are subjected to bending acting in the direction of the flanges. Furthermore, ceiling profiles must be developed in such a way that a secure connection with hangers can be guaranteed.

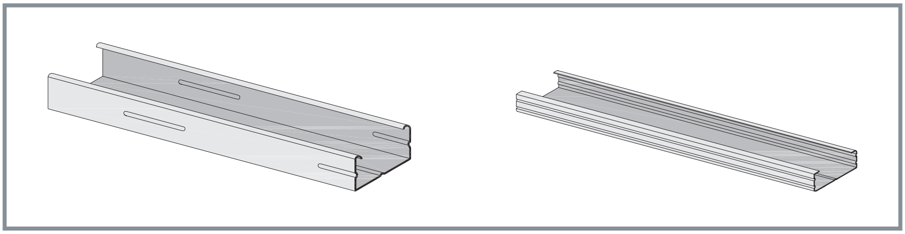


Figure 4.71 Ceiling channels, e. g. *CD 60x27 and F47*

If required, the edge connection for suspended ceilings is carried out by using UD runnerss, which in their web height are precisely adapted to the flange width of the ceiling profiles.

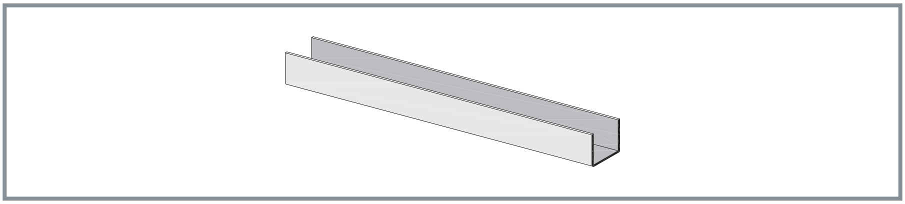


Figure 4.72 UD runner for edge connections in subceilings

Ceiling profiles can be pre-molded at the factory so as to realize round ceiling constructions like domes or convex or concave arched ceilings or arches.

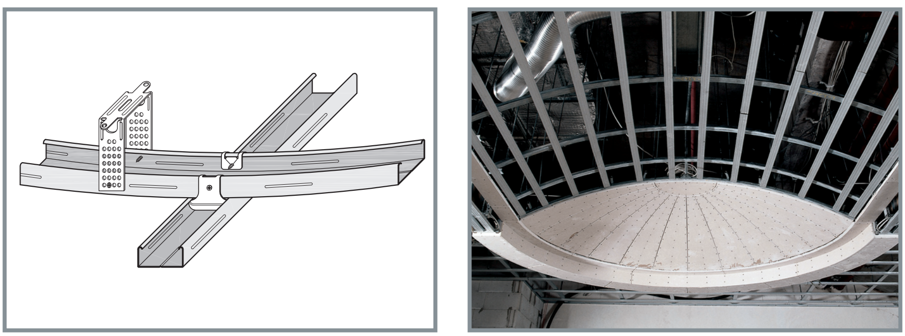


Figure 4.73 Vaulted ceiling grids

For ceiling linings, where the substructure is directly anchored to the fixing base, hat-shaped channels or resilient channels are used for lower construction heights.

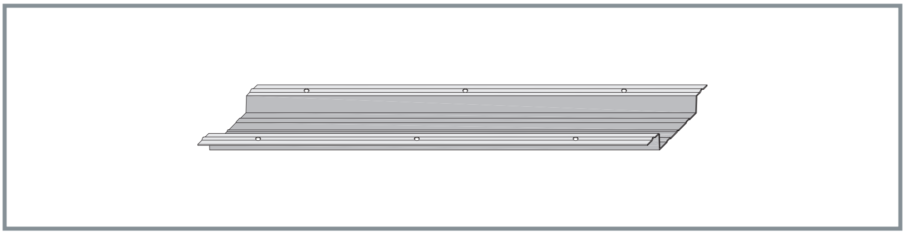


Figure 4.74 Hat-shaped channel

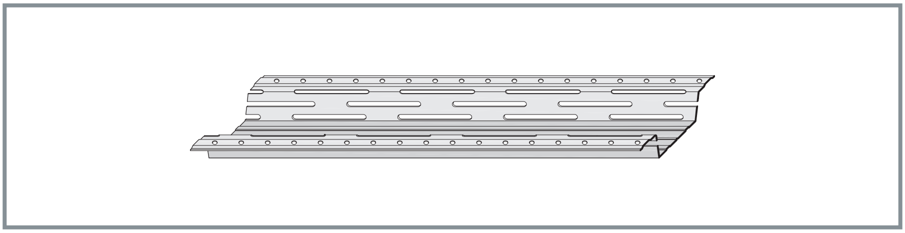


Figure 4.75 Resilient channel

In the case of free-spanning ceilings, profiles are used that are usually used for metal stud partitions (CW, UW), as their construction and also the direction of load is similar to that of the walls. To increase the load bearing capacity, two profiles are connected to each other on the web side to form double profiles.

Figure 4.76 Substructure for a free-spanning ceiling

4.3.2.3 Special Profiles

Special profiles are available for special applications, e. g. in light-gauge steel construction systems or for special structures.
UA profiles are profiles with a U-shaped cross-section and a plate thickness of 2 mm.
They are used in areas where high load bearing capacities are required, e. g. in room-in-room systems or to strengthen door and wall openings.
Angle profiles are used for connections, e. g. ceiling connections on walls, reduced wall connections, wall connections for bullet-resistant walls, etc.

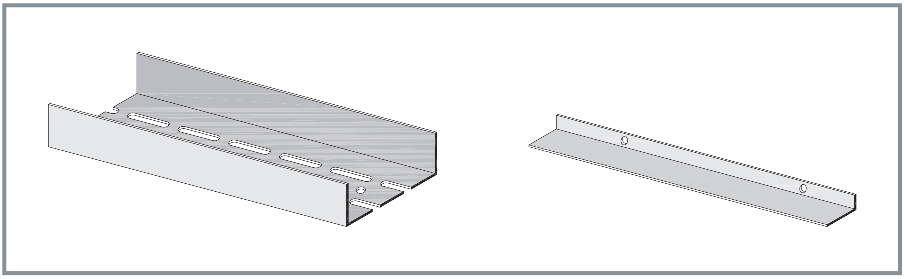


Figure 4.77 UA profile and angle profile

4.3.2.4 Corrosion Protection

To ensure the necessary corrosion protection, the profiles are galvanized at the factory. However, this corrosion protection is only sufficient for interior areas, including domestic bathrooms and kitchens. In other areas, e. g. where there is outdoor air or in wet areas, you must provide additional corrosion protection measures.

4.3.3 Anchorings, Fasteners, Connectors and Hangers

The following definitions and classifications for anchorings, fasteners and connectors offer a practical subdivision of these components with regard to their function:

Anchorings
Component that mechanically anchors the substructure to the base.

Hangers
Component for suspended ceilings, between the anchoring and the substructure with variable lengths to achieve the desired suspension height.

Connectors
Metal component that connects the parts of the substructure with one another.

Fasteners
Metal component that mechanically fixes the cladding to the substructure.

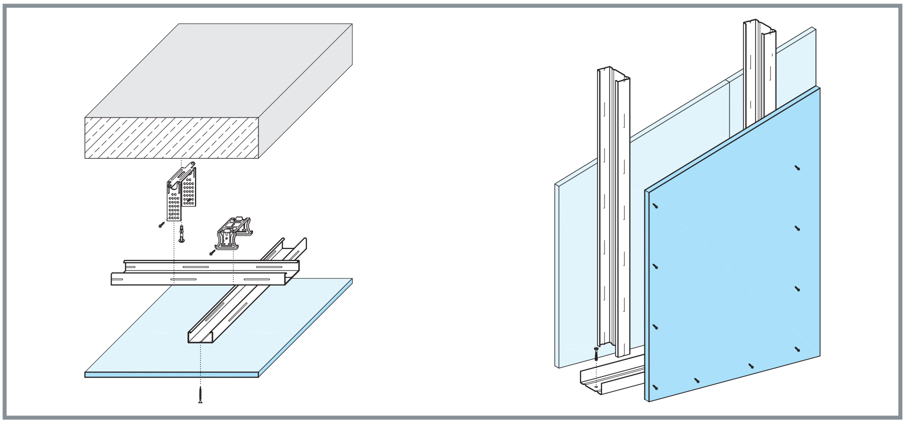


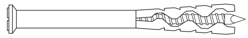
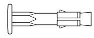
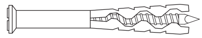
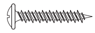
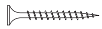
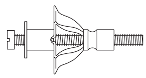
Figure 4.78 Anchorings, fasteners and connectors for subceilings and walls

As the anchorings, fasteners and connectors connect the individual components of the drywall systems with each other as well as the entire construction component with flanking components, they are also of great importance for ensuring the required properties. This particularly applies to ensuring the load bearing capacity with regard to earthquake resistance. If anchorings, fasteners and connectors are used that do not possess the properties that are assumed in the dimensioning, then this could cause premature failure. Therefore, the manufacturer's instructions regarding fastening spacings, penetration depths and the anchorings, fasteners and connectors that are to be used with regard to make and type, must be complied with.

4.3.3.1 Anchorings

The fixing of the drywall structures to flanking components is carried out by means of anchorings that are suitable for the respective base.

Table 4.12 Anchorings for drywall partitions

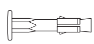
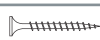
Base	Anchoring material	
Reinforced concrete	Nailable plug	
	Ceiling steel dowel	
Masonry	Nailable plug	
Wood or metal stud partitions	Multi purpose screw (screwed into stud)	
	Drywall screw (screwed into stud)	
	Cavity dowel	

For external walls the anchoring is carried out exclusively by means of anchorings made of steel, like, for example, steel dowels. Synthetic dowels are not permitted.

As the anchoring of the hangers for ceiling linings or subceilings on the basic ceiling are key components they must be authorized for the respective application.
This means that in areas that are endangered from earthquakes and an anchoring on reinforced concrete basic ceilings, an authorization form a construction supervisory board for use in the cracked concrete tension zone and for quasi-static or dynamic loads must be in place.
Furthermore, it must be noted that the load bearing capacity of the dowel can only be guaranteed if a minimum concrete quality, in accordance with the pertinent standards including, if necessary, additional conditions specified in the authorization, can be proven.

In dependence on the fixing base, the following anchoring can be used:

Table 4.13 Anchoring materials for hangers of drywall ceilings

Base	Anchoring material	
Reinforced concrete	Ceiling steel dowel	
Wood joist ceiling	Flat headed screw	
	Drywall screw	
Corrugated metal sheets	Multi purpose screw	

4.3.3.2 Hangers and Clips for Ceiling Linings and Subceilings

Ceiling linings

The simplest variant of ceiling lining is the direct fastening of the cladding without substructure. Chapter 4.3.3.3 has already covered this application.

The substructure of ceiling linings is either screw attached directly to the basic ceiling or anchored to the basic ceiling by means of clips.

In the case of ceiling linings with wooden substructures, the lathing is screw attached directly to the wooden beams of the basic ceiling by means of special trumpet head screws with pin points (TN). In doing so, it is possible to compensate for the tolerances of uneven wooden beams only by means of spacers and wedges.

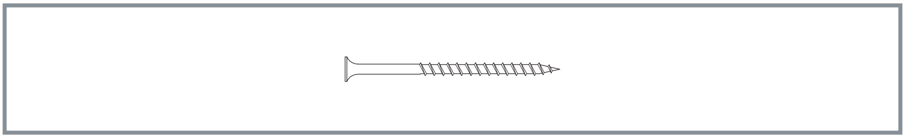


Figure 4.79 Drywall screw TN 4.5 x 70

In the case of considerable unevenness, the use of a direct hanger for wooden battens is recommended - refer to the chapter on subceilings.

Metal substructures made of resilient channels or hat-shaped channels are anchored directly to the basic ceiling by means of the appropriate screws or dowels. It is possible to have only a very slight tolerance compensation of a few millimeters for the unevenness of the basic ceiling.

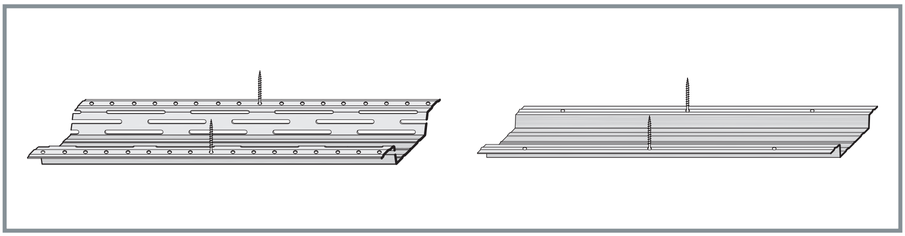


Figure 4.80 Resilient channel and hat-shaped channel with screws

The clips for fixing metal substructures of ceiling linings made of CD profiles are anchored on the basic ceiling by means of screws or dowels, the profiles are either enclosed by the clip (Clip Fastener) or are pressed into the fixed clip from below (Direct Bracket). With the Clip Fastener you can compensate for a slight amount of unevenness of the basic ceiling.

It is not possible to compensate by means of the Direct Bracket. In the case of considerable unevenness, the use of a Direct hangers is recommended (refer to subceilings).

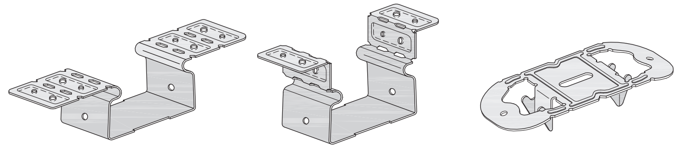


Figure 4.81 Clip Fastener with tolerance compensation and Direct Bracket

<u>Subceilings</u>

With the exception of free-spanning ceilings, subceilings with hangers are fixed to the basic ceiling. The most important property of a hanger is its load bearing capacity, which determines the maximum spacings of the hangers as well as the capability for the respective ceiling weight. The load bearing capacity of the hanger must be determined by the manufacturer in accordance with pertinent standards, e. g. in accordance with EN 13964 or DIN 18168-2 by means of test series and respective statistical evaluations. When using hangers, ensure the appropriate proof has been submitted. Basically, you should use only those hangers that are recommended by the manufacturer of the drywall systems.

The following hangers can be used in dependence on the application area:

- Nonius suspension system

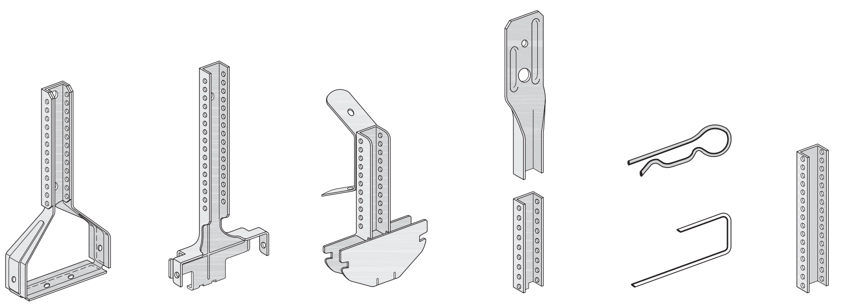


Figure 4.82 Components of the Nonius suspension system

A Nonius suspension consists of the Nonius Top, which is anchored to the basic ceiling, the hanger, the Splint or the Bop Pin, which connect the top and the bottom,

and, if required for greater bracket heights, a Nonius Connector.
Nonius suspension possess a proven load bearing capacity of 0.4 kN per hanger.
In areas endangered by earthquakes, the Nonius Stirrup is the ideal hanger, as, thanks to the complete coverage of the ceiling channels, there is a very secure connection between the hanger and the ceiling channel. For connecting with the Nonius Top or the Nonius Connector, two Nonius Bop Pins or a Nonius Splint should be used in earthquake areas. The latter must be bent back to ensure they do not slide out.

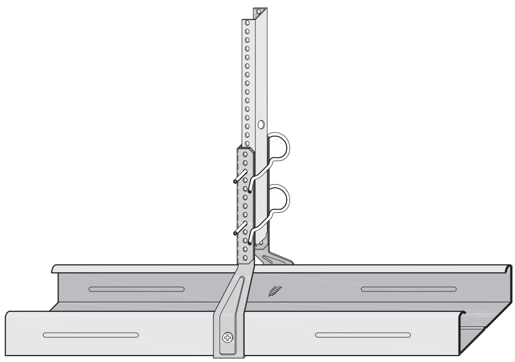


Figure 4.83 Nonius hanger with two Nonius Bop Pins

- Direct Hanger
  A variant of hanger that is simple to fit is the Direct Hanger.
  If fitted properly it offers a secure connection to the basic ceiling. Depending on the fixing base material the anchoring can be carried out on the basic ceiling by means of a screw or an appropriate dowel that is fitted to the center of the Direct Hanger, or by using two screws that are screwed into the wings.
  The ceiling channels or battens are screw attached at the sides into both flanges with the Direct Hanger by means of metal screws. The tongues can be bent down or cut off according to the required suspension height.

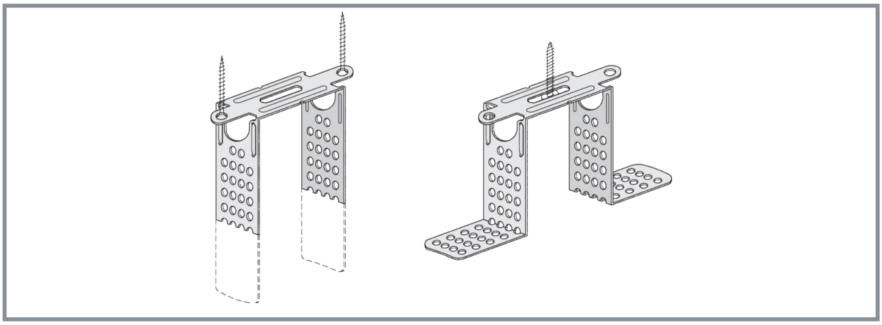
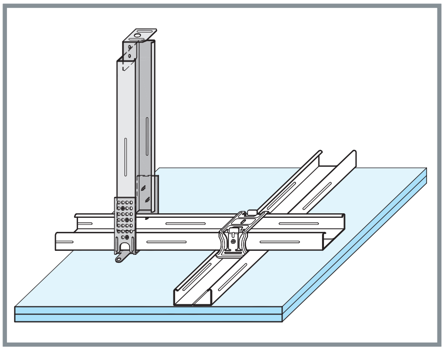


Figure 4.84 Options for anchoring and height adjustment of Direct Hangers

The Direct Hanger also has a load bearing capacity of 0.4 kN per hanger.
These hangers can also be used in earthquake-endangered areas. Thanks to a degree of compressive rigidity, it can resist a certain amount of pressure forces. The disadvantage compared to the Nonius suspension is merely its restricted suspension height.

- Suspensions made of metal profiles



A particular compressive rigidity and also an easy to fit suspension can be created using profile segments that are connected to the ceiling channels by means of Direct Hangers and with special angles.

Figure 4.85 Suspension system D112i

4.3.3.3 Connectors

For connections between the profiles, such as profile extensions, cross points, direction changes, etc., precise molded elements made of metal sheet are available. The main application areas for these connectors are ceiling constructions that have a metal substructure.

- Using the Intersection Connector, the carrying channel and the furring channel of the ceiling constructions are connected as a double grid.

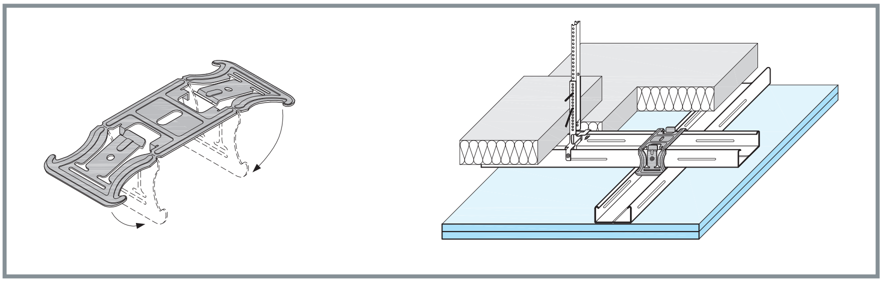


Figure 4.86 Intersection Connector

- Using the Flush Connector, the carrying channel and the furring channel are linked on the same level. In doing so, the carrying channels are continuous.

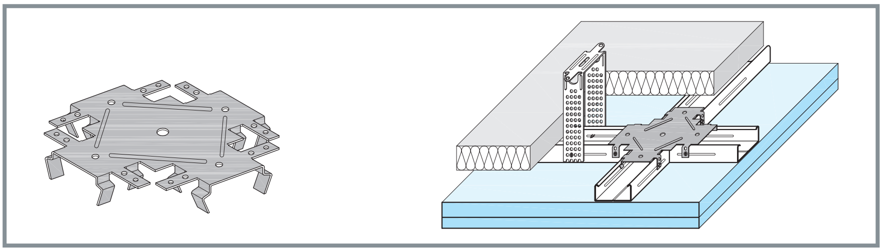


Figure 4.87 Flush Connector

- You can use the Universal Connector to connect the carrying channel and the furring on the same level as well. You can also create T connections of ceiling channels for special constructions.

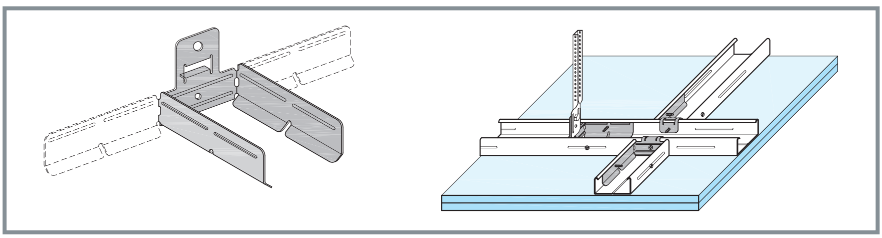


Figure 4.88 Universal Connector

■ Using the Daisy Chain Clip, for example, you can create non-orthogonal crossing points of profiles or ceiling fins.

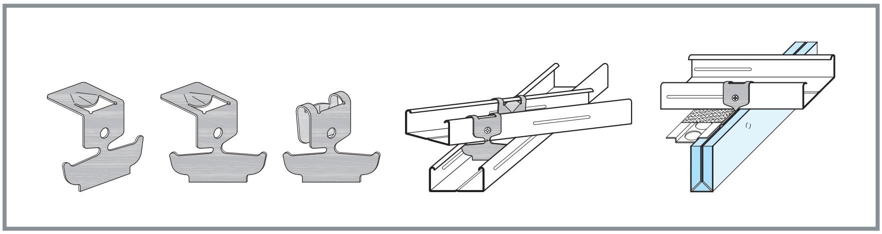


Figure 4.89 Daisy Chain Clip

■ Longitudinal profile joints of ceiling channels are created by using the Multi Connector. In connection with the associated adapters, the substructures of design ceilings can be linked in virtually all conceivable variants.

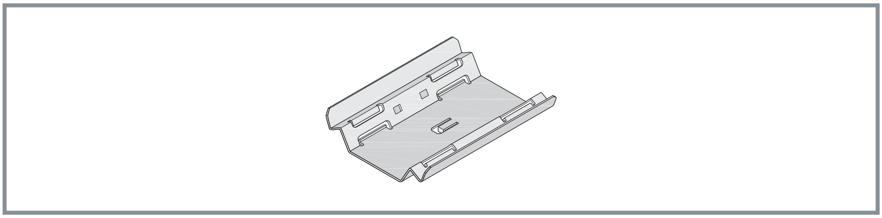


Figure 4.90 Multi Connector

There are three methods of connecting metal profiles to each other: Screws, rivets and crimping.

The screw connection is important, particularly for ceiling constructions in earthquake-endangered areas, but also, e. g. in the case of fire protection requirements.

All the previously mentioned profile connectors and hangers that offer a secure connection for pure dead weights without screwing, should, in the case of requirements on earthquake resistance in areas with design acceleration of more than 0.8 m/s², be screwed with the ceiling profiles. In dependence on the metal thickness, metal screws with a fillister-head and pin points or drill points should be used to this purpose.

Figure 4.91 Metal screws with pin point (LN) and drill point (LB)

Most hangers and connectors already have the holes to this purpose.

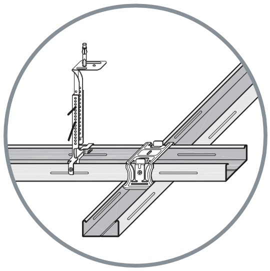


Figure 4.92 Hangers and connectors, screwed into the ceiling substructure

The substructures of free-spanning ceilings and metal stud partitions can also be connected using metal screws.

For constructional connections (not for load transfer), as an alternative to using screws you can also use rivets. The disadvantage however is that this connection is very difficult to reverse and you cannot do this without destroying it.

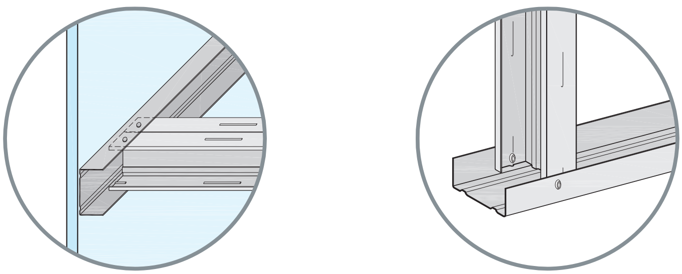
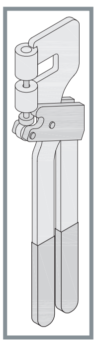


Figure 4.93 Riveted profiles



A third option for a constructional connection between profiles is to crimp. Here, the sheets of the profiles that are lying on top of each other are stamped using stamp pliers, which creates a connection that can be particularly resistant to shear forces.

Figure 4.94 Stamp Pliers

4.3.3.4 Fasteners - Attachment of the Cladding to the Substructure

Depending on the material of the substructure, the cladding can be fastened to the substructure using various fasteners. The minimum edge distances and penetration depths must be complied with. In doing so, on account of the quasi-static equivalent load method during dimensioning, generally speaking, the properties for static loaded

constructions are also used under earthquake load.
However, cladding made of gypsum or gypsum fiber boards used as a bracing cladding in earthquake zones, must comply with greater edge distances, in order to ensure the necessary ductility even after a number of load reversals.

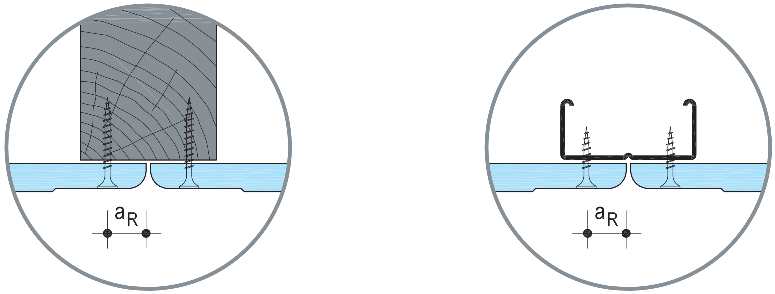


Figure 4.95 Edge distances when fastening the cladding

Table 4.14 Edge distances of fasteners for drywall constructions

Purpose	**Edge distance a_R** **Paper/fleece sheathed edge**	**Cut edge**
Cladding material: gypsum boards		
Ceiling lining / subceiling	≥ 10 mm	≥ 15 mm
Metal stud partitions (non-load bearing)	≥ 10 mm	≥ 15 mm
Wooden stud partitions (non-load bearing)	≥ 10 mm	≥ 15 mm
Structural wood frame wall panels (bracing)	$\geq 7 \cdot d_n$ [1)]	$\geq 7 \cdot d_n$ [1)]
Cladding material: gypsum fiber boards		
Ceiling lining / subceiling	-	≥ 15 mm
Metal stud partitions (non-load bearing)	-	≥ 15 mm
Wooden stud partitions (non-load bearing)	-	≥ 15 mm
Structural wood frame wall panels (bracing)	-	$\geq 7 \cdot d_n$ [1)]
Cladding material: cement wallboards		
Ceiling lining / subceiling	≥ 15 mm	≥ 15 mm
Metal stud partitions (non-load bearing)	≥ 15 mm	≥ 15 mm
Wooden stud partitions (non-load bearing)	≥ 15 mm	≥ 15 mm

[1)] d_n ... Nominal diameter of the fastener

- Screws are used both on metal as well as on wooden substructures.

  Drywalling uses drywall screws, which, on account of their special thread, screw form and screw point, are particularly suitable for attaching the boards to the substructure. Different board materials require different screws.

  Screw attaching boards to metal profiles is a relatively complex process from a technical material point of view, as two materials with very different properties are penetrated very quickly and the result must form a strong bond. Therefore, it is imperative to adhere to the recommendations of the manufacturer.

  In the area of the ceiling linings, it is possible to fasten the cladding without a substructure directly on to level basic ceilings made of wooden beams or steel sheeting (e. g. trapezoid sheet ceilings). Drywall screws are also used to this purpose.

  Table 4.15 shows the different drywall screws and their applications.

  To ensure the load bearing capacity of the connection between the boards and the substructures, minimum penetration depths of the screws into the substructure must be complied with.

  For non-load bearing walls, ceiling linings and subceilings, the required penetration depths of the screws in metal profiles is 10 mm, in wooden substructures the penetration depth must amount to at least five times the nominal diameter of the screw.

  In the case of reinforced structural wood frame panels, the required penetration depth is also five times the rated diameter, however, at least 24 mm.

  The length of the drywall screw is chosen in accordance with the required penetration depth.

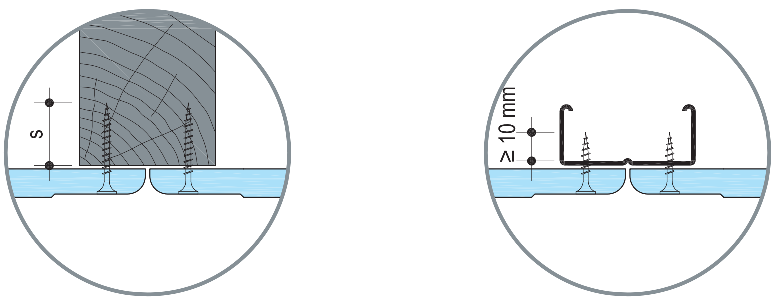
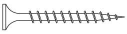
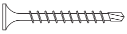
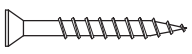
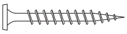
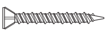
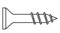
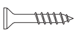
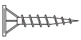
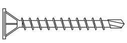
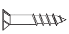


Figure 4.96 Penetration depth of screws in the substructure

Table 4.15 Drywall screws for fastening the cladding

Type of drywall screw	Properties	Use
TN 	Trumpet head with pin point, double thread	Gypsum boards on metal substructure up to 0.7 mm gauge or on wooden substructure
TB 	Trumpet head with drill point, metal screw thread	Gypsum boards on metal substructure as of 0.7 mm to 2.25 mm gauge or directly on trapezoid sheet
SN 	Counter-sunk with pin point, metal screw thread, lower suppression to protect against breaking	Apertura gypsum boards on metal substructure up to 0.7 mm gauge
HGP 	Screw head with surrounding underneath edge, minimum suppression volumes	Hard gypsum boards (Diamant) on metal substructures up to 0.7 mm gauge or on wooden substructures
Vidiwall Screws 	Trumpet head with milling ribs, pin point	Gypsum fiber boards on metal substructure up to 0.9 mm gauge or on wooden substructure
Screws for gypsum fiber prefabricated screed  	Trumpet head	Connection of gypsum fiber prefabricated screed elements in the area of the tier edge joints
AQUAPANEL® Maxi Screw SN (25/39/55) 	Screw head plate with milling ribs; pin point	Cement wallboards on metal substructure up to 0.7 mm gauge or on wooden substructure
AQUAPANEL® Façade Screw SN 40 	Screwhead plate with milling ribs; pin point on stainless steel V2A; HiLo thread	Cement wallboards on wooden substructure
AQUAPANEL® Maxi Screw SB (25/39) 	Screwhead plate with milling ribs; drill point	Cement wallboards on metal substructure as of 0.7 mm to 2.5 mm gauge
AQUAPANEL® Floor Screw 24 (21) 	Half-round peeling head	Connection of cement prefabricated screed units in the area of the tier edge joints

The spacings of the fasteners depend on the application area and the construction. Table 4.16 shows the fastening spacings for the most common application scenarios. In doing so, the fastening spacings for constructions with single-layer cladding or for the top (visible) cladding layer of constructions with multi-layer cladding are given in the middle column, the fastening spacings for concealed cladding layers of multi-layered clad constructions are shown in the right column.

Fastening spacings of concealed cladding layers can be increased for gypsum and gypsum fiber boards, as the external cladding layers are fastened by screwing through the previously fixed layers.

Table 4.16 Fastening spacings for cladding of drywall constructions with screws

Purpose	Maximum fastening spacing	
	Single-layer cladding resp. topmost (visible) layer in case of multi-layer cladding	Lower (concealed) layers in case of multi-layer cladding
Cladding material: gypsum boards		
Ceiling lining / subceiling	170 mm	500 mm (300 mm*)
Metal stud partitions (non-load bearing)	250 mm	750 mm (500 mm**)
Wooden stud partitions (non-load bearing)	250 mm	750 mm (500 mm**)
Structural wood frame panels (bracing)	Depending on load bearing capacity, max. 250 mm	Depending on load bearing capacity, max. 750 mm
Cladding material: gypsum fiber boards		
Ceiling lining / subceiling	170 mm	500 mm (300 mm*)
Metal stud partitions (non-load bearing)	250 mm	750 mm
Wooden stud partitions (non-load bearing)	250 mm	750 mm
Structural wood frame panels (bracing)	Depending on load bearing capacity, max. 250 cm	Depending on load bearing capacity, max. 750 cm
Cladding material: cement wallboards		
Ceiling lining / subceiling	170 mm	170 mm
Metal stud partitions (non-load bearing)	215 mm	215 mm
Wooden stud partitions (non-load bearing)	215 mm	215 mm

* In case of boards with more than 18 mm thickness, on account of the high dead weight, the distances of concealed cladding layers on ceilings is increased only to 300 mm.

** Middle layer in the case of triple-layer cladding.

For bracing structural wood frame wall panels, the edge distance in the wood studs must be restricted in dependence on the fastener's diameter. Because of the greater diameter compared with the nails and staples, screws are not favorable for this application area, as large stud cross-sections could be necessary on account of the required edge distances alone.

- Nails are used to fix gypsum or gypsum fiber boards to wooden substructures. It is standard to drive in the nails using the respective nailing equipment quickly and efficiently. The shaft of the nail can be smooth or ridged for better interlocking with the wood fibers. You should use only ridged nails or those with a resin covered shaft to fix the cladding to ceilings or roof pitches. The nails must be hammered in flush.

  The compliance with minimum penetration depths in the substructure also applies to nails. For non-load bearing walls, ceiling linings and subceilings, the required penetration depth is
  - at least 12 x the nominal diameter of the nail for smooth nails
  - at least 8 x the nominal diameter of the nail for ridged nails

  The length of the nail is chosen in accordance with the required penetration depth.

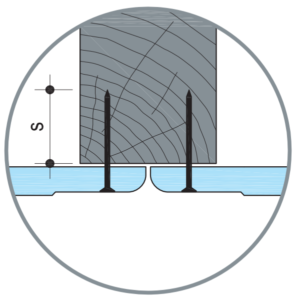


Figure 4.97 Penetration depth of nails in the substructure

Table 4.17 Fastening spacings for cladding of drywall constructions with nails

Purpose	Maximum fastening spacing	
	Single-layer cladding resp. topmost (visible) layer in case of multi-layer cladding	Lower (concealed) layers in case of multi-layer cladding
Cladding material: gypsum boards		
Wooden stud partitions (non-load bearing)	120 mm	360 mm
Structural wood frame panels (bracing)	Depending on load bearing capacity, max. 150 mm	Depending on load bearing capacity, max. 240 mm
Cladding material: gypsum fiber boards		
Wooden stud partitions (non-load bearing)	250 mm	400 mm
Structural wood frame panels (bracing)	Depending on load bearing capacity, max. 150 mm	Depending on load bearing capacity, max. 150 mm

- Staples are used to fasten gypsum boards to wooden substructures or gypsum boards with each other. However, their use with structurally active cladding should be restricted to areas with medium seismic activity.
  During application, ensure a high quality of the connections.
  They must be driven in so that the angle between the paper fibers and the spine of the staple amounts to 45°, and the spine of the staple is flush with the surface. Under no circumstances should the staples jut out.

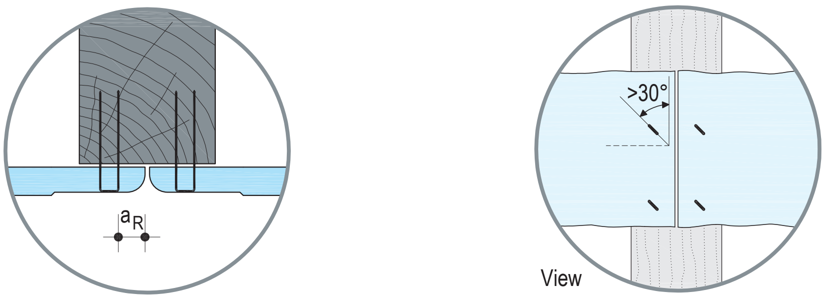


Figure 4.98 Edge distances and angle to the paper fiber

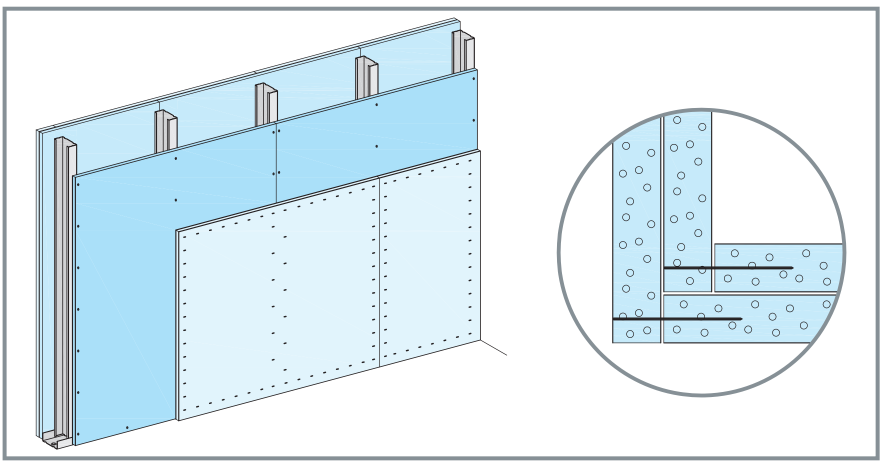


Figure 4.99 Stapled 2nd board layer, corner detail of cable duct, stapled

The minimum penetration depths for screws and nails also applies to staples in the substructure:

- For non-load bearing partitions and ceiling linings or subceilings at least 15 times the nominal diameter of the staple
- For reinforced structural wood frame panels and their structurally active cladding at least 12 times the nominal diameter of the staple

For structurally active (bracing) cladding you must use resine covered staples.

The length of the staple must be chosen in accordance with the required penetration depth.

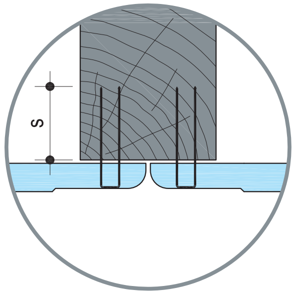


Figure 4.100 Penetration depth of staples in the substructure

Table 4.18 Fastening spacings for cladding of drywall constructions with staples

Purpose	**Maximum fastening spacing**	
	Single-layer cladding resp. topmost (visible) layer in case of multi-layer cladding	**Lower (concealed) layers in case of multi-layer cladding**
Cladding material: gypsum / gypsum fiber boards		
Wooden stud partitions (non-load bearing)	80 mm	240 mm
Structural wood frame panels (bracing)	Depending on load bearing capacity, max. 150 mm	Depending on load bearing capacity, max. 360 mm

4.3.4 Joint Treatment

The filling and finishing of the joints in drywall structures has a multitude of functions:

- Creating a smooth, consistent and even surface
- Flawless tightness for sound insulation requirements
- Tightness for fire protection requirements
- Adhesive connection between the boards to ensure structural stability

Apart from the creation of smooth visible surfaces, all the functions that are to be carried out apply to all cladding layers. This means that, in the case of multi-layer cladding drywall constructionss, the joints of the concealed layers must also be carefully filled with filling compound.
Otherwise you cannot guarantee any requirements on sound insulation, fire protection and structural stability. The properties of drywall systems specified by the manufacturers assume a proper filling of all the clad layers.

It is merely the visual quality that is of no real importance when filling the concealed layers.

Depending on the board material, the edge shape and the desired surface quality, there are various filling techniques that require the use of different materials.

In principle: Filling should only be carried out if no more great changes in length to the boards, e. g. as a result of moisture or temperature changes, are expected.
For filling purposes, the minimum surface and room temperature is 10 °C.
It is recommended to generally fill cut edge joints with paper joint tape, independently of the filling compound being used.

4.3.4.1 Filling and Finishing of Gypsum Boards

The different edge shapes of gypsum boards are illustrated in Chapter 4.3.3.1
The filling procedure that is suitable here shall now be described.
Joint filling can be carried out by hand application or with the aid of a so-called Ames machine, i.e. mechanically.

Figure 4.101 Hand application

Figure 4.102 Machine application

Joint filling is possible by applying reinforcement tape, so-called joint tape, e. g. made of paper or glass fiber, or with special filling materials without using joint tape.
If movement from temperature or moisture changes or load is expected, always fill using joint tape in order to achieve sufficient security against crack formation. This particularly applies to drywall structures in earthquake-endangered areas.

Figure 4.103 Filling with paper joint tape

Figure 4.104 Filling without joint tape

For the filing compounds that are being used, please note the recommendations of the gypsum board manufacturer.

The filling is carried out in several work cycles.

First of all, the joint is filled with the filling compound, then the tape is filled in with a second layer of filler.

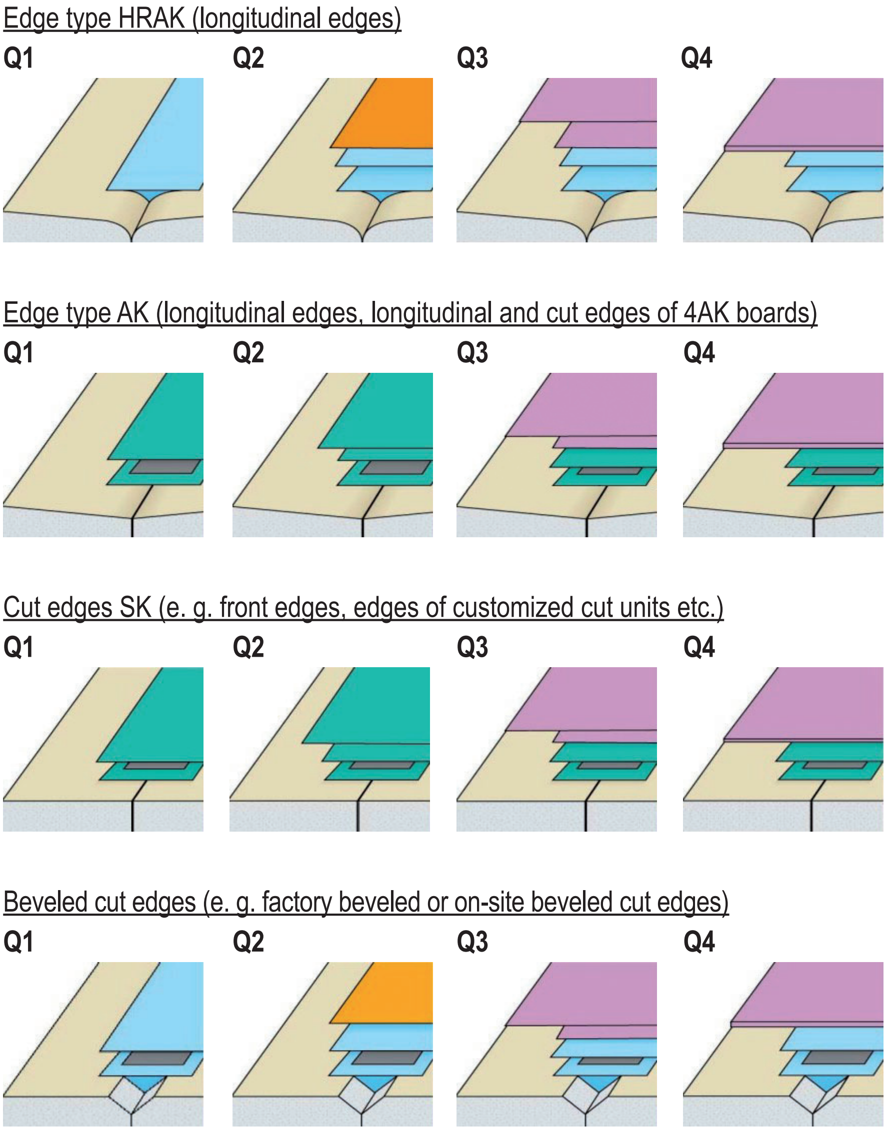
The further work cycles are aimed at achieving the desired surface quality.

As surface quality is very subjective, but requires a technical definition for contractual purposes and also for its execution, four quality levels have been defined in the German-speaking area, which specify the nature of the execution and the evenness tolerances that are to be adhered to.

More information /4.24/ (refer to the bibliography)

Table 4.19 Classification of the surface quality levels

Description	Execution	Application area
Q1 - Basic joint filling		
Surface quality of joints where there are no requirements placed on the visual (decorative) appearance.	A filling of the joints of the gypsum boards and covering the visible parts of the fasteners is sufficient. Protruding filler material must be chipped off, tool-related markings, ridges and grooves are permitted.	Filling the concealed boards in the case of multi-layered cladding. Surfaces that are to be provided with linings and coverings of tiles and panels or even thicker layers of plaster.
Q2 - Standard finishing		
Corresponds to the standard finishing and the standard requirements on wall and ceiling surfaces suffice. The aim is to level out the joint area with a gradual transition of the board surface. The same applies to fasteners, internal and external corners as well as connections.	The finishing in accordance with quality level Q2 includes basic joint filling (Q1) and finishing (fine filling) until an even transition to the board surface is achieved. No processing imprints or trowel marks must be evident. If necessary, the filled areas must be sanded.	For medium and rough textured wall claddings (e. g. ingrain wallpaper), matt filling paints (e. g. dispersion paint), which are being applied manually using a lambskin or textured roller, and for finishing plasters with a coarse grain thicker than 1 mm. Shadowing and marks that can occur in case of streaks of light cannot be excluded.
Q3 - Special finishing		
In case of advanced requirements on the finished surface, i.e. additional measures beyond basic filling and standard finishing are required.	Quality level Q3 incorporates the standard finishing (Q2) and a clear covering with jointing compound of the rest of the board up to the pore line in the board liner. If required, the filled areas must be sanded down to create a smooth, flush surface.	For finely structured wall coverings, matt non-textured paints / coatings and finishing plasters with a coarse grain < 1 mm. Even in the case of enhanced jointing and finishing techniques Q3, certain types of shadowing and marks cannot be totally avoided, especially when the drywall surface is highlighted by streaks of light. The degree and range of such shadowing and marks on the drywall surface is, however, lower than in the case of standard finished surfaces (Q2).
Q4 Skim coating		
Quality level 4 is created by an all-over skim coating of the entire surface, in order to comply with the highest demands on surface quality.	By contrast with special finishing, (Q3) the entire board surface is covered with a consistent coat of either finishing compound or skim plaster. Quality level Q4 incorporates the standard finishing (Q2) and a broad covering of the joints as well as an all-over covering and smoothing of the entire surface with suitable finishing compound (coat thickness up to around 3 mm).	For smooth or textured glossy wall linings, translucent glaze or paints / coatings up to medium gloss or stuccolustro or other high-quality smoothing techniques. This surface treatment minimizes the possibility of occuring marks and shadows in the board surface and joints. Undesirable effects from light (e. g. streaks of light) are largely avoided.



- Joint compound, e. g. Uniflott or for of impregnated boards Uniflott impregnated
- Joint compound, e. g. Fugenfüller Leicht or Jointfiller Super
- Joint tape
- Finishing compound, e. g. Finish-Pastös
- Skim coat, e. g. Readygips or for Q4 also Multi-Finish

Figure 4.105 Composition of the filling and finishing for various surface quality categories and for various board edge types

When filling apertura boards joint tape cannot be used in view of the perforations. What makes this more difficult is that the edges of the apertura boards with continuous perforations have square edges.
Therefore, the surface for the bonding between the filling compound and the gypsum board is very small. For this reason, these boards have to be filled with particular carefulness. The joints must be thoroughly free from building dust and primed sufficiently. When applying the filling compounds ensure that the joints are filled completely.

Figure 4.106 Joint filling of apertura boards

Another area that needs a lot of attention in order to avoid undesired cracking is the connections to flanking components.
Because of the different deformation behaviors of the adjacent components, caused by thermal stress, moisture or loads, it is easy for cracks to occur in these areas.
The mostly standard but unfavorable variant is to fill the connection joints with acryl. Acryl filling compounds with a ductility of between 10 and 20% are available. This means that connection joints which are completely filled with acryl filling compound must be at least 5 mm wide in order to enable a deformation of 0.5 mm. This is, however, only ensured if the acryl filling compound is bonded to two opposing flanks and only to these two flanks. If it is bonded to two neighboring flanks, i.e. in a corner situation, not even this small degree of deformability is guaranteed.

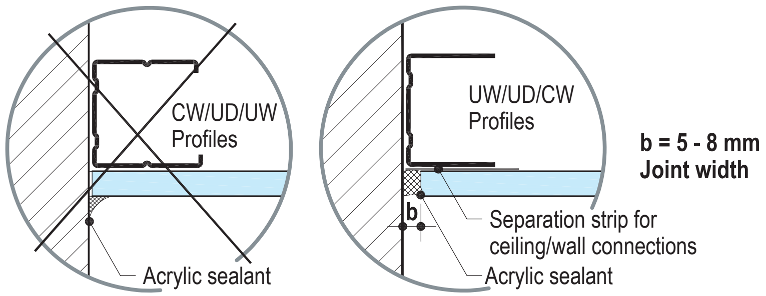


Figure 4.107 Connection joint with acryl: incorrect and correct design

However, because of the reduced deformability, even if the acryl connection joint has been executed correctly, it will crack and the fracture will be jagged and clearly visible. A better option is offered if you fill using separation strips.

To do this, a self-adhesive paper strip is stuck to the flanking wall or to the edge profile of the drywall structure, and normal filling compound is then used to fill the tape.

The joint should therefore be between 3 – 5 mm wide. After it has dried, the protruding tape is simply cut off.

In the subsequent wallpapering or painting, the separation joint must be carried over into the connection area, for example, the wallpaper must meet in the connection area and not bypass it.

Even this option doesn't offer total security against cracking. However, in the event of a crack, the crack formation is barely visible.

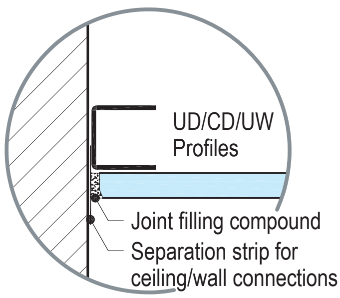


Figure 4.108 Connection design with separation strip

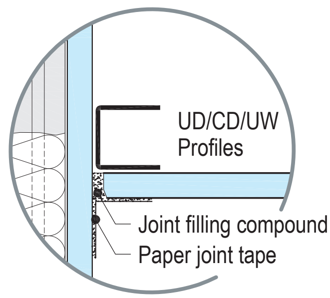


Figure 4.109 Connection design with joint tape

When connecting to a drywall structure, by bypassing the joint tape you can achieve a good crack security (Figure 4.109).

The only possibility of completely excluding cracks is to form a shadow gap. This variant is recommended for areas where there are frequent earthquakes, in order to reduce the reconstruction work. Shadow gaps can be combined with sliding connections, hence further contributing towards minimizing the damage potential.

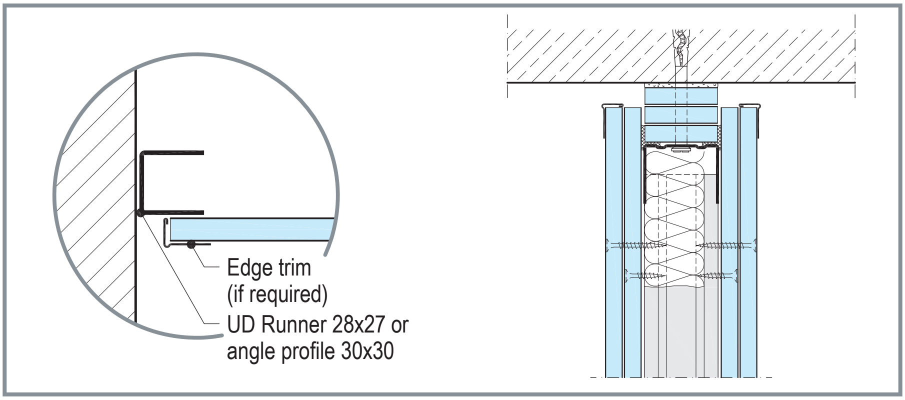


Figure 4.110 Connection design sliding, as shadow joint (ceiling and partition)

More Information /4.25/ (refer to the bibliography)

4.3.4.2 Joint Treatment for Gypsum Fiber Boards

Joints of gypsum fiber boards for wall cladding can be filled using three different procedures

<u>Filler joint</u>

In the case of filler joints, the boards are undersized by approx. 5 mm in the board width, so that a joint of around 5 mm width is available for filling on the board joint.
The filling is carried out manually using a suitable filling compound with considerable adhesion, e. g. Knauf Uniflott with or without joint tape. It is recommended to always use joint tape in order to prevent crack formation.
The joints are completely filled with filling compound. Protruding material (ridges) is chipped off after the respective drying time (e. g. in the case of Knauf Uniflott, around 40 minutes).
When filling using paper joint tape, the filling compound is also applied over the edges of the boards and the tape is embedded. Visible fastener heads are also filled.

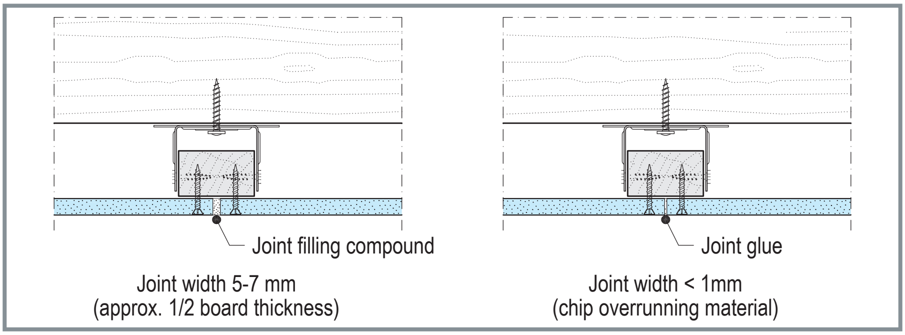


Figure 4.111 Filler joint and bond joint on gypsum fiber board joints

Bond joint KLF

When carrying out bonded joints, the boards are cut precisely to the dimensions (adapted to the grid dimensions of the stud spacing). The board is fitted to the substructure, then a suitable joint glue is applied to the edge of the fitted board evenly and in beads. The next board is joined with pressure (joint width < 1 mm) and immediately screw attached or stapled. The adhesive that seeps out can then be completely chipped off within an hour.

VT edge

The VT edge type is particularly suitable for filling using joint tape.

The longitudinal edges of the board have a milled recess so that they can be filled with joint tapes and without elevations. Filling is carried out manually using a suitable filler compound and joint tape. The boards are joined tightly, the recess is filled with filling compound and the joint tape embedded.

Visible fastener heads are also filled.

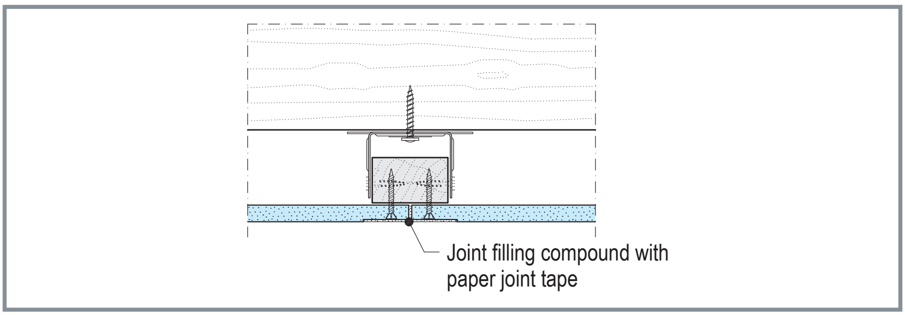


Figure 4.112 Filling with VT edge

In the case of all three procedures a subsequent fine filling, and, if necessary, a sanding of the filled areas is carried out.

4.3.4.3 Joint Treatment of Cement Wallboards

Cement wallboards AQUAPANEL® Cement Board Outdoor are fitted in the external area and also in the internal area and in the case of ceilings with a joint width of 3 – 5 mm.
The filling is carried out using a suitable filling compound and a 10 cm wide joint tape that is embedded during filling. If, in the façade area, just a coating of paint is to be applied to the basic plaster, you use a 33 cm wide reinforcement tape in place of the joint strip.
To protect the substructure from weathering, all the joints are filled using AQUAPANEL® Joint Filler - grey, and the 10 cm wide AQUAPANEL® Tape immediately after assembly. In addition, all the screw heads are filled using AQUAPANEL® Joint Filler - grey. The areas that are filled in this manner can be subjected to weathering for up to six months.

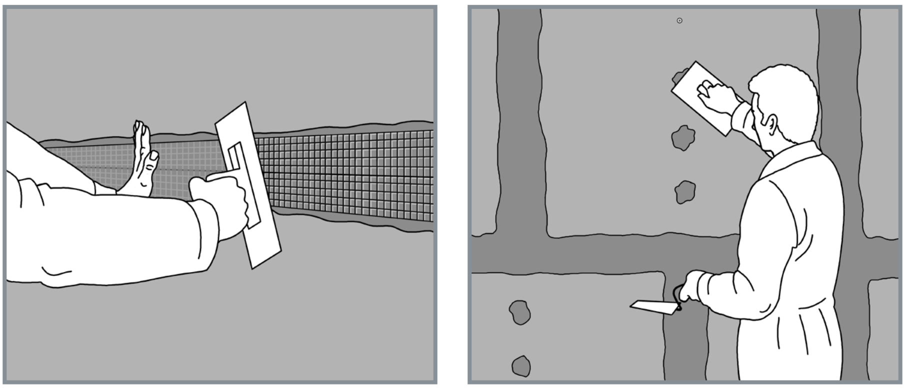


Figure 4.113 Filling in the joint tape and filling the screw heads

The AQUAPANEL® Exterior Basecoat is ideal for taking thin layers of finishing coats. Prior to application, the surface of the component made up of AQUAPANEL® Cement Board Outdoor must be dry and free from dust.

AQUAPANEL® Cement Board Outdoor can be provided with the AQUAPANEL® plaster system or a multitude of other plaster systems. Prior to plastering a reinforcement mortar with a mat embedded close to the surface is applied to the entire area.
After the respective setting time, the façade can be provided with any recommended finishing plaster or with lining tiles applied with mortar.
In earthquake areas, clinker or ceramic coverings (e. g. tiles) should only be applied if the manufacturer of the tiling system permits this. The connections to adjacent components are formed using pre-compressed sealing tape. Profiles are used as separation joints between plastering system and windows or doors.
All the external corners are initially provided with fabric corner angles or corner trim

profiles.
The AQUAPANEL® Exterior Basecoat is applied with a minimum thickness of 5 to 7 mm and then shaped to be level and true in alignment. Prior to applying the actual reinforcement, the corners of all building openings are reinforced diagonally with reinforcement strips (approx. 300 x 500 mm). The mat reinforcement is then embedded throughout, close to the surface, horizontally and free from folds. An overlapping of at least 10 cm must be complied with in the joint area.
The entire thickness of the reinforcement plaster made of AQUAPANEL® Exterior Basecoat and AQUAPANEL® Exterior Reinforcement Mesh should amount to at least 5 mm.
In the case of plaster structures of ≤ 1 mm thickness or with brushable, felted surfaces, AQUAPANEL® Tape should not be used. Instead of this, AQUAPANEL® Exterior Reinforcement Tape is applied on the joints.
Then basic plaster is applied by inserting the AQUAPANEL AQUAPANEL® Exterior Reinforcement Mesh.

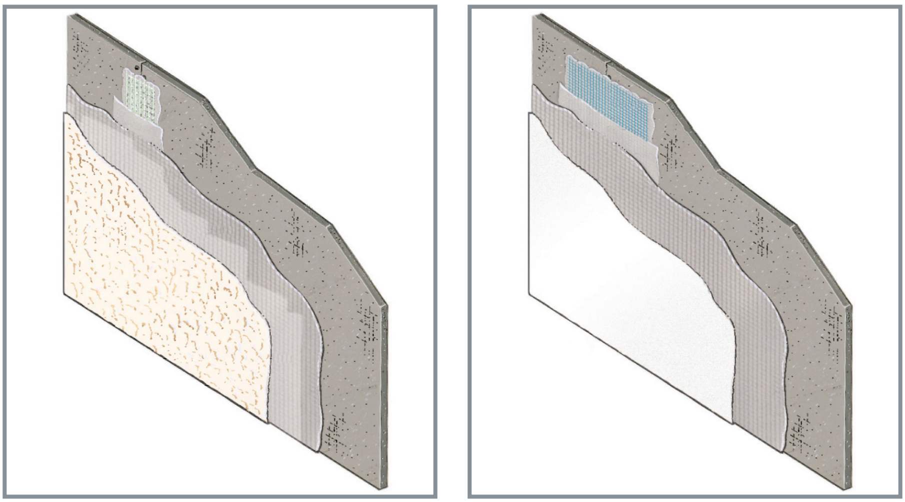


Figure 4.114 Filling and coating external walls with cement wallboards, left: plastering system, right: paint

Cement wallboards AQUAPANEL® Cement Board Indoor are bonded to each other in the wall area on the joints during assembly. On the edge of the already fitted board AQUAPANEL® Joint Adhesive (PU) is applied prior to applying the next board, and then the next board is attached and fixed with pressure. After the adhesive hardens, the surplus AQUAPANEL® Joint Adhesive (PU) is removed (usually the next day).
AQUAPANEL® Cement Board Indoor can be prepared for painting, by completely filling it using AQUAPANEL® Joint Filler and Skim Coating - White (minimum thickness 4mm). Then, AQUAPANEL® Interior Reinforcing Mesh is applied and embedded in the

layer by means of a trowel. To achieve a smooth surface AQUAPANEL® Joint Filler and Skim Coating - White is again applied in a thin layer. After drying, painting can commence.

More Information /4.14/ (refer to the bibliography)

4.3.5 Insulation

Insulation in drywall structures can fulfill a series of functions combined or even individually. In many drywall constructions, the insulating layer is a required constituent to ensure the desired or required properties.

4.3.5.1 Functions of Insulation in Drywall Constructions

Sound insulation

On account of its weight and the absorption of sound waves, the insulation helps to provide sound insulation. This also depends on the thickness of the insulation layer, its density and its flow resistance. In wall constructions it effectively works against airborne sound, in ceilings and floor structures it works against airborne and impact sound, and in connection with apertura boards as a sound absorber to improve the room acoustics.

Fire protection

The insulation layer's contribution towards fire protection consists in the insulation against the high temperatures from the fire stress. In the case of walls, the insulation layer has a particularly dampening affect on the surface temperature of the side that faces away from the fire.

In the case of ceiling constructions, the insulation protects the substructure from fire loads from above or dampens in terms of the temperature increase on the side that is facing away from the fire. The properties of the insulation relevant for the fire protection are density, insulation layer thickness, melting point and fire behavior (building material class).

It is differentiated between the insulation layers that are necessary in terms of fire protection, which generally place high demands on the fire behavior and the permissible insulation layer for fire protection, which do not themselves contribute towards the fire protection properties of the construction, but are permissible to ensure further properties.

Thermal insulation

Insulation in wall or ceiling cavities or under flooring constructions considerably reduces the thermal conductivity of the entire structure. This way, the transmission energy loss can be minimized by these components. This applies both for heating energy as

well as air-conditioning energy.
The result depends on the thickness and thermal conductivity capability of the insulation layer.

4.3.5.2 Insulation Materials

Insulation materials are divided into two priority groups.

- Fiber insulating material
  - Insulating materials made of mineral fibers (rock / glass wool) as mineral wool
  - Insulating materials made of plant fibers (coconut / wood / peat / flax / cellulose fibers)

  The fiber insulation materials are used in drywall constructions mainly as mineral wool in the area of the fire protection and sound insulation and the sound absorption in connection with thermal insulation in ceilings and wall constructions, but they are also used in flooring constructions.
  Rock wool with a melting point in excess of 1000 °C is often required in constructions with a high fire protection requirement. Wool made of glass belongs to the same building class (A1), but has a lower melting point and is therefore not suitable for all fire protection constructions.
  On account of their fire behavior, insulation materials made of plant fibers are unsuitable as a necessary insulation layer for fire protection constructions, but can be permitted. They are used for sound and thermal insulation.
  The specifications of the manufacturer and standards must be observed.

- Cellular plastics
  - Polystyrene (PS)
  - Polyurethane (PUR)
  - Phenol resin (PF)

  Cellular plastics, especially polystyrene, are mainly used in drywalling in the area of thermal and sound insulation for floor constructions or in association with gypsum boards as dry lining. Polystyrene is unsuitable as a necessary insulation layer for fire protection constructions; however, if an insulation layer is not required for fire protection purposes, polystyrene can be authorized for use as a sound or thermal insulation.
  The specifications of the manufacturer and standards must be observed here.
  The strength of these plastics and hence their restricted compressibility makes them particularly suitable for floor constructions.

4.4 Choosing Suitable Drywall Structures and Constructions in Accordance with Requirements

On account of the virtually unrestricted combination possibilities of their components, drywall systems place high demands on choosing the correct systems for concrete applications. Serious manufacturers make it easier for the user to choose by offering tested system solutions shown in a user-friendly manner in the technical documents for a multitude of application cases with respective requirements.
This particularly concerns what has been “tested”, otherwise, the required properties are ensured neither by the manufacturer of the components nor by the producer of the component (processor).

The demands on the component that are of main importance; are those which guarantee the safety for the user, i.e. structural stability and fire protection. In doing so, structural stability must be guaranteed for dead weight, earthquake load and, if necessary, further loads specified by standards or purpose.
The structural stability mainly restricts geometries, like the wall heights and maximum span, or spacing distances of the substructure.
Design of the substructure and the cladding thickness also play a role.
The fire protection requirements result from the application area of the component and the constructional supervisory requirement that exists in this case. Fire protection requirements are usually taken into consideration by choosing the building materials, the cladding thickness and the insulation materials as well as the insulation material thickness.
If both these criteria are defined, the constructional conditions, especially the geometries, the wall heights, maximum spans, component thickness, etc. are taken into consideration. They are closely connected to the specifications in terms of structural stability.
Sound insulation is a further criterion, it is also guaranteed by the selection of cladding thickness and also the type, the insulation and, in the case of particularly high requirements, by a special type of substructure.

4.4.1 Choosing for Ceiling Linings and Subceilings

Establishing the required fire resistance of the component in accordance with constructional supervisory requirements: Fire resistance duration, direction of the fire stress and whether the drywall ceiling is acting alone or in connection with the basic ceiling e. g. *0.5 h / F30 / EI 30 for the subceiling for the fire load solely from below*

↓

Establishing the required cladding thickness and insulation in accordance with requirements of the system manufacturer and/or standards

↓

Establishing the parameters for earthquake dimensioning: soil factor S and design acceleration a_g

↓

Establishing the load class for required cladding, insulation and, if necessary, additional loads whilst taking into consideration the earthquake load with tables in Appendix C1

↓

Determining the variant of the substructure: wood/metal, substructure with one or two levels, depending on available construction height, material and load class

↓

Determining the spacing and fixing distances of the substructure in accordance with the specifications of the system manufacturer

↓

Choosing the suitable hangers whilst taking into consideration the suspension height

4.4.2 Choosing for Partitions

Establishing the required fire resistance of the component in accordance with constructional supervisory requirements: e. g. *0.5 h / F30 / EI 30*

Establishing the required sound reduction index of the partition from the required resulting sound reduction index whilst taking into account the longitudinal sound reduction values

↓

Determining the substructure variants, cladding thickness and possible insulation in accordance with the specifications of the system manufacturer and/or standards

↓

Establishing the parameters for earthquake dimensioning: soil factor S and design acceleration a_g

↓

Determining the required profile dimensions via permissible wall height in accordance with Appendix C2

4.4.3 Choosing for Floor Constructions

Establishing the intended live load

Establishing the required airborne and impact sound insulation in interaction with the basic ceiling and possibly subceiling / ceiling lining

↓

Prefabricated screed:	Selecting the substructure and the thickness of the prefabricated screed elements; the substructure depends also on any necessary surface compensation
Hollow floors:	Selecting the pedestal grid and the thickness of and number of layers of hollow floor units

↓

Hollow floors:	If necessary, proof of earthquake resistance by the structural engineer

4.4.4 Choosing for External Walls

Establishing the required fire resistance of the component in accordance with constructional supervisory requirements: e. g. *1 h / F60 / EI 60*

↓

Determining the substructure variants, cladding thickness and possible insulation in accordance with the specifications of the system manufacturer and/or standards

↓

Establishing the parameters for earthquake dimensioning: soil factor S and design acceleration a_g

Establishing the design wind load in accordance with local conditions and valid standards/guidelines on load assumptions

↓

Determining the required profile dimensions via permissible wall height in accordance with Appendix C4

Appendix

A Basics of Structural Dynamics - Multi-Degree-of-Freedom-Systems

A.1 Equations of Motion

In simple cases, when dealing with systems with few degrees of freedom the equations of motion can be established directly. Nowadays, the use of the finite-element method for this purpose has become general practice. When using the matrix notation there is only little difference between the resulting equations of motion and the equation of motion of the oscillator with one degree of freedom:

$$\underline{M} \cdot \underline{\ddot{y}} + (\underline{R} \cdot \underline{\dot{y}} +) \underline{K} \cdot \underline{y} = -\underline{M} \cdot \underline{e} \cdot \ddot{z}(t) \qquad \text{(A.1)}$$

with:

$\underline{M}$ = Mass matrix, (n x n)

$\underline{R}$ = Damping matrix, (n x n)

$\underline{K}$ = Stiffness matrix, (n x n)

$\underline{x}$ = Vector of the absolute displacements (coordinates, n)

$\underline{y}$ = Vector of the relative displacements = $\underline{x} - \underline{e} \cdot z(t)$, ($n$)

$\underline{e}$ = Rigid-body vector, (n)

$z(t)$ = (Scalar) excitation at all points of support

Mass, damper stiffness and spring stiffness are replaced by the respective matrices, absolute and relative displacements by the respective vectors. Since the displacement vectors can also include angles of rotation the rigid-body vector $\underline{e}$ must provide for the difference to be calculated only at the proper displacement co-ordinates; therefore, it is 1 there and on the angle co-ordinates it is 0.

The damping term is put in brackets because it is most rarely that an explicit damping matrix exists. Particularly with earthquake it is usual to consider damping only implicitly, by a corresponding response spectrum (cf. sections to follow). For that reason the damping subject including its various variants and their mathematical formulation are not further addressed in this book.

Nor does this book address the finite-element method in more detail; extensive specialized literature for the expert is available. The method is automatically included in the relevant software, though partly not explicitly noticed by the practicing engineer, to whom it is all the more important to be in the position to evaluate and check the analysis results. For logical reasons analysis results should be evaluated and checked in two steps:

(1) Check of the load bearing behavior of the structural model on the basis of three static load cases with unit accelerations 1 g in the three directions.
(2) Check of the vibration behavior on the basis of elementary systems with one of few degrees of freedom.

By doing so, the engineer will avoid that by mistake he assumes the dynamic behavior of the structure to be the reason for absurd results of the dynamic analysis - which in reality are due to insufficient model generation.

A.2 Free Vibrations

As a first step the free vibrations shall be considered again. When for that purpose the right side is set to zero and the damping matrix is neglected, (A.1) results to:

$$\underline{M} \cdot \underline{\ddot{y}} + \underline{K} \cdot \underline{y} = \underline{0} \tag{A.2}$$

With the assumption $\underline{y} = \underline{\phi} \cdot e^{i \cdot \omega \cdot t}$ leads to the (real) eigenvalue problem:

$$(\underline{K} - \omega^2 \cdot \underline{M}) \cdot \underline{\phi} = \underline{0} \tag{A.3}$$

which can be solved with standard software, showing N natural frequencies ω and N related mode shapes (eigenvectors) $\underline{\phi}$ as a result. The eigenvectors present a shape only, the amplitude is free and can be chosen arbitrarily (normalized). The results get most lucid for the engineer when the maximum value of the displacement is normalized to 1. Figure A.1 shows the three mode shapes of a three-story, plane frame as example, with φ_{ik} meaning the elements of the k-th eigenvector at the i-th position.

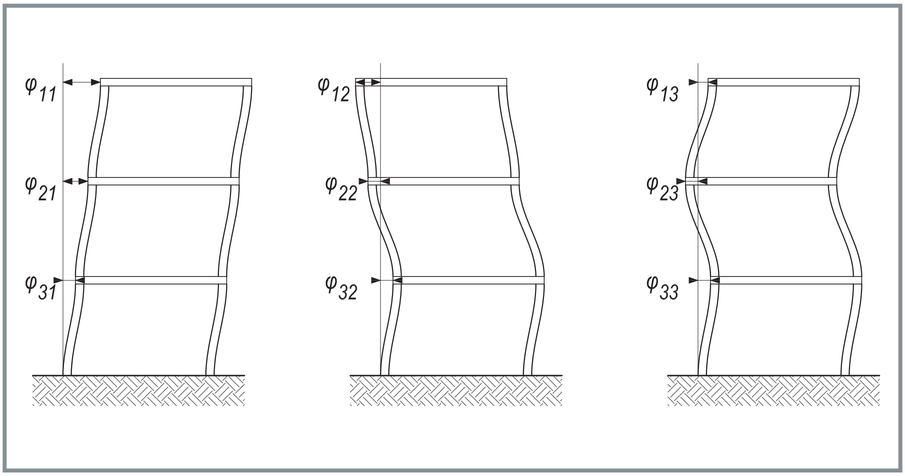


Figure A.1 a Mode shapes of a three-story plane frame

Compared to Figure A.1 a, Figure A.1 b shows the first three spatial mode shapes of a five-story building.

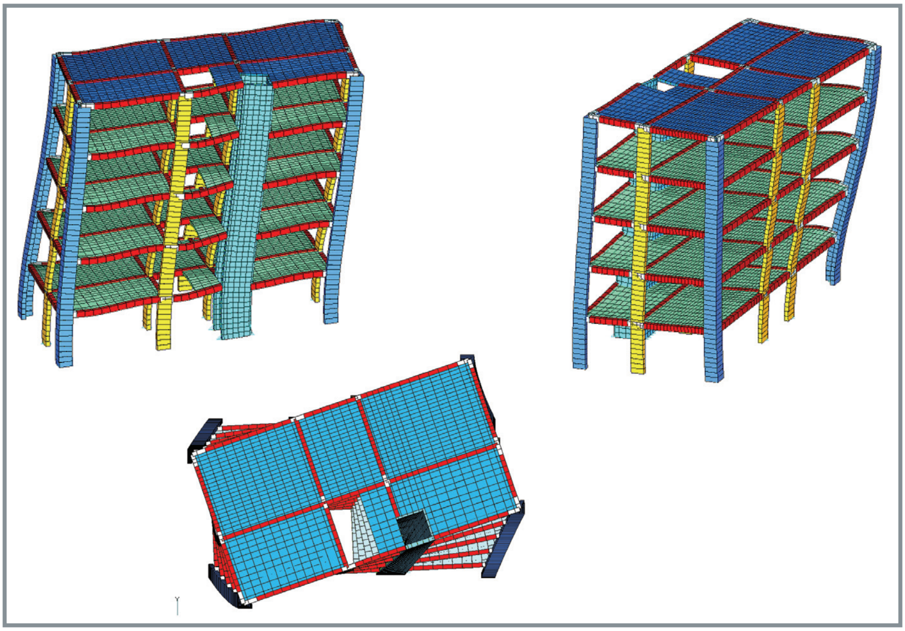


Figure A.1 b Spatial mode shapes of a complete building

The eigenvectors of a system are independent of each other, i. e. in a generalized sense they are orthogonal to each other. Their product weighted with mass or stiffness matrix differs from 0 only if they are multiplied by themselves:

$$\begin{aligned} &\underline{\phi}_K^T \cdot \underline{M} \cdot \underline{\phi}_L = 0 \quad for\, K \neq L \quad or = m_{LL} \quad for\, K = L \\ &\underline{\phi}_K^T \cdot \underline{K} \cdot \underline{\phi}_L = 0 \quad for\, K \neq L \quad or = k_{LL} \quad for\, K = L \end{aligned} \qquad \omega_L^2 = \frac{k_{LL}}{m_{LL}} \tag{A.4}$$

For indexing purposes this formula uses capital letters when the corresponding mode shape is addressed (modal index), small letters index co-ordinates in the coupled equation system.

By means of this procedure it is possible to decompose any free vibration of the global system into the N natural modes which each for itself present a system with one degree of freedom. And the same must apply to the forced vibrations, the result of which must be able to be composed from that resulting from N systems with one degree of freedom, as shown in the following chapter.

A.3 Forced Vibrations, Modal Analysis

It is possible, of course, to integrate the equations of motion (A.1) also directly, which - with the exception of some special cases - presents the only way possible when dealing with non-linear structural behavior. The mathematical methods as necessary for this purpose are highly sophisticated and are offered by the relevant program systems. Regarding the parameters to be applied the information in the respective manuals must be observed.

When dealing with linear problems, however, it is more useful to follow the modal analysis method which has already been addressed in sections A.1 and A.2, i. e. to compose the solution from the individual modal contributions. This procedure does not only present an especially fast analysis method, but the determination of natural frequencies, mode shapes and modal parameters carried out beforehand provides a better understanding of the dynamic behavior of the structure. This method is now considered in more detail:

When now assuming the solution of (A.1) as sum of the weighted contributions of all eigenvectors

$$\underline{y} = \sum_{L=1}^{N} \underline{\phi}_L \cdot q_L(t) = \underline{\Phi} \cdot \underline{q}(t), \tag{A.5}$$

safter inserting (A.5), multiplying the equation from left by the transposed eigenvector

$\underline{\phi}_K^T$ and neglecting the damping, this leads to the expression

$$\underline{\phi}_K^T \left| \sum_{L=1}^{N} \left\{ \underline{M} \cdot \underline{\phi}_L \cdot \ddot{q}_L + \underline{K} \cdot \underline{\phi}_L \cdot q_L \right\} = -\underline{M} \cdot \underline{e} \cdot \ddot{z}(t) \right. \tag{A.6}$$

As expected, due to the orthogonality relation (A.4) the coupled equation system with n degrees of freedom resolves into N decoupled equations that correspond to those of the system with one degree of freedom formally:

$$m_{LL} \cdot \ddot{q}_L + k_{LL} \cdot q_L = -\underline{\phi}_L^T \cdot \underline{M} \cdot \underline{e} \cdot \ddot{z}(t) = -m_L \cdot \ddot{z}(t) \tag{A.7}$$

Mass and spring stiffness are replaced by generalized values, on the right side results the excited mass m_L. And while in the case of the real oscillator with one degree of freedom after division by the mass only the base acceleration remains on the right side, in the case of the generalized oscillator with one degree of freedom the modal participation factor Γ_L adds as a factor:

$$\Gamma_L = \frac{m_L}{m_{LL}} = \frac{\underline{\phi}_L^T \cdot \underline{M} \cdot \underline{e}}{\underline{\phi}_L^T \cdot \underline{M} \cdot \underline{\phi}_L} \tag{A.8}$$

All standard programs of structural dynamics show this factor as important intermediate result. Due to the fact that it contains the eigenvector only once in the numerator, but twice in the denominator, this factor depends on the normalization of the eigenvectors. Only parameter combinations bringing an eigenvector in the numerator additionally do not depend on the normalization. The most important combinations which at the same time present valuable checks are as follows:

$$\sum_{L=1}^{N} \underline{\phi}_L \cdot \Gamma_L = \underline{e} \tag{A.9}$$

$$\sum_{L=1}^{N} m_L \cdot \Gamma_L = m_{ges} \tag{A.10}$$

Thus, the sum of all eigenvectors, weighted with *Γ*, results in a constant 1. Equation (A.9) shows that the static load case of a unit acceleration can be described via the sum of all eigenvectors. Consequently, equation (A.10) offers a possibility to verify whether or not sufficient eigenvectors were considered with the forced vibrations; for the total mass m_{ges} of the system must be reached to a high percentage.

If with the forced vibrations the time functions $\ddot{q}_L(t)$ and $q_L(t)$ are determined as result from N modal equations, the individual modal contributions can be superpositioned in phase according to assumption (A.5) for the global solution. This procedure is called Time History Modal Analysis, THMA. If only the absolute maximum values $|\ddot{q}_L|^{max}$ and $|q_L|^{max}$ from a response spectrum are determined,

$$\begin{aligned}
\left|q_L\right|^{max} &= DLF(\omega_L, D_L) \cdot \frac{m_L}{k_{LL}} \cdot \ddot{z}^{max} \\
&= DLF(\omega_L, D_L) \cdot a_0 \frac{m_L}{m_{LL}} \cdot \frac{1}{\omega_L{}^2} = \Gamma_L \cdot S_a(\omega_L, D_L) \frac{1}{\omega_L{}^2} \\
\left|\ddot{q}_L\right|^{max} &= DLF(\omega_L, D_L) \cdot \frac{m_L}{m_{LL}} \cdot \ddot{z}^{max} = \Gamma_L \cdot S_a(\omega_L, D_L) \quad ,
\end{aligned} \tag{A.11}$$

the superpositioning must be carried out approximately. This procedure is called Response Spectrum Modal Analysis, RSMA. The method of superposition chosen for the individual modal contributions is decisive for the quality of the result, i. e. for the quality of the approximation of the result to an “exact” Time History Modal Analysis. An inappropriate method of superposition can lead to completely absurd results. This will be explained in the following chapter.

A.4 Special: Response Spectrum Modal Analysis

The essence of the Response Spectrum Modal Analysis (RSMA) consists in the method of the approximate superposition of the individual modal contributions. By today's standards there are four methods of superposition available, based on mechanical aspects, which are sufficient for practical application. Further formula variants can be found in various codes and regulations. They can be explained historically, but in part are doubtful or overaged.

With R being an arbitrary system response quantity and R_L, R_K the related individual modal contributions, the four methods of superposition mentioned above, namely Algebraic Sum (ALS), Absolute Sum (ABS), Root of the Sum of Squares (RSS) and Complete Quadratic Combination (CQC) read (always summation of all N contributions):

ALS (Algebrahic Sum) $$R = \sum_{L=1}^{N} R_L \qquad \text{(A.12 a)}$$

ABS (Absolut Sum) $$R = \sum_{L=1}^{N} |R_L| \qquad \text{(A.12 b)}$$

RSS (Root of the Sum of the Squares) $$R = \sqrt{\sum_{L=1}^{N} (R_L)^2} \qquad \text{(A.12 c)}$$

CQC (Complete Quadratic Combination) $$R = \sqrt{\sum_{L=1}^{N} R_L \, R_K \, \rho_{LK}} \qquad \text{(A.12 d)}$$

with $$\rho_{LK} = \frac{8D^2 \cdot (1+r) \cdot r^{1.5}}{(1-r^2)^2 + 4D^2 r \cdot (1+r)^2} \text{ for } D = \text{const. and } r = \frac{\omega_L}{\omega_K}$$

With constant spectrum, equation (A.12 a), the algebraic superposition with consideration of signs, corresponds - as can be expected because of (A.9) - to a static load case “constant acceleration”. Within the scope of the earthquake analysis it becomes important with the so-called rigid-body term (see Chapter A.5).
Equation (A.12 b) presents a theoretical upper limit. For that reason it has been integrated in many codes and regulations - in both the original as well as in a modified form - although it often leads to absurd high results.
Equation (A. 12 c) presents the standard superposition in most codes and regulations. It is based on the assumption that the individual modal contributions R_L are statistically independent, so it indicates the statistically expected value. However, prerequisite is

that the individual natural frequencies are spaced sufficiently, at least 10 %. Then the RSS-superposition provides good results. If, however, this prerequisite is not fulfilled - i. e. with closely spaced natural frequencies - it provides absurd results, partly super-conservative, partly uncertain.

This deficiency is remedied by equation (A.12 d). It was developed in 1981 by Wilson and Der Kuregian (/A.1/), based on the theory of random vibrations and representing the state-of-the-art: In contrast to (A.12 c), here not only the diagonal elements of the interaction matrix $\underline{P}$ are considered, but also the interaction factors ρ_{LK} (≤ 1) outside the diagonal which for closely spaced frequencies can also reach values near 1. The results match those from the Time History Modal Analysis with high exactness, independent of the spacing of the natural frequencies. With sufficient spacing the CQC-superposition merges into the RSS-superposition identically. As a consequence and for reasons of safety and efficiency, the CQC-superposition should also be used if the code provides for the RSS-superposition method. Figure A.2 shows the interaction factors for constant damping *D* as a graph.

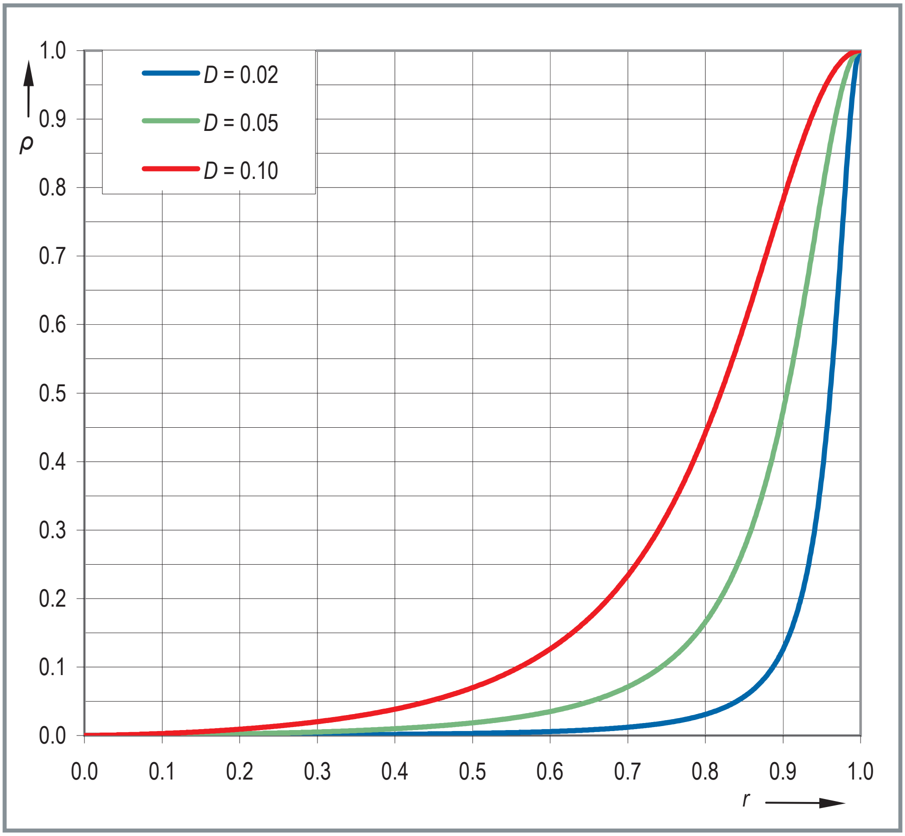


Figure A.2 Interaction factors of the CQC-method for D = const.

It can be recognized that only with very small damping factors (D < 0.02) and with a spacing of frequencies that exceeds 10 % the interaction of the modes can really be neglected.
A detailed discussion of all aspects of modal superposition within the scope of RSMA can be found with Gupta (/A.2/).

A.5 Rigid-Body Term

The methods of superposition on the basis of RSMA as addressed in Chapter A.4 assume that all individual natural frequencies are excited and that they show different phasing. Regarding the load case earthquake, however, this can apply only to natural frequencies up to approx. 25 Hz. As already mentioned before, within earthquake action higher natural modes show quasistatic behavior. As a consequence such modes must be superpositioned among each other with consideration of signs (ALS) and must be subjected to random superposition with the excitable natural modes (RSS). In connection with the CQC-superposition, with M natural modes in the excited range and M+1 to N remaining modes this means:

$$R_i = \sqrt{\sum_{L=1,K=1}^{M} \left(R_{L,i} R_{K,i} \rho_{LK} \right) + \left(\sum_{L=M+1}^{N} R_{L,i} \right)^2} \tag{A.13}$$

Moreover, an analysis of all these higher modes cannot be useful at all. For that reason the second part of equation (A.13) is modified, by considering that the algebraic sum (ALS) of all modes corresponds to a static load case with constant acceleration:

$$R_i = \sqrt{\sum_{L=1,K=1}^{M} \left(R_{L,i} R_{K,i} \rho_{LK} \right) + \left(R^0_{St,i} - \sum_{L=1}^{M} R^0_{L,i} \right)^2} \tag{A.14}$$

So the second part of equation (A.13) can be described by the difference between a static load case $R^0_{St,i}$ with the rigid-body acceleration of the response spectrum and the algebraic sum of the M natural frequencies that were needed already in the first term. Thus there is no more need to determine higher modes.

The second term of equation (A.13) or (A.14) respectively is the so-called rigid-body term (missing mass effect). It can be neglected mostly when dealing with weak systems. In limiting cases with very stiff systems, however, it presents the total solution. With systems with medium stiffness it is of essential importance so that it should be considered in any event. The relevant software generally offers respective options. If the software exceptionally does not possess an option for the handling of the rigid-body term, as a conservative assumption the second part of the second term can be dropped. This means that the result of the dynamic analysis with a limited number of considered natural modes then is superpositioned by a static load case with the rigid-body acceleration of the respective response spectrum under the root of the sum of squares:

$$R_i = \sqrt{\sum_{L=1,K=1}^{M} \left(R_{L,i} R_{K,i} \rho_{LK} \right) + \left(R_{St,i}^{0} \right)^2} \qquad \text{(A.15)}$$

Besides the modal superposition the rigid-body term presents the second core problem of Response Spectrum Modal Analysis since the objective of RSMA consists in approaching the result of Time History Modal Analysis as far as possible, at the same time also covering the limiting case of very stiff structures.

B Building Physical Properties of Drywall Constructions

Alongside the load bearing capacity and structural stability, drywall constructions mainly provide building physical properties that have already been discussed many times in the previous chapters.
This chapter aims to give a more detailed explanation of these properties.

B1 Fire protection

Alongside structural stability, fire protection is the property that plays a major role in the safety of those using buildings.
Fire protection encompasses preventing a fire outbreak and stopping it from spreading.

B1.1 Fire behavior and fire resistance

The most important terms in connection with fire protection are fire behavior and fire resistance.

In terms of fire behavior it is differentiated between the fire behavior of a single building material and that of a component which can be made up of materials with different fire behavior properties.
The most important indicator for fire behavior is the building material class. This is established for the respective building material either by means of standards or must be determined by a test upon the basis of which a constructional supervisory permission is presented as a building material of a respective building material class. The building material class makes a statement about the combustibility of the building material.
Further properties of fire behavior in accordance with EN 13501 are the smoke development and the dripping in the case of high temperatures.
When combined, the building material class, smoke behavior and dripping behavior make a statement about the fire behavior of the building material.
Using combustible materials should be avoided, that said, the use of these as building materials is not permitted in most countries.

Table B.1 Terms of fire behavior with reference to smoke development and dripping

Smoke development	
s1	No / barely any smoke development
s2	Limited smoke development
s3	Unlimited smoke development

s = smoke

Dripping / falling of burning elements	
d0	No dripping
d1	Limited dripping
d2	Considerable dripping

d = droplets

Table B.2 *Fire behavior according to EN 13501*

Constructional supervisory designations	Additional requirements		Fire behavior according to EN 13501-1
	No smoke	No dripping / falling down of burning elements	
Non-combustible	X	X	A1
	X	X	A2 -s1,d0
Not easily flammable	X	X	B,C -s1,d0
		X	A2, B, C -s3,d0
	X		A2, B, C -s1, 2
			A2, B, C -s3,d2
Flammable		X	D -s3,d0
			E
			D -s3,d2
			E -d2
Easily flammable			F

Gypsum boards are usually assigned fire behavior A2-s1,d0

The technical fire protection classification of components, i.e. building elements and constructions is carried out in accordance with fire resistance classes. In doing so, serviceability in the event of a fire is tested.
The tests on the components are carried out in the furnace at temperatures in accordance with a specified unit temperature time curve.

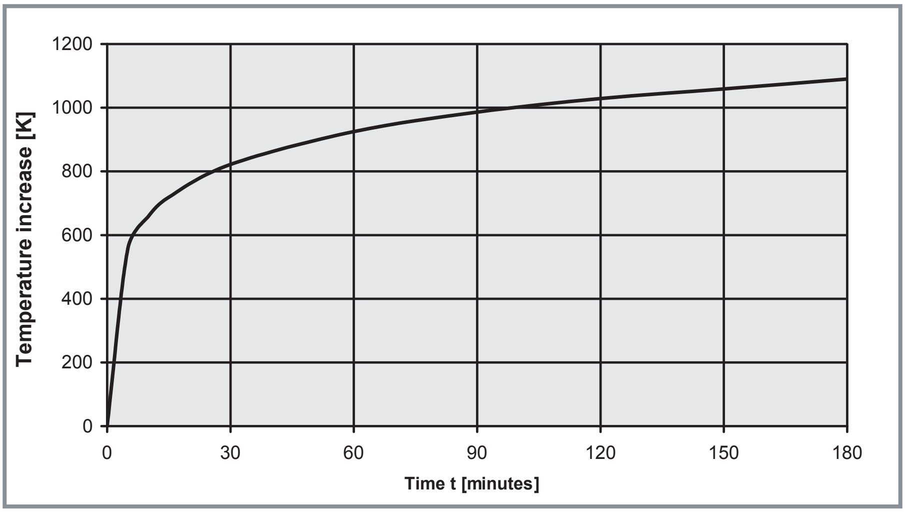


Figure B.1 Unit temperature time curve

A classification is carried out in accordance with the amount of time the component or the structure offers for resisting fire, whereby, depending on the function of the component in accordance with Table B.3, the combination of letters R, E and I and the time of fire resistance in minutes is specified. Failure criteria are the loss of enclosure of space and the loss of load bearing capacity as well as the increase in temperature on the side that is facing away from the fire.

Table B.3 Abbreviations for the function of the component in terms of fire resistance

Abbreviation		Criteria
R	(Résistance)	Load bearing capacity
E	(Étanchéité)	Room-enclosure
I	(Isolation)	Thermal insulation (under the effects of fire)
W	(Radiation)	Radiation protection
M	(Mechanical)	Mechanical effects on the wall (impact load)

Constructional supervisory requirements on components are usually set by means of terms.

One method of allocating these terms to technical properties that represent a combination of fire behavior and fire resistance class, based on German planning and building laws, is shown in Table B.4. This allocation should be used as an indication, but the individual national regulations could deviate from this.

In addition to this, it is differentiated between the fire behavior of the building materials used in components in

1. Components made up of non-combustible building materials,
2. Components whose load bearing and bracing parts are made of non-combustible building materials and, in the case of room-enclosing components, also have a consistent layer of non-combustible building materials at component levels,
3. Components, whose load bearing and reinforcing parts are made of combustible building materials and which on all sides have a cladding made of non-combustible building materials (fire protection cladding) and thermal insulation materials,
4. Components made up of combustible building materials.

Table B.4 Constructional supervisory designation and allocation of fire resistance classes

Construc-tional su-pervisory designation	**Load bearing components Without room enclosure**	**With room enclosure**	**Non-load bearing internal partitions**	**Non-load bearing external walls**	**Raised access floors**	**Indepen-dently effective subceilings**
Fire-retardant	R30	REI30	EI30	E30 (i→o) and EI30 (i←o)	REI30	EI30 (a↔b)
Highly fire-retardant	R60	REI60	EI60	E60 (i→o) and EI60 (i←o)		EI60 (a↔b)
Fire-resis-tant	R90	REI90	EI90	E90 (i→o) and EI90 (i←o)		EI90 (a↔b)
Fire resis-tance dura-tion 120 min.	R120	REI120	-	-		-
Firewall	-	REI-M90	EI-M90	-		-

i = in   o = out   a = above   b = below

B1.2 Fire Behavior of Drywalling Materials

Gypsum and gypsum fiber boards

Gypsum building materials are inorganic, non-combustible building materials. They belong to the classic fire protection building materials. The good protection in the event of a fire is essentially based on the content of around 20 % bonded crystal water (1 m² gypsum board, 15 mm thick, contains around 3 l crystal water). In the event of a fire, the gypsum is dehydrated, i.e. the crystal water evaporates. Energy is consumed, and because of the vapor haze that forms between the fire and the gypsum building material, the fire's progress is delayed.

Alongside the fire protection effects of the crystal water, the dehydrated gypsum layer acts as an additional insulator, as it has a lower thermal conductivity value than gypsum that has not been dehydrated.

Gypsum and gypsum fiber boards are usually assigned to building material classes A1 or A2 (DIN 4102) or fire behavior A1 or A2-s1,d0 (EN 13501).

Cement wallboards

Cement wallboards for façade applications rank among building material class A1.

Wooden substructure

If the substructure is made of wood, which belongs to building material class D or E (flammable), the ceiling lining or subceiling is classified as a construction whose load bearing and bracing parts are made of combustible building materials.

Metal substructure

Metal belongs to building material class A1. Drywall constructions with a metal substructure and a cladding made of gypsum boards therefore are components whose load bearing and bracing parts are made of non-combustible building materials and that have, in the case of room-enclosing components, additionally a continuous layer of non-combustible building materials in the components plane direction.

Insulation

The various insulation materials available on the market could belong to virtually all building material classes. Therefore it is very important to choose the insulation material that suits the construction carefully. Tested fire protection constructions often include minimum requirements on the fire behavior of the insulation materials being used.

It is essential that flammable insulation materials should not increase the fire load on the fire protection structure in the event of a fire, as these additional loads cannot be taken into consideration when establishing the fire resistance.

B1.3 Fire Behavior of Drywall Constructions

Drywall ceilings

Drywall ceiling constructions can ensure fire protection in two different ways.

1. If the drywall ceiling construction has a fire resistance class without the participation of the basic ceiling, it is called “sole fire protection”.
2. If the fire protection is achieved only in combination with the basic ceiling, then it is called “fire protection in connection with the basic ceiling”. The basic ceiling itself is assigned to various classes depending on its structure, and the overall fire protection effect depends on this.

Furthermore, the fire protection of ceiling constructions can be guaranteed from two directions alternatively or simultaneously, in connection with the basic ceiling or solely effective.

a) Fire protection of ceiling linings or subceilings “in connection with the basic ceiling“, is always guaranteed for a fire load from the top and from the bottom of the overall ceiling construction so as to protect the other rooms simultaneously, whereby, in a serious scenario, the fire load will of course take place only on one of the two sides. This type of fire protection can be guaranteed both by the subceilings as well as by the ceiling linings.

b) Fire protection “solely from above“ means that the fire protection is guaranteed for a fire load in the plenum between the basic ceiling and the subceiling to protect the room that is located beneath the subceiling. This type of fire protection is only taken into consideration in connection with subceilings, as ceiling linings do not have a plenum.

c) In the case of fire protection “solely from below“ the fire protection is given for a fire load from below the ceiling lining or subceiling, to protect the ceiling plenum and the basic ceiling. This type of fire protection can be guaranteed both by the subceilings as well as by the ceiling linings.

d) Fire protection “solely from above and from below“ means that the requirements as specified in b) and c) are fulfilled simultaneously by a subceiling. The fire load is therefore expected alternatively from the room beneath the subceiling or from the plenum.

Non-load bearing drywall partitions

For non-load bearing drywall partitions, the respective fire load duration regarding the the room-enclosing function and the maximum surface temperatures on the side facing away from the fire (as specified in the fire resistance class) are guaranteed.
In the case of firewalls, the resistance to impulsive loads is also tested in addition to the fire resistance.

Installation shaft walls

The fire protection of installation shaft walls has an effect both on the fire load from the shaft to prevent the fire from spreading to the surrounding rooms as well as the fire load from the room so as to maintain the function of the installations located in the shaft and the fire spreading to other floors.

Load bearing components

In the case of load bearing components, the protective aim is not just, as with non-load bearing walls, to offer fire protection to the neighboring room, but also to offer fire protection in view of the load bearing capacity of the overall structure. So, in the case of bracing wall constructions, which are braced by the cladding, it must be ensured that the bracing effect is also maintained for an adequate period of time in the event of a fire. For cladding on load bearing components, like for example, steel columns or girders, the temperature transition must be restricted to such an extent that critical temperatures that could have a negative effect on the load bearing capacity of steel, are prevented from penetrating the cladding for as long as required.

B1.4 Fire Protection in the System

Particularly in the area of fire protection it is of great importance to use only those components and materials that have been recommended by the manufacturer. Renowned manufacturers of drywall systems offer tested construction variants, which compared to constructions as regulated in standards, achieve an equal amount of fire protection with considerably less work. However, these constructions have been tested using only those components specified by the manufacturer and can also only guarantee the respective properties if the same components are used. Therefore, in the interests of safety and for drywall constructions of utmost quality it is absolutely necessary to remain "true to the system" when choosing materials. It should be considered that the warranty of a fire protection cannot be tested on the construction site (which is the case with sound insulation) and therefore a possibly defective construction cannot be complained. A respective fire protection will only be illustrated in a serious scenario. But then it's too late to make any improvements.

B2 Sound Insulation

Sound insulation is of considerable importance for the well-being of the users of buildings during the entire period of utilization.

Two types of sound transmission play a role here. Airborne sound, which is spread via the air and, structure-borne sound, which is spread through components (e. g. as impact sound).

Room-enclosing components like ceilings, walls and floors, have the task of insulating the sound that originates in one room from the other rooms to an acceptable degree. While airborne sound waves from the air meet the components, cause the components to vibrate, which in turn cause the air that is located on the side facing away from the component to create sound waves, the impact sound is caused by the direct bodily contact with the components, it is then transmitted through the component and the occurring vibrations convert this into perceivable (secondary) airborne sound.

If the room-enclosing components are able to dampen these vibrations as much as possible, which can be achieved by means of soft components and increased mass, then the sound penetration is reduced.

The extent of this reduction depends on the use of the rooms and the sound source. Sound protection standards, like, for example, the German standard DIN 4109 “Sound insulation in buildings“, offer reference values. Personal requirements can however require sound protection levels that exceed the requirements of standards.

It must be noted that it is not only the room-enclosing components themselves that contribute towards sound insulation, but also all the flanking components, like adjacent walls, continuous flooring constructions and continuous ceilings that transmit so-called longitudinal sound.

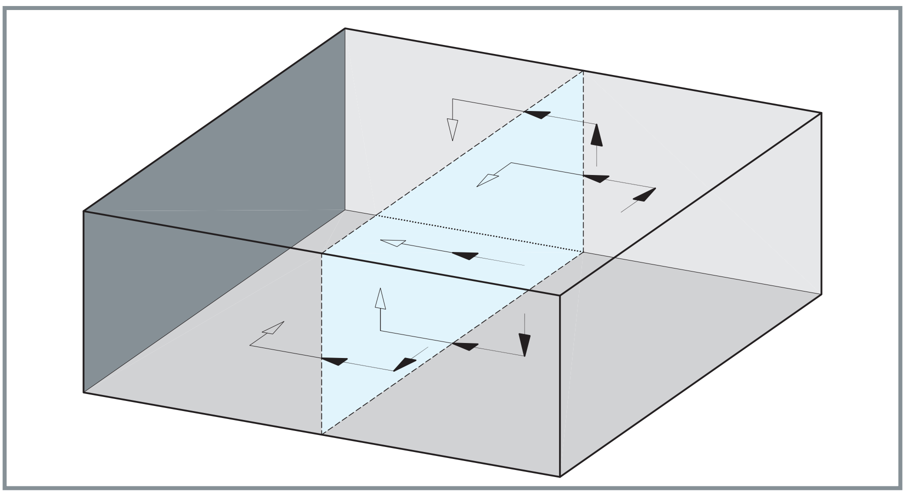


Figure B.2 Transmission paths of airborne sound

Therefore, room-enclosing components can always only ensure a resulting and hence effective sound protection in cooperation with its flanking components.
Individual sound protection values for specific constructions serve exclusively as a comparable assessment between the construction variants and for the establishment of the resulting sound protection whilst incorporating the longitudinal sound insulation of all flanking components.
Figure B.4 shows the establishment of the resulting sound reduction index for a partition.

The absolute tightness of the construction is imperative for a good sound insulation, as sound, similar to air or water, can “flow through” holes, so even a very small hole could render the sound insulation of the entire component useless.
These holes should exist neither in the surface nor on the edge connections of the component.
In the case of drywall structure surfaces, the required tightness is achieved by filling the board joints of all the cladding layers. In the area of the connections with flanking components, it is necessary to seal using a plasto-elastic joint cement, which ensures the component will also maintain a tight connection even in the case of unlevel bases.

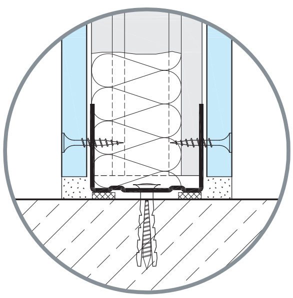
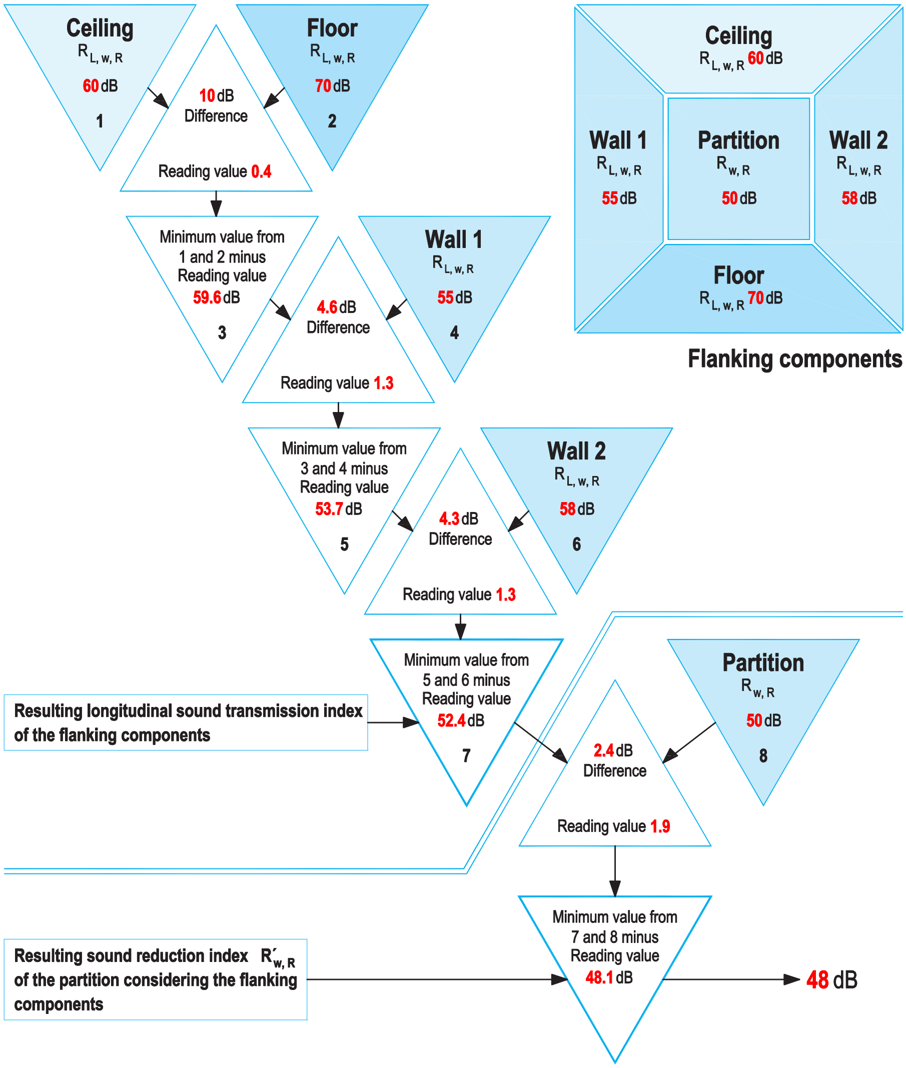


Figure B.3 Sealing the floor connection using joint mastic

Porous sealant tape, which is often applied beneath the perimeter profiles, does not provide sufficient tightness against sound.



Difference	0.0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5
Reading value	3.0	2.8	2.5	2.3	2.1	1.9	1.8	1.6	1.5	1.3	1.2	1.1	1.0	0.9	0.8	0.7

Difference	8.0	8.5	9.0	9.5	10.0	10.5	11.0	11.5	12.0	12.5	13.0	13.5	14.0	14.5	15.0 - 19.5	≥ 20
Reading value	0.6	0.6	0.5	0.5	0.4	0.4	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.2	0.1	0.0

Figure B.4 Calculation example for establishing the resulting sound insulation index

Impact sound can be transmitted across the flanking components in exactly the same way as airborne sound. Therefore, it is important to separate screed constructions above the impact sound insulation layer from the adjacent walls, by means of edge tape. Separation joints must also be provided in doors or in the case of continuous screeds under lightweight walls, in order to avoid the transmission of impact sound from one room to the next and also to prevent it passing through the walls and, as a result, in the ceiling and the rooms above it.
In the case of impact sound insulation, the entire ceiling structure is sound-insulating, i.e. the impact sound insulation can also be improved by using the appropriate subceilings or ceiling linings.
This option is of particular interest for later improvements in the event of a change of use or redevelopment work.

It also and especially applies to sound insulation requirements, that the sound insulation values specified by manufacturers will only apply if the recommended components are used. Sound insulation values therefore do not apply to constructions alone, but for the construction including the materials used when test-proofing the sound insulation values.
The sound insulation values of drywall systems especially in the area of profiles and boards react very sensitively to changes in the used products.

B3 Sound Absorption

In the case of acoustic measures it must be decided whether sound sources and the listeners are located in different or the same room.
In the first case scenario, the sound insulation is achieved mainly from the sound insulation, in the second case, from the sound absorption. This chapter explains the basics of sound absorption for improving room acoustics.

B3.1 Sound Absorption Terms

Sound absorption is generally known as the reduction of the sound energy in a room. Basically sound absorption is the loss of sound energy when it meets boundary surfaces, objects or people that are located in the room. The loss comes about primarily by converting sound to heat (dissipation).

- Sound absorption level a
  Sound absorption level a describes the ratio of the non-reflected to the initial sound energy. In the case of a complete reflection, a = 0, with complete absorption a = 1.
- Reverberation process
  Reverberation process is the reduction in the sound energy in a closed room after the finished acoustic irradiation or sound transmission.
- Reverberation time
  The sound absorption in a room is described by the reverberation time.
  It indicates the time frame in which a noise level decreases by 60 dB (decibel) after the end of a sound transmission.
- Absorbers
  Absorbers are components that contribute towards sound absorption.

B3.2 Absorbers

Absorbers can comprise different materials or component properties, which, when optimally combined, generate the best results.

- Mineral wool
  Mineral wool is predominantly used where requirements have been placed on fire protection and where a particularly high sound absorption is required in the low frequency range.
- Acoustic fleece
  In the case of acoustic ceilings from renowned manufacturers, the apertura boards are lined on the rear side with acoustic fleece which in most cases is a sufficient sound absorber.
  Additional sound-absorbing layers of insulation are only required for utmost demands.

- Air space

  The distance between the suspended acoustic ceiling and the basic ceiling is of significant importance for the degree of sound absorption. With suspension heights < 100 mm, the sound absorption values move in the direction of the high-frequency range.

  Large air spaces cause an increase in the sound absorption in the low-frequency range. As of 500 mm air space, the values change only slightly.
- Perforation ratio

  Experience has shown that the highest degree of sound absorption can be achieved with perforation ratios of 10 to 15%. In the case of perforation ratios < 10%, the values in the high frequencies fall and remain constant in the low frequencies.

  Precisely the opposite behavior turns out with a perforation ratio of > 15%, i.e. falling values for low and constant values for high frequencies.
- Perforation dimensions

  With a comparable ratio of perforated area, aperture boards with lots of small perforations achieve a better absorption in the high-frequency range.

B3.3 Audibility in Classrooms

The audibility of teachers in classrooms and auditoriums plays a critical role. A lack of room acoustics and the negative effects this has on learning performance cannot be compensated for by any other influencing factor. This is where a clear and precise planning by specialists can play a significant role in the usability of the room.

Alongside the useful signal (language) there is the interference signal (background noise). In order to understand the lecturers acoustically, a speaking volume for normal hearing must be at least twice as loud (10 dB sound pressure level difference), in the case of an audience with impaired hearing, at least three to four times as loud (15 – 20 dB sound pressure level difference) as this background noise.

Generally speaking, a teacher is capable of constantly talking at double the volume of any surrounding background noise level, if this does not exceed approx. 45 dB (A).

The A-rated sound pressure level of speakers at a distance of 1 m in classrooms and talking
- in a relaxed manner amounts to 54 dB(A)
- in a normal manner 60 dB(A)
- in an elevated manner 66 dB(A)
- in a loud manner 72 dB(A)

The A rating matches the graphs of equal sound volume levels at approx. 20 to 40 Phon.

However, should, as a result of constructional inadequacies or restless behavior, the background noise level increase to values in excess of 50 dB(A), then (even for a small

period of time) the teacher is subjected to an extremely high, unacceptable physical strain. This disrupts performance. You cannot adapt to background noises. Noise reduces the intake and processing of the lesson's content, the concentration capability of the listener / teacher is disrupted, the efficiency of the teacher is reduced.
The path difference between direct sound and the sound reflections makes a considerable difference to the clarity of speech intelligibility. A time difference below 50 ms has a positive effect, differences in excess of 50 ms however reduce clarity.
If the direction from which the speaker is viewed (visually perceived), does not agree with the direction from which the speaker is heared (acoustically perceived), concentration problems arise in trying to record the content of the presentation.
In classrooms, for example, a significant reduction in the initial sound energy from the direction of the speaker on account of built-in elements (girders, etc.) or a completely sound-absorbing subceiling can be of a disadvantage. In this case, the listener locates the sound source, e. g. in the direction of an energy-rich lateral or rear wall reflection.
Generally, the speech intelligibility reduces as the reverberation time increases. In reverberant rooms (e. g. in the corridors of schools), the speaker is much more difficult to understand than in office rooms. In doing so, the desired level of reverberation time is determined by the required understanding of syllables.
The most significant frequency range of reverberation time for speech lies between 100 to 5000 Hz.
For people with even minor hearing problems, the frequency range of 125 Hz should have a reverberation time that is as short as possible (T < 0.6 s). To achieve the same understanding of speech (more precisely: understanding of syllables) as someone of normal hearing, those of harder hearing require an even more reduced reverberation time in the room.
Classrooms with a length as of approx. 10 m which have no or only minor sound-absorbing materials in it are characterized by the lack of speech intelligibility at the back of the room.
As the reverberation time increases so does the expected behavior-related background noise. People who are residing in a quiet environment (short reverberation time) behave more calmly.

These influences can be controlled with the arrangement of absorbers in such a way that good speech intelligibility and hence optimum learning conditions can be created.
The acoustic planning for the room should, however, be carried out by specialists, as room acoustics are very complex and there are many factors that can influence the spread of sound in rooms. In the case of properties with considerable demands,

it could even be necessary to check the acoustics in the installation stages and, if necessary, use these results to carry out the necessary measures to improve the situation.

In areas of earthquake resistance, when installing absorbers ensure that, if required, the substructure is braced. When dimensioning the anchoring elements of absorbers, the earthquake forces must be taken into consideration. The same structural measures or dimensioning procedures apply to apertura board ceilings as they do for ceiling linings and subceilings. However, other spacing values must be complied with, refer to Appendix C1.5.

B4 Thermal Insulation

Saving energy serves to reduce the operating costs of a building as well as protect the environment's limited energy resources.

If the used energy is employed as effectively as possible, i.e. with as little loss as possible, there are great benefits from a considerable saving potential without reducing the user comfort.

In doing so, it should be taken into account that additional costs for a good thermal insulation will be a one-off, calculable payment. Greater energy costs will remain for the entire period of use and, on account of the incalculable energy prices remain unforeseeable.

The primary amount of energy is applied in buildings during their use in order to maintain a comfortable room temperature. Depending on the climatic zone, this includes air-conditioning energy to cool the interior air in the case of high temperatures or heating energy to heat up the room's air in the case of low external temperatures.

The thermal conductivity of the external components, at least the external walls and roof structures, on their external and internal surfaces are subjected to extremely different temperatures, and therefore have a major influence on the effectiveness of the energy being used.

In the case of poorly insulated building envelopes, the energy escapes via the roof, via the non-insulated external walls, leaking windows and doors and non-insulated cellar ceilings or base plates. This loss of energy could be prevented by means of a complete insulation of the building's external envelope. Every non-insulated area will cause a thermal bridge and increase the loss of energy and, simultaneously, the risk of mould development.

When looking at the energy account of a building, i.e. the total of energy gained and lost, four types of energy flow are considered:

Table B.5 Energy gains and losses

High external temperature, building is air-conditioned		**Low external temperature, building is heated**	
Effect	**Implication**	**Effect**	**Implication**
Transmission heat gain	Negative (room air is heated up)	Transmission heat loss	Negative (room air is cooled down)
Ventilation heat gain	Negative (cooled air leaks to outside)	Ventilation heat loss	Negative (heated up room air leaks to outside)
Radiation heat gain from windows and opaque components	Negative (room air is heated up)	Radiation heat gain from windows and opaque components	Positive (room air is additionally heated up)
Inner thermal sources (people, electrical devices, lighting)	Negative (room air is heated up)	Inner thermal sources (people, electrical devices, lighting)	Positive (room air is additionally heated up)

If designed properly, external components or room-enclosing components to unheated / non-air-conditioned rooms using drywalling can have a great influence on the transmitted heat gain or loss.

To determine this influence in order to plan energy savings, the thermal resistance is calculated for the entire construction. This is calculated from the thermal conductivity and thickness of the individual materials of the component.

The thermal conductivity λ specifies how much energy in watts (W) is transmitted through the material per 1 meter (m) component thickness and per 1 Kelvin (K) temperature difference on both sides of the material.

The the thermal resistance is made up of the total of the products

Thickness • thermal conductivity

for all material layers of the component

<u>Example:</u>

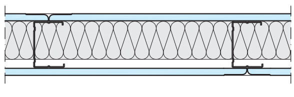
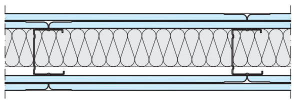
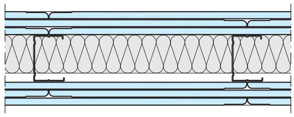
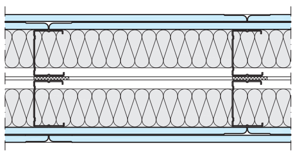
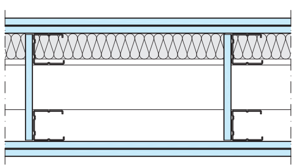
- Metal stud partition W112,
  double-layer cladding with 2 x 12.5 mm GKF boards
- Insulation in the cavity of the partition with mineral wool, thickness: 60 mm, thermal conductivity group 040 (λ = 0.040 W/(m•K))
- Thermal conductivity of the materials: Gypsum board: 0.25 W/(m•K)
  Mineral wool: 0.04 W/(m•K)
- Thermal resistance *R*: 15 mm air layer: 0.17 m²•K/W
- Thermal transition resistance on both wall surfaces: 0.13 m²•K/W

■ Total the thermal resistance *R*:

$$R = 2 \cdot 0.13 \frac{m^2 \cdot K}{W} + \frac{4 \cdot 0.0125\,m}{0.25 \frac{W}{m \cdot K}} + \frac{0.06\,m}{0.04 \frac{W}{m \cdot K}} + 0.17 \frac{m^2 \cdot K}{W} = \mathbf{2.13} \frac{\mathbf{m^2 \cdot K}}{\mathbf{W}}$$

The so-called thermal transmittance of a component, also known as the *U* value (former K value), is the reciprocal value of the overall thermal resistance, in the example *U* = 0.47 W/(m²•K).

Table B.6 Heat transition coefficients of Knauf metal stud partitions

Partition	Thickness [mm]	Cavity [mm]	Cladding gypsum boards[1] thickness [mm]	Insulation layer thickness[2] [mm]	Thermal transmittance [W/m²·K]
W111, metal stud partition, single studs, single-layer cladding					
	75	50		40	0.66
	100	75	12.5	60	0.50
	125	100		80	0.40
W112, metal stud partition, single studs, double-layer cladding					
	100	50		40	0.61
	125	75	2x 12.5	60	0.47
	150	100		80	0.38
W113, metal stud partition, single studs, triple-layer cladding					
	125	50		40	0.57
	150	75	3x 12.5	60	0.44
	175	100		80	0.36
W115, metal stud partition, double studs, double-layer cladding					
	155	105		2x40	0.37
	205	155	2x 12.5	2x60	0.27
	255	205		2x80	0.21
W116, installation wall, double studs, double-layer cladding					
	≥ 220	≥ 170	2x 12.5	40+60	0.34

[1] Gypsum boards acc. to EN 520 with thermal conductivity of 0.25 W/(m·K) acc. to EN 12524

[2] Insulation with thermal conductivity 0.04 W/(m·K)

Table B.7 Thermal conductivity of the materials used in drywalling

Building material	Thermal conductivity in W/(m•K)	Thickness in mm	Thermal resistance in m²•K/W
Gypsum board	0.25	12.5	0.05
		15	0.06
		20	0.08
		25	0.10
Gypsum fiber board Vidiwall	0.30	12.5	0.04
		15	0.05
Gypsum fiber board Brio	0.38	18	0.05
		23	0.06
Cement board AQUAPANEL® Outdoor	0.32	12.5	0.04
Cement board AQUAPANEL® Indoor	0.36	12.5	0.03
Cement board AQUAPANEL® Floor	0.79	22	0.03
Insulation material WLG 040	0.04	40	1.00
		60	1.50
		80	2.00
Insulation material WLG 035	0.035	40	1.00
		60	1.50
		80	2.00
Layer of air, static	-	10	0.15
		15	0.17
		20	0.17
		≥ 25	0.18

A seamless encasing of the building is also of utmost importance in terms of thermal insulation.
Non-insulated areas, even if small, will bring about a loss of energy and damage to the building on account of condensation.

In areas where earthquake resistance is a concern, it is important to choose insulation materials that weigh as little as possible but offer utmost insulation. The extra weight from insulation adds additional stress to drywall constructions in the event of an earthquake and must be taken into consideration in the dimensioning. This particularly applies to ceiling constructions.

B5 Protection Against Moisture

When using drywall systems in areas of considerable moisture or even wetness, it is important to remember that gypsum building materials are especially sensitive to moisture and therefore some special features must be observed for these application areas.

There is a defined classification for various degrees of moisture in rooms:

- Dry areas in which the air humidity is on a level which is common for living or office areas.
- Moderately damp areas in which the relative air humidity lies below 70% (daily average), whereby the maximum air humidity values are permissible for restricted periods of time, as shown in Table B.8.

Table B.8 Maximum moisture intensities in moderately damp areas

Maximum relative air humidity	Maximum period of occurrence in hours per day	Exemplary use
85	8	Nurseries, large kitchens
90	3	Laundries
95	2	Laboratories

- Damp areas with a consistently high relative air humidity > 70%

The application possibilities of the various types of board materials are unrestricted in dry areas.

However, in moderately damp areas, only impregnated gypsum boards (type GKBI or GKFI acc. to DIN 18180 or type H1/H2/H3 acc. to EN 520) can be used. Gypsum fiber boards can also be used here if they have the respective density.

In damp areas with more than 70 % consistent air humidity, only cement wallboards like AQUAPANEL® are permissible.

In addition to this, in wet areas like in showers and other splash zones, you must also provide a coat of sealant.

Figure B.5 Applying sealant to splash zones

A coating of sealant is generally recommended on the floors of damp areas; as this is where direct water can never be completely excluded and this area is usually subjected to the effects of water for a longer period of time. Water always flows to the lowest point and then remains there until it evaporates.

A further area of moisture protection is steam diffusion, especially in areas where different climatic conditions prevail on both sides of a component. Then it is important to avoid condensation from entering the moisture-sensitive construction, or to drain occurring condensation.

A typical case is external components, for example, roofs or external walls.

The vapor barrier membrane is a moisture-proof layer that is applied to the inside of an internal insulation at the sides of a room, in order to prevent the penetration of dampness to the insulation layer. The vapor barrier membrane serves to seal against warm air penetrating the insulation layer, which is where condensate would arise when the warm air cools down.

Gypsum boards with laminated aluminum foil serve simultaneously as the cladding of the drywall construction and also as the vapor barrier membrane. It is also here where you need to ensure that the joints are treated carefully, in order to guarantee the required tightness.

Protection against moisture in the external area (façade)

Cement wallboards as façade cladding, for example, AQUAPANEL®, do justice to the following moisture-protection requirements that are placed on a façade building material:

- Resistance to wetness and dimensional stability of the material
- Resistance to mould formation
- Water vapor permeability for an optimum room climate
- Weather-resistance

AQUAPANEL® Cement Board Outdoor is hence the ideal plaster baseboard for external applications, as numerous tests and trials have proven.

Cement wallboards are resistant to wet. Under water conditions, for example, the AQUAPANEL® Cement Board Outdoor shows an extremely low and technically harmless change in shape. The cement board changes neither its structural composition nor its structural properties.

Furthermore, AQUAPANEL® Cement Board Outdoor has a very good moisture vapor diffusion behavior, for a cement-based board, with a diffusion resistance of $\mu = 19$. This ensures that the cladding does not have any steam-blocking properties, which is of great importance for an optimum layer configuration in terms of the building physics concepts of the building.

C Dimensioning Aids for Drywall Constructions

C1 Ceilings

Dimensioning of the ceilings is done on the basis of the dead weight of the cladding and insulation layers as well as additional loads, such as loads from earthquakes.
These loads convey a load class for which, depending on the ceiling construction, the geometrical conditions (spacing of the substructure, room widths, profile heights) must be complied with. These are specified in Chapters C1.1 to C1.3.
In dependence on the chosen cladding thickness and the earthquake parameters, the load class is recorded in diagram for Figure C.4. The ordinates (Y-axis values) established from the cladding thickness must be increased by the amount of additional loads from the insulation layer.
In the case of radiation protection ceilings, the cladding of which is made up of 12.5 mm thick gypsum boards, the load class must be established according to the diagram in Figure C.5 depending on the lead sheet lamination thickness.

With the established load class, the dimensioning tables for the respective ceiling construction can now be used to ascertain the geometry.
The following parameters are the result of the dimensioning:

- Ceiling linings with one or two substructure levels or hat-shaped channel or resilient channel
  a ... Spacing of the fixing elements of the substructure on the basic ceiling
  b ... Spacing of the furring battens / channels (maximum span of the cladding)

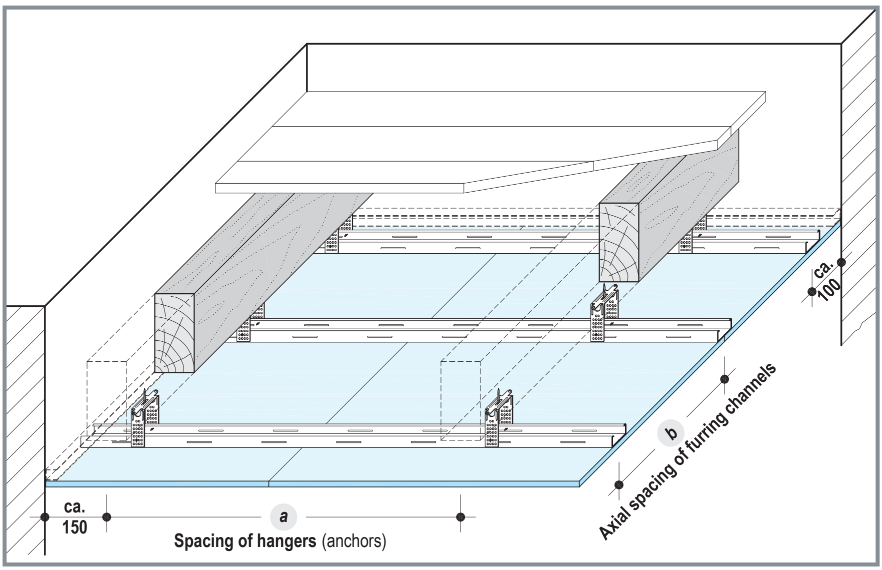


Figure C.1 Spacings of the substructure, consisting only of furring battens/channels

■ Suspended subceilings with two substructure levels (double grid)

a ... Spacing of the hangers

b ... Spacing of the furring battens/channels (maximum span of the cladding)

c ... Spacing of the carrying battens/channels

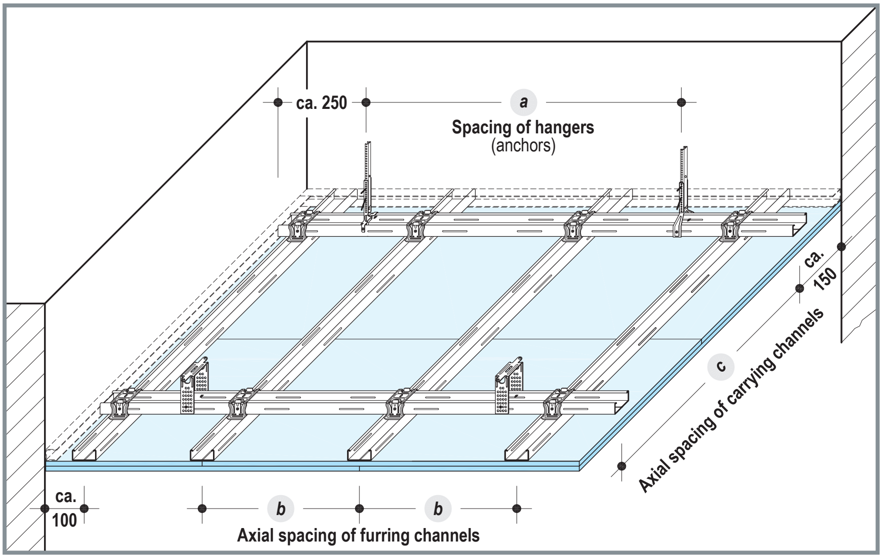


Figure C.2 Spacings of the substructure: double grid of battens/channels

■ Free-spanning subceilings: ***b*** ... Spacing of profiles (max. span of the cladding)

B ... Maximum room width (max. span of profiles)

h ... Required profile height

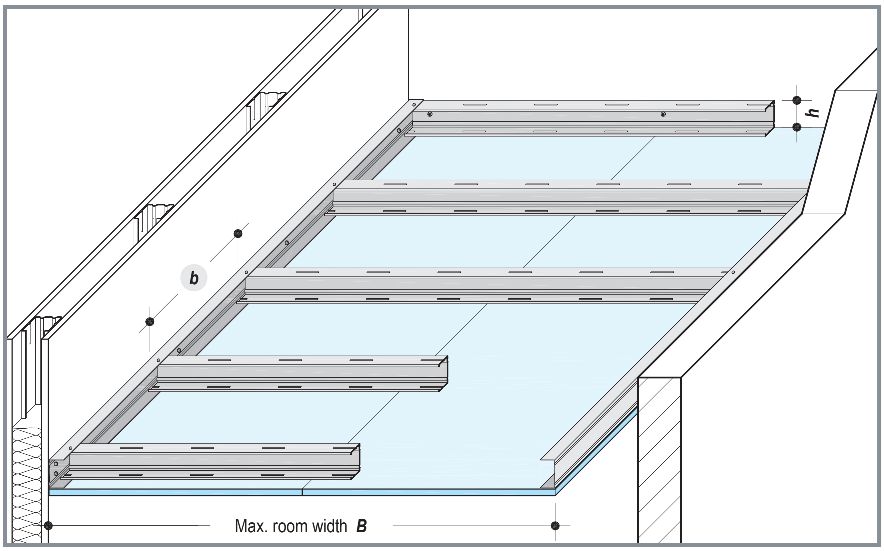


Figure C.3 Spacings of the substructure: free-spanning subceilings

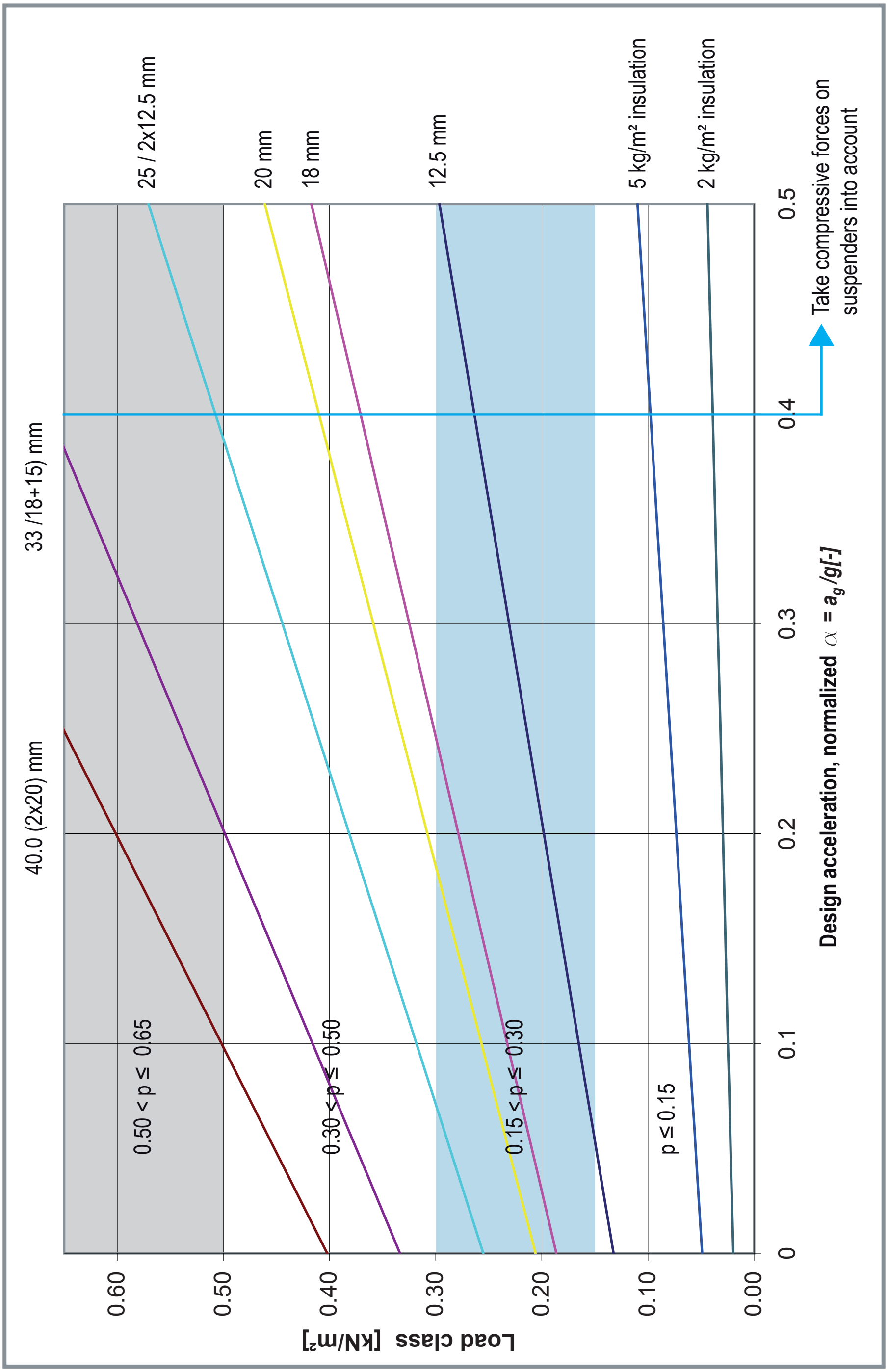


Figure C.4 Determination of the load class of ceiling linings and subceilings with gypsum board cladding, in dependence on the board thickness and earthquake parameters

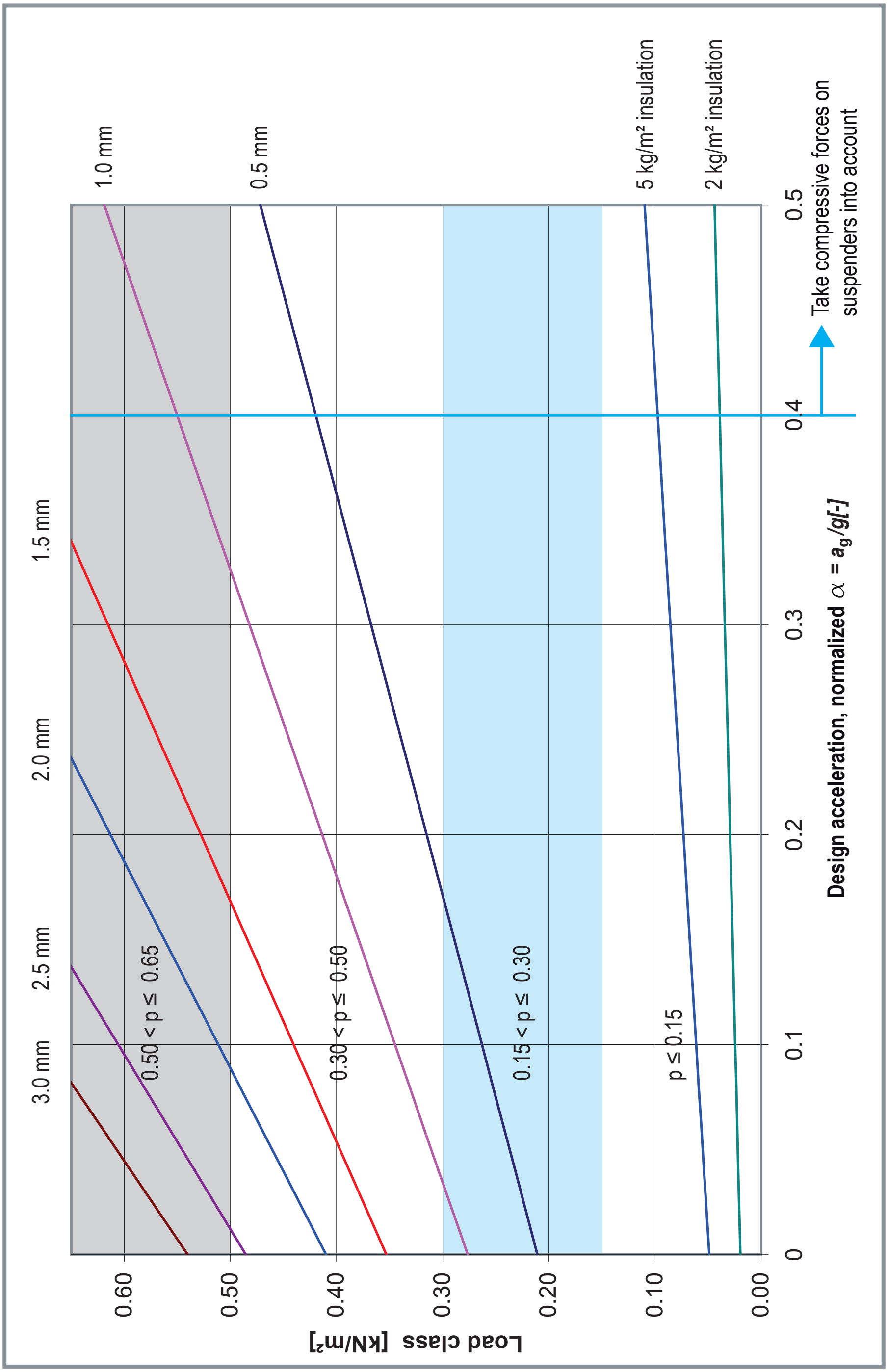


Figure C.5 *Determination of the load class of <u>radiation protection</u> ceiling linings and subceilings with a 12.5 mm gypsum board cladding, in dependence on the lead sheet thickness and earthquake parameters*

Example:

Drywall ceiling with 12.5 mm single-layer cladding made of gypsum boards

- Insulation layer: 40 mm mineral wool (40 kg/m³), weight 1.6 ≈ 2 kg/m²
- Construction: Suspended subceiling with substructure made of carrying and furring channels
- Public building: Importance factor acc. to EC 8 $\gamma_1 = 1.2$
- Seismic zone III (Greece):
  Design acceleration $a_g = a_{gR} \cdot \gamma_1 = 3.6\ \text{m/s}^2 \cdot 1.2 = 4.3\ \text{m/s}^2$
  $\alpha = a_g / g = 4.3\ \text{m/s}^2 / 10\ \text{m/s}^2 = 0.43$

1.) Determining the load class

- Reading:
  → Ordinate cladding: 0.27 kN/m²
  → Ordinate insulation: 0.04 kN/m²
- Assign to a load class: 0.27 kN/m² + 0.04 kN/m² = 0.31 kN/ m²
  → **Load class 0.30 < *p* ≤ 0.50 kN/m²**

2.) Determining the spacings of the substructure for the established load class

- Spacing of the furring channels acc. to Table C.1 for gypsum boards GKB:
  b **= 500 mm**
- Spacing of the carrying channels acc. to Table C.6:
  c **= 600 mm**
- Spacing of the hangers acc. to Table C.6 for load class up to 0.50 kN/m²:
  a **= 750 mm**

Information on dropped ceilings

As the spacing of the channels for dropped ceilings is not variable on account of the grid dimensions of the insert panels, for various loads the variables to be dimensioned are the load bearing capacity of the profiles (by choosing the cross-section) as well as the spacing of the hangers. As the panels are inserted loose, these ceilings should be used only in areas of low to medium earthquake risk and each panel must be secured by means of two brackets. The greatest problem in the case of considerable earthquake loads are the vertical loads against the dead weight, which could cause the cassette elements to lift and fall down and the virtually non-existent diaphragm action for the transfer of the horizontal forces.

For this reason, at this point no specific information is given; it is recommended to contact the respective system provider.

C1.1 Ceiling Linings and Suspended Subceilings

The permissible maximum span of the cladding *b* is independent of the load class and type of construction. It is established, in accordance with the following table, from the cladding thickness as well as the fire protection requirements.

Table C.1 Maximum span of cladding in accordance with DIN 18181

Board thickness / type of board in mm		Maximum axial spacing of furring battens/channels *b* in mm	
		Without fire protection lateral cladding	**With fire protection**
12.5 / 2x12.5	GKB / GKF	500	Lateral cladding, spacing in dependence on the fire resistance class, refer to the documentation of the system provider
15	GKB / GKF	550	
18	GKB / GKF	625	
20	GKB / GKF	625	
25	GKB / GKF	800	
12.5 / 15	Apertura boards (perforated boards)	333 or depending on perforation pattern	-
12.5 / 15 / 20 / 25 / 30	Fireboard	-	Longitudinal or lateral cladding, spacing in dependence on the fire resistance class, refer to the documentation of the system provider

All the other parameters are in dependence on the type of construction as well as the spacings of the furring channels, which are established in accordance with the table above or as defined by the fire protection requirements.

In the case of suspended subceilings the spacings of the anchors elements or hangers and spacings of the carrying channels can be varied in certain limits. When determining this the workmanship involved in erecting the construction should be taken into consideration.

Table C.2 Spacings of the substructure for ceiling lining with a wooden substructure, made up solely of furring battens

Axial spacing *b* in mm – furring battens 50x30	**Spacings of hangers / anchors *a* in mm Load class in kN/m²**		
	Up to 0.15	**Up to 0.30**	**Up to 0.50**
≤ 500	1200	950	800
625	-	900	750
800	-	800	700

Table C.3 Spacings of the substructure for ceiling lining with a metal substructure, made up solely of furring channels CD60x27

Axial spacing *b* in mm – furring channels CD 60x27	**Spacings of hangers / anchors *a* in mm Load class in kN/m²**			
Metal gauge 0.6 mm	**Up to 0.15**	**Up to 0.30**	**Up to 0.50**	**Up to 0.65**
≤ 500	1500	1200	1000	750
625	-	1100	800	600
800	-	1000	600	-

Table C.4 Spacings of the substructure for ceiling lining with a metal substructure, made up solely of furring channels F47

Axial spacing *b* in mm – furring channels F47	**Spacings of hangers / anchors *a* in mm Load class in kN/m²**			
Metal gauge 0.6 mm	**Up to 0.15**	**Up to 0.30**	**Up to 0.50**	**Up to 0.65**
≤ 500	1250	1000	850	750
600	1100	950	800	650
800	1000	850	650	500

Table C.5 Spacings of the substructure for ceiling lining with a metal substructure, made up of resilient channels/hat-shaped channels

Axial spacing *b* in mm – resilient channels / hat-shaped channels	**Spacings of anchors *a* in mm Load class in kN/m²**	
Metal gauge 0.6 mm	**Up to 0.15**	**Up to 0.30**
≤ 500	1200	950
625	-	900
800	-	800

Table C.6 Spacings of the substructure for subceilings with a wooden substructure, made up of carrying battens and furring battens as a double grid

Axial spacing *c* in mm – carrying batten 50x30	**Spacings of hangers / anchors *a* in mm Load class in kN/m²**		
	Up to 0.15	**Up to 0.30**	**Up to 0.50**
≤ 500	1200	950	800
600	1150	900	750
700	1050	850	700 [1)]
800	1050	800	-
900	1000	800 [1)]	-
1000	950	-	-
1100	900	-	-
1200	900	-	-

[1)] Does not apply to spacing of furring batten *b* = 800 mm

Table C.7 Spacings of the substructure for subceilings with a metal substructure, made up of carrying channels and furring channels CD60x27 as a double grid

Axial spacing *c* in mm – carrying channel CD 60x27	**Spacings of hangers / anchors *a* in mm Load class in kN/m²**			
Metal gauge 0.6 mm	**Up to 0.15**	**Up to 0.30**	**Up to 0.50**	**Up to 0.65**
≤ 500	1200	950	800	750
600	1150	900	750	600
700	1100	850	700 [2)]	550
800	1050	800	600 [2)]	-
900	1000	800	-	-
1000	950	750	-	-
1100	900	750 [2)]	-	-
1200	900	-	-	-

[2)] Does not apply to spacing of furring channel *b* = 800 mm

Table C.8 Spacings of the substructure for subceilings with a metal substructure, made up of carrying channels and furring channels F47 as a double grid

Axial spacings *c* in mm – carrying channel F47 Metal gauge 0.6 mm	**Spacings of hangers / anchors *a* in mm** **Load class in kN/m²**			
	Up to 0.15	**Up to 0.30**	**Up to 0.50**	**Up to 0.65**
≤ 500	850	650	550	500
600	800	600	500	500
700	750	600	500	450
800	700	550	450	-
900	700	550	-	-
1000	650	-	-	-

C1.2 Free-spanning subceilings

Table C.9 Permissible room widths for free-spanning subceilings, axial spacing of free-spanning profiles 500 mm

Free-spanning profile	**Maximum room width *B* in m** **Load class in kN/m²**			
	Up to 0.15	**Up to 0.30**	**Up to 0.50**	**Up to 0.65**
Knauf CW 50, Metal gauge 0.6 mm	2.5 [a)]	2 [a)]	1.75 [a)]	1.5 [a)]
Knauf CW 75, Metal gauge 0.6 mm	3.25 [a)]	2.5 [a)]	2.25 [a)]	2 [b)]
Knauf CW 100, Metal gauge 0.6 mm	3.5 [a)]	3 [a)]	2.5 [b)]	2.5 [c)]
Knauf CW 125, Metal gauge 0.6 mm	4 [a)]	3.5 [a)]	3 [c)]	2.75 [c)]
Knauf CW 150, Metal gauge 0.6 mm	4.5 [a)]	3.75 [a)]	3.25 [c)]	3 [d)]
2x Knauf CW 50, Metal gauge 0.6 mm	3 [a)]	2.5 [a)]	2.25 [a)]	2 [b)]
2x Knauf CW 75, Metal gauge 0.6 mm	3.5 [a)]	3 [a)]	2.75 [b)]	2.5 [c)]
2x Knauf CW 100, Metal gauge 0.6 mm	4 [a)]	3.5 [a)]	3.25 [c)]	3 [d)]
2x Knauf CW 125, Metal gauge 0.6 mm	4.5 [a)]	4 [b)]	3.5 [c)]	3.5 [d)]
2x Knauf CW 150, Metal gauge 0.6 mm	5 [a)]	4.5 [b)]	4 [d)]	3.75 [d)]
2x Knauf UA 50, Metal gauge 2 mm	3.25 [a)]	3 [a)]	2.5 [b)]	2.5 [c)]
2x Knauf UA 75, Metal gauge 2 mm	4 [a)]	3.75 [b)]	3.25 [c)]	3 [d)]
2x Knauf UA 100, Metal gauge 2 mm	4.75 [b)]	4.25 [c)]	4 [c)]	3.5 [d)]
2x Knauf UA 125, Metal gauge 2 mm	5.5 [c)]	4.75 [c)]	4.25 [d)]	3.75 [d)]
2x Knauf UA 150, Metal gauge 2 mm	6 [c)]	5.5 [d)]	4.25 [d)]	3.75 [d)]

[a)] Spacing of anchors on the perimeter profile: 62.5 or 60 cm
[b)] Spacing of anchors on the perimeter profile: 50 cm
[c)] Spacing of anchors on the perimeter profile: 40 cm
[d)] Spacing of anchors on the perimeter profile: 30 cm

Table C.10 Permissible room widths for free-spanning subceilings, axial spacing of free-spanning profiles 625 or 600 mm

Free-spanning profile	**Maximum room width *B* in m Load class in kN/m²**			
	Up to 0.15	**Up to 0.30**	**Up to 0.50**	**Up to 0.65**
Knauf CW 50, Metal gauge 0.6 mm	2,25 [a)]	2 [a)]	1,75 [a)]	1,5 [a)]
Knauf CW 75, Metal gauge 0.6 mm	3 [a)]	2,5 [a)]	2 [a)]	1,75 [a)]
Knauf CW 100, Metal gauge 0.6 mm	3,5 [a)]	2,75 [a)]	2,25 [a)]	2,25 [b)]
Knauf CW 125, Metal gauge 0.6 mm	4 [a)]	3,25 [a)]	2,75 [b)]	2,5 [c)]
Knauf CW 150, Metal gauge 0.6 mm	4,25 [a)]	3,5 [a)]	3 [b)]	2,75 [c)]
2x Knauf CW 50, Metal gauge 0.6 mm	2,75 [a)]	2,25 [a)]	2 [a)]	1,75 [a)]
2x Knauf CW 75, Metal gauge 0.6 mm	3,5 [a)]	3 [a)]	2,5 [b)]	2,25 [b)]
2x Knauf CW 100, Metal gauge 0.6 mm	4 [a)]	3,5 [a)]	3 [b)]	2,75 [c)]
2x Knauf CW 125, Metal gauge 0.6 mm	4,5 [a)]	4 [b)]	3,5 [c)]	3,25 [d)]
2x Knauf CW 150, Metal gauge 0.6 mm	5 [a)]	4,25 [b)]	3,75 [c)]	3,5 [d)]
2x Knauf UA 50, Metal gauge 2 mm	3,25 [a)]	2,75 [a)]	2,5 [b)]	2,25 [c)]
2x Knauf UA 75, Metal gauge 2 mm	4 [a)]	3,5 [b)]	3,25 [c)]	3 [d)]
2x Knauf UA 100, Metal gauge 2 mm	4,75 [a)]	4,25 [c)]	3,75 [d)]	3,5 [d)]
2x Knauf UA 125, Metal gauge 2 mm	5,25 [b)]	4,75 [c)]	4 [d)]	3,5 [d)]
2x Knauf UA 150, Metal gauge 2 mm	6 [b)]	5,25 [d)]	4 [d)]	3,5 [d)]

[a)] Spacing of anchors on the perimeter profile: 62.5 or 60 cm
[b)] Spacing of anchors on the perimeter profile: 50 cm
[c)] Spacing of anchors on the perimeter profile: 40 cm
[d)] Spacing of anchors on the perimeter profile: 30 cm

C.1.4 Acoustic Ceilings

Table C.11 Spacings of the substructure for acoustic subceilings with metal substructure, made up of carrying channels and furring channels CD60x27 and cladding consisting of apertura boards

Axial spacing *c* in mm – carrying channel CD 60x27 Metal gauge 0.6 mm	**Spacings of hangers / anchors *a* in mm** **Load class in kN/m²** **Up to 0.15**	**Up to 0.30**	**Up to 0.50**
≤ 500	1200	950	800
600	1150	900	750
700	1100	850	700
800	1050	800	600
900	1000	800	-
1000	950	750	-
1100	900	750	-
1200	900	650	-
1300	850	-	-
1400	850	-	-
1500	850	-	-

C2 Partitions

C2.1 Bracket Loads

The following threshold values and conditions apply to fixing loads on drywall partitions. There are dowel loads, which mainly have a local effect on the load introduction point, and bracket loads, which transfer across the entire wall structure. The method for fixing loads should be chosen in dependence on the extent of the load:
Light objects, for example pictures, as single loads up to 15 kg, can be fixed by means of hooks.

Figure C.6 Dowels for fixing single loads up to 15 kg

Higher loads, totaling up to 0.7 kN per m wall length, are fixed using dowels.

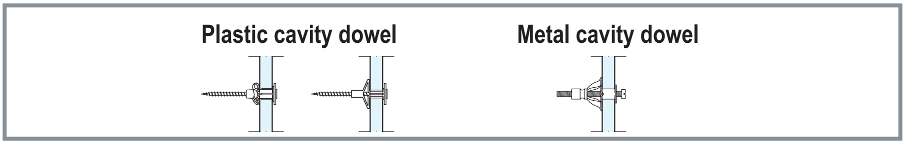


Figure C.7 Dowels for fixing bracket loads up to 0.7 kN/m

The fixing distance of the dowels must amount to at least 75 mm.
To fix bracket loads, at least two plastic or metal cavity dowels must be used, e. g. Tox Universal, Fischer Universal, Molly Scre Anchor.
In addition to the restrictions of the maximum overall load per m wall length, the following maximum values of load for the individual dowels must be complied with.

Table C.12 Dowel load bearing capacity – tensile load and shear load

Cladding thickness	**Plastic cavity dowel**	**Metal cavity dowel**
	ø 8 or ø 10 mm	Screw M5 or M6
mm	kg	kg
12.5	25	30
20	35	40
≥ 2x12.5	40	50

Drywall partitions can be stressed at any point by bracket loads up to 0.7 kN/m wall length, whilst taking into consideration the lever arm length (cabinet height ≥ 30 cm) and eccentricity (cabinet depth ≤ 60 cm).

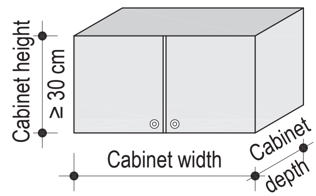
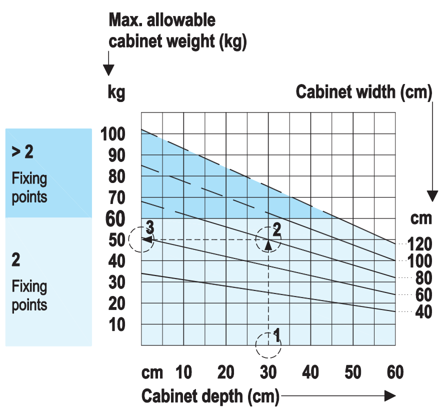


Figure C.8 Cabinet dimensions for establishing the permissible cabinet weight

Max. bracket loads up to 0.4 kN per m wall length

Cladding:
- 12.5 mm
- 15 mm



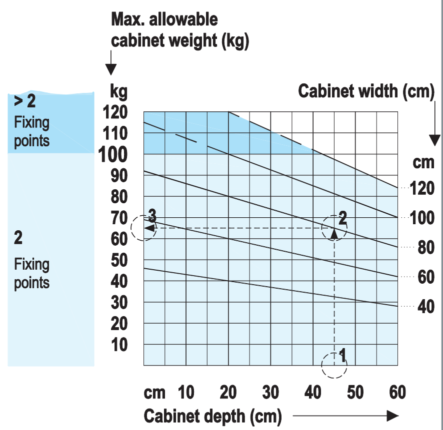
Example: Cabinet depth 30 cm, cabinet width 80 cm

In diagram for cabinet depth 30 cm (1) go vertically upwards, up to line for cabinet width 80 cm (2), go from intersection point horizontally to the left and read off maximum cabinet weight (3).

The max. allowable cabinet weight amounts to 50 kg for this example.

Max. bracket loads up to 0.7 kN per m wall length

Cladding:
- 18 mm / 25 mm / 2 x 12.5 mm / 2 x 15 mm / 2 x 18 mm
- 3 x 12.5 mm



Example: Cabinet depth 45 cm, cabinet width 80 cm

In diagram for cabinet depth 45 cm (1) go vertically upwards, up to line for cabinet width 80 cm (2), go from intersection point horizontally to the left and read off maximum cabinet weight (3).

The max. allowable cabinet weight amounts to 65 kg for this example.

Figure C.9 Establishing the permissible cabinet weight

If the load that is to be fixed exceeds 0.7 kN per m wall length, loads up to 1.5 kN/m wall length can be transferred via sanistands or traverses that are integrated in the drywall partition.

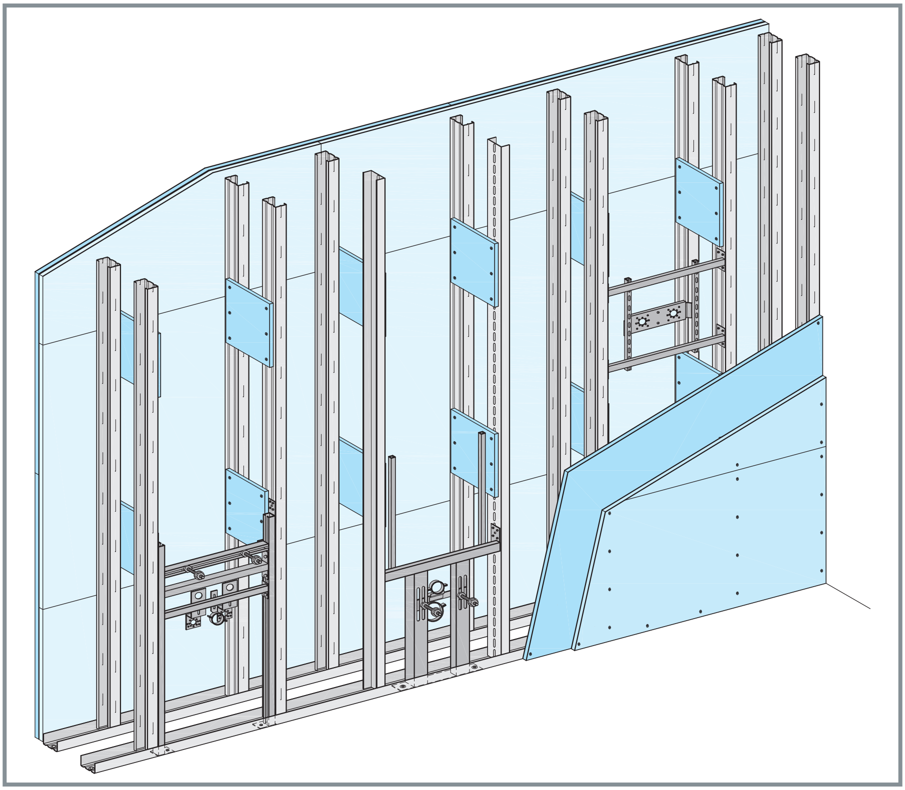


Figure C.10 Sanistands and traverses

C2.2 Permissible Wall Heights

C2.2.1 Metal Stud Partitions

*Table C.13 Permissible wall heights of metal stud partitions **W111** with single stud frame, spacing of studs **a = 30 or 31.25 cm**, single-layer cladding with **12.5 mm** gypsum boards in dependence on the seismic load*

<table>
<tr><th>$S \cdot a_g$</th><th colspan="8">Permissible wall heights in m</th></tr>
<tr><th>m/s²</th><th>CW50</th><th>CW75</th><th>CW100</th><th>M48</th><th>M62</th><th>M70</th><th>M90</th><th>M100</th></tr>
<tr><td colspan="9">Single studs Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 3.2</td><td>4.75</td><td rowspan="2">6.25</td><td rowspan="3">7.25</td><td rowspan="9">3</td><td rowspan="9">3.5</td><td rowspan="9">3.7</td><td rowspan="9">4.3</td><td rowspan="9">4.5</td></tr>
<tr><td>3.6</td><td>4.5</td></tr>
<tr><td>4.0</td><td rowspan="2">4.25</td><td>6</td></tr>
<tr><td>4.5</td><td>5.75</td><td>7.25 *</td></tr>
<tr><td>5.0</td><td>4</td><td>5.5</td><td>7 *</td></tr>
<tr><td>5.4</td><td rowspan="2">3.75</td><td rowspan="2">5.25</td><td>6.75 *</td></tr>
<tr><td>5.6</td><td>6.5 *</td></tr>
<tr><td>6.3</td><td rowspan="2">3.5</td><td>5</td><td>6.25 *</td></tr>
<tr><td>7.2</td><td>4.75</td><td>6 *</td></tr>
<tr><td colspan="9">Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 3.2</td><td rowspan="5">4.75</td><td>7.5</td><td rowspan="2">9</td><td rowspan="9">3.5</td><td rowspan="9">4.2</td><td rowspan="9">4.5</td><td rowspan="7">5.2</td><td rowspan="7">5.5</td></tr>
<tr><td>3.6</td><td>7.25</td></tr>
<tr><td>4.0</td><td>7</td><td>8.75 *</td></tr>
<tr><td>4.5</td><td>6.75</td><td>8.25 *</td></tr>
<tr><td>5.0</td><td>6.5</td><td>8 *</td></tr>
<tr><td>5.4</td><td rowspan="2">4.5</td><td rowspan="2">6.25 *</td><td rowspan="2">7.75 *</td></tr>
<tr><td>5.6</td></tr>
<tr><td>6.3</td><td>4.25</td><td>6 *</td><td>7.25 **</td><td rowspan="2">5.2 *</td><td rowspan="2">5.5 *</td></tr>
<tr><td>7.2</td><td>4</td><td>5.5 *</td><td>7 **</td></tr>
</table>

* Reduction of the anchoring distance of the circumferential perimeter profiles to 0.75 m

** Reduction of the anchoring distance of the circumferential perimeter profiles to 0.50 m

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

*Table C.14 Permissible wall heights of metal stud partitions **W112** with single stud frame, spacing of studs **a = 30 or 31.25 cm**, double-layer cladding with **2x12.5 mm** gypsum boards in dependence on the seismic load*

<table>
<tr><th>S • a_g</th><th colspan="8">Permissible wall heights in m</th></tr>
<tr><th>m/s²</th><th>CW50</th><th>CW75</th><th>CW100</th><th>M48</th><th>M62</th><th>M70</th><th>M90</th><th>M100</th></tr>
<tr><td colspan="9">Single studs Metal gauge 0.6 mm</td></tr>
<tr><td>0</td><td>5.5</td><td rowspan="3">7</td><td rowspan="2">8.5</td><td rowspan="8">3.6</td><td rowspan="6">4.2</td><td rowspan="6">4.5</td><td rowspan="4">5.3</td><td rowspan="4">5.5</td></tr>
<tr><td>1.8</td><td rowspan="2">5.25</td></tr>
<tr><td>2.3</td><td>8.5 *</td></tr>
<tr><td>2.7</td><td>4.75</td><td>6.5 *</td><td>8 *</td></tr>
<tr><td>3.2</td><td>4.5</td><td>6 *</td><td>7.5 **</td><td rowspan="3">5.3 *</td><td rowspan="2">5.5 *</td></tr>
<tr><td>3.6</td><td>4.25</td><td>5.75 *</td><td>7 **</td></tr>
<tr><td>4.0</td><td>4</td><td>5.5 *</td><td>6.75 **</td><td rowspan="3">4.2 *</td><td rowspan="2">4.5 *</td><td rowspan="4">5.5 **</td></tr>
<tr><td>4.5</td><td>3.75 *</td><td>5.25 **</td><td>6.5 **</td><td rowspan="4">5.3 **</td></tr>
<tr><td>5.0</td><td>3.5 *</td><td>5 **</td><td>6 **</td><td>3.4 *</td><td rowspan="2">4.5 **</td></tr>
<tr><td>5.4</td><td rowspan="2">3.5 *</td><td rowspan="2">4.75 **</td><td>5.5 **</td><td>3.3 *</td><td>4 *</td></tr>
<tr><td>5.6</td><td>5.25 **</td><td>3.2 *</td><td>4 **</td><td>4.4 **</td><td>5.4 *</td></tr>
<tr><td>6.3</td><td>3.25 *</td><td>4.5 **</td><td>4.75 **</td><td>3.1 *</td><td>3.8 **</td><td>4.2 **</td><td>4.8 **</td><td>4.8 **</td></tr>
<tr><td>7.2</td><td>3 *</td><td>4 **</td><td>4 **</td><td>2.9 *</td><td>3.5 **</td><td>3.9 **</td><td>4.2 **</td><td>4.2 **</td></tr>
<tr><td colspan="9">Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm</td></tr>
<tr><td>0</td><td rowspan="4">5.5</td><td>8.75</td><td>10</td><td rowspan="6">4.2</td><td rowspan="5">5</td><td rowspan="4">5.4</td><td rowspan="3">6.2</td><td rowspan="3">6.5</td></tr>
<tr><td>1.8</td><td>8.5</td><td>10 *</td></tr>
<tr><td>2.3</td><td>7.75 *</td><td>9.5 *</td></tr>
<tr><td>2.7</td><td>7.25 *</td><td>9 **</td><td rowspan="2">6.2 *</td><td rowspan="2">6.5 *</td></tr>
<tr><td>3.2</td><td>5</td><td>6.75 *</td><td>8.25 **</td><td rowspan="2">5.4 *</td></tr>
<tr><td>3.6</td><td>4.75 *</td><td>6.5 **</td><td>8 **</td><td rowspan="2">5 *</td><td rowspan="3">6.2 **</td><td rowspan="3">6.5 **</td></tr>
<tr><td>4.0</td><td>4.5 *</td><td>6.25 **</td><td>7.5 **</td><td>4.2 *</td><td rowspan="2">5.4 **</td></tr>
<tr><td>4.5</td><td>4.25 *</td><td>5.75 **</td><td>6.5 **</td><td>4 *</td><td>5 **</td></tr>
<tr><td>5.0</td><td rowspan="2">4 *</td><td rowspan="2">5.5 **</td><td>6 **</td><td>3.8 *</td><td>4.7 **</td><td>5.2 **</td><td>6 **</td><td>6 **</td></tr>
<tr><td>5.4</td><td>5.5 **</td><td rowspan="2">3.7 *</td><td>4.6 **</td><td>5.1 **</td><td>5.6 **</td><td>5.6 **</td></tr>
<tr><td>5.6</td><td>4 **</td><td>5.25 **</td><td>5.25 **</td><td>4.5 **</td><td>5 **</td><td>5.4 **</td><td>5.4 **</td></tr>
<tr><td>6.3</td><td>3.75 **</td><td>4.75 **</td><td>4.75 **</td><td>3.5 **</td><td>4.3 **</td><td>4.8 **</td><td>4.8 **</td><td>4.8 **</td></tr>
<tr><td>7.2</td><td>3.5 *</td><td>4 **</td><td>4 **</td><td>3.3 **</td><td>4 **</td><td>4.2 **</td><td>4.2 **</td><td>4.2 **</td></tr>
</table>

* Reduction of the anchoring distance of the circumferential perimeter profiles to 0.75 m

** Reduction of the anchoring distance of the circumferential perimeter profiles to 0.50 m

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

Table C.15 Permissible wall heights of metal stud partitions ***W111*** *with single stud frame, spacing of studs* ***a = 40 or 41.7 cm****, single-layer cladding with* ***12.5 mm*** *gypsum boards in dependence on the seismic load*

<table>
<tr><th>$S \cdot a_g$</th><th colspan="8">Permissible wall heights in m</th></tr>
<tr><th>m/s²</th><th>CW50</th><th>CW75</th><th>CW100</th><th>M48</th><th>M62</th><th>M70</th><th>M90</th><th>M100</th></tr>
<tr><td colspan="9">Single studs Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 4.5</td><td>3.75</td><td>5.25</td><td rowspan="4">5.75</td><td rowspan="6">2.8</td><td rowspan="6">3.3</td><td rowspan="6">3.6</td><td rowspan="6">4.1</td><td rowspan="6">4.3</td></tr>
<tr><td>5.0</td><td rowspan="3">3.5</td><td>5</td></tr>
<tr><td>5.4</td><td rowspan="2">4.75</td></tr>
<tr><td>5.6</td></tr>
<tr><td>6.3</td><td>3.25</td><td>4.5</td><td>5.75 *</td></tr>
<tr><td>7.2</td><td>3</td><td>4.25</td><td>5.5 *</td></tr>
<tr><td colspan="9">Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 3.2</td><td rowspan="9">3.75</td><td>7</td><td>8.5</td><td rowspan="9">3.3</td><td rowspan="9">3.9</td><td rowspan="9">4.2</td><td rowspan="8">4.8</td><td rowspan="8">5.1</td></tr>
<tr><td>3.6</td><td>6.75</td><td>8.25</td></tr>
<tr><td>4.0</td><td>6.5</td><td>8</td></tr>
<tr><td>4.5</td><td>6</td><td>7.75 *</td></tr>
<tr><td>5.0</td><td rowspan="2">5.75</td><td>7.5 *</td></tr>
<tr><td>5.4</td><td>7.25 *</td></tr>
<tr><td>5.6</td><td>5.5</td><td>7 *</td></tr>
<tr><td>6.3</td><td>5.25 *</td><td>6.75 *</td></tr>
<tr><td>7.2</td><td>5 *</td><td>6.5 **</td><td>4.8 *</td><td>5.1 *</td></tr>
</table>

* Reduction of the anchoring distance of the circumferential perimeter profiles to 0.75 m

** Reduction of the anchoring distance of the circumferential perimeter profiles to 0.50 m

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

*Table C.16 Permissible wall heights of metal stud partitions **W112** with single stud frame, spacing of studs **a = 40 or 41.7 cm**, double-layer cladding with **2x12.5 mm** gypsum boards in dependence on the seismic load*

<table>
<tr><th>$S \cdot a_g$</th><th colspan="8">Permissible wall heights in m</th></tr>
<tr><th>m/s²</th><th>CW50</th><th>CW75</th><th>CW100</th><th>M48</th><th>M62</th><th>M70</th><th>M90</th><th>M100</th></tr>
<tr><td colspan="9">Single studs Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 1.8</td><td rowspan="2">4.5</td><td rowspan="3">6</td><td>7</td><td rowspan="6">3.3</td><td rowspan="6">4</td><td rowspan="5">4.2</td><td rowspan="4">4.9</td><td rowspan="3">5.1</td></tr>
<tr><td>2.3</td><td rowspan="3">7 *</td></tr>
<tr><td>2.7</td><td>4.25</td></tr>
<tr><td>3.2</td><td>4</td><td>5.5 *</td><td rowspan="3">5.1 *</td></tr>
<tr><td>3.6</td><td>3.75</td><td>5.25 *</td><td>6.5 **</td><td rowspan="2">4.9 *</td></tr>
<tr><td>4.0</td><td rowspan="2">3.5</td><td>5 *</td><td>6.25 **</td><td rowspan="2">4.2 *</td></tr>
<tr><td>4.5</td><td>4.75 *</td><td rowspan="2">6 **</td><td>3.2</td><td>4 *</td><td rowspan="3">4.9 **</td><td rowspan="4">5.1 **</td></tr>
<tr><td>5.0</td><td>3.25</td><td>4.5 *</td><td>3.1</td><td>3.8 *</td><td>4.1 *</td></tr>
<tr><td>5.4</td><td>3.25 *</td><td rowspan="2">4.25 **</td><td>5.5 **</td><td>3</td><td>3.6 *</td><td>4 *</td></tr>
<tr><td>5.6</td><td rowspan="2">3 *</td><td>5.25 **</td><td>2.9</td><td>3.5 *</td><td>4 **</td><td>4.8 **</td></tr>
<tr><td>6.3</td><td>4 **</td><td>4.75 **</td><td>2.7 *</td><td>3.4 *</td><td>3.7 **</td><td>4.6 **</td><td>4.8 **</td></tr>
<tr><td>7.2</td><td>2.75 *</td><td>3.75 **</td><td>4 **</td><td>2.6 *</td><td>3.1 **</td><td>3.5 **</td><td>4.2 **</td><td>4.2 **</td></tr>
<tr><td colspan="9">Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm</td></tr>
<tr><td>0</td><td rowspan="3">5</td><td>8.25</td><td>9.5</td><td rowspan="6">3.9</td><td rowspan="5">4.7</td><td rowspan="5">5</td><td rowspan="3">5.8</td><td rowspan="3">6.1</td></tr>
<tr><td>1.8</td><td>8</td><td>9.5 *</td></tr>
<tr><td>2.3</td><td>7</td><td>8.75*</td></tr>
<tr><td>2.7</td><td>4.75</td><td>6.5 *</td><td>8.25 *</td><td rowspan="3">5.8 *</td><td rowspan="2">6.1 *</td></tr>
<tr><td>3.2</td><td>4.5</td><td>6.25 *</td><td>7.75 **</td></tr>
<tr><td>3.6</td><td>4.25</td><td>5.75 *</td><td>7.25 **</td><td rowspan="2">4.7 *</td><td rowspan="2">5 *</td><td rowspan="3">6.1 **</td></tr>
<tr><td>4.0</td><td>4</td><td>5.5 *</td><td>7 **</td><td>3.8</td><td rowspan="3">5.8 **</td></tr>
<tr><td>4.5</td><td>4 *</td><td>5.25 **</td><td>6.5 **</td><td>3.6</td><td>4.5 *</td><td>4.9 **</td></tr>
<tr><td>5.0</td><td>3.75 *</td><td>5 **</td><td>6 **</td><td>3.5 *</td><td>4.3 *</td><td>4.7 **</td><td>6 **</td></tr>
<tr><td>5.4</td><td rowspan="2">3.5 *</td><td>5 **</td><td>5.5 **</td><td>3.4 *</td><td rowspan="2">4.1 **</td><td>4.6 **</td><td>5.6 **</td><td>5.6 **</td></tr>
<tr><td>5.6</td><td>4.75 **</td><td>5.25 **</td><td>3.3 *</td><td>4.5 **</td><td>5.4 **</td><td>5.4 **</td></tr>
<tr><td>6.3</td><td>3.25 *</td><td>4.5 **</td><td>4.75 **</td><td>3.1 *</td><td>3.9 **</td><td>4.3 **</td><td>4.8 **</td><td>4.8 **</td></tr>
<tr><td>7.2</td><td>3.25 **</td><td>4 **</td><td>4 **</td><td>3 **</td><td>3.6 **</td><td>4 **</td><td>4.2 **</td><td>4.2 **</td></tr>
</table>

* Reduction of the anchoring distance of the circumferential perimeter profiles to 0.75 m

** Reduction of the anchoring distance of the circumferential perimeter profiles to 0.50 m

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

*Table C.17 Permissible wall heights of metal stud partitions **W111** with single stud frame, spacing of studs **a = 60 or 62.5 cm**, single-layer cladding with **12.5 mm** gypsum boards in dependence on the seismic load*

<table>
<tr><th>S • a_g</th><th colspan="8">Permissible wall heights in m</th></tr>
<tr><th>m/s²</th><th>CW50</th><th>CW75</th><th>CW100</th><th>M48</th><th>M62</th><th>M70</th><th>M90</th><th>M100</th></tr>
<tr><td colspan="9">Single studs Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 6.3</td><td>2.75</td><td rowspan="2">3.75</td><td>4.25</td><td>2.6</td><td rowspan="2">3</td><td rowspan="2">3.1</td><td rowspan="2">3.7</td><td rowspan="2">3.9</td></tr>
<tr><td>7.2</td><td>2.5</td><td>4.25*</td><td>2.4</td></tr>
<tr><td colspan="9">Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 5.0</td><td rowspan="5">2.75</td><td rowspan="3">5</td><td>6.5</td><td rowspan="4">3</td><td rowspan="5">3.5</td><td rowspan="5">3.7</td><td rowspan="5">4.3</td><td rowspan="5">4.5</td></tr>
<tr><td>5.4</td><td rowspan="2">6.25*</td></tr>
<tr><td>5.6</td></tr>
<tr><td>6.3</td><td>4.75</td><td>6*</td></tr>
<tr><td>7.2</td><td>4.5</td><td>5.75*</td><td>2.75</td></tr>
</table>

* Reduction of the anchoring distance of the circumferential perimeter profiles to 0.75 m

** Reduction of the anchoring distance of the circumferential perimeter profiles to 0.50 m

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

Table C.18 Permissible wall heights of metal stud partitions ***W112*** *with single stud frame, spacing of studs* ***a = 60 or 62.5 cm****, double-layer cladding with* ***2x12.5 mm*** *gypsum boards in dependence on the seismic load*

<table>
<tr><th rowspan="2">$S \cdot a_g$
m/s²</th><th colspan="8">Permissible wall heights in m</th></tr>
<tr><th>CW50</th><th>CW75</th><th>CW100</th><th>M48</th><th>M62</th><th>M70</th><th>M90</th><th>M100</th></tr>
<tr><td colspan="9">Single studs Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 2.3</td><td rowspan="3">3.5</td><td rowspan="2">5</td><td>5.75</td><td rowspan="4">3</td><td rowspan="5">3.6</td><td rowspan="5">3.8</td><td rowspan="3">4.4</td><td rowspan="3">4.6</td></tr>
<tr><td>2.7</td><td>5.75*</td></tr>
<tr><td>3.2</td><td>4.75</td><td rowspan="2">5.75**</td></tr>
<tr><td>3.6</td><td>3.25</td><td>4.5*</td><td rowspan="3">4.4 *</td><td rowspan="3">4.6 *</td></tr>
<tr><td>4.0</td><td rowspan="2">3</td><td>4.25*</td><td>5.5**</td><td>2.9</td></tr>
<tr><td>4.5</td><td>4*</td><td>5**</td><td>2.8</td><td>3.4</td><td>3.7 *</td></tr>
<tr><td>5.0</td><td rowspan="2">2.75</td><td rowspan="2">3.75*</td><td rowspan="2">4.75**</td><td>2.7</td><td>3.2</td><td>3.5 *</td><td>4.3 **</td><td rowspan="2">4.6 **</td></tr>
<tr><td>5.4</td><td rowspan="2">2.5</td><td>3.1 *</td><td rowspan="2">3.4 *</td><td>4.2 **</td></tr>
<tr><td>5.6</td><td>2.75*</td><td>3.5*</td><td>4.5**</td><td>3 *</td><td>4.1 **</td><td>4.5 **</td></tr>
<tr><td>6.3</td><td>2.5*</td><td>3.25*</td><td rowspan="2">4**</td><td>2.4</td><td>2.9 *</td><td>3.2 *</td><td>3.9 **</td><td>4.3 **</td></tr>
<tr><td>7.2</td><td>2.5*</td><td>3.25**</td><td>2.3</td><td>2.7 *</td><td>2.9 *</td><td>3.6 **</td><td>3.8 **</td></tr>
<tr><td colspan="9">Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 1.8</td><td rowspan="6">3.5</td><td rowspan="2">6</td><td>7.5</td><td rowspan="4">3.6</td><td rowspan="5">4.2</td><td rowspan="5">4.5</td><td rowspan="3">5.2</td><td rowspan="3">5.5</td></tr>
<tr><td>2.3</td><td rowspan="2">7.5*</td></tr>
<tr><td>2.7</td><td>5.75</td></tr>
<tr><td>3.2</td><td>5.25*</td><td>6.75**</td><td rowspan="3">5.2*</td><td rowspan="2">5.5*</td></tr>
<tr><td>3.6</td><td>5*</td><td>6.5**</td><td rowspan="2">3.25</td></tr>
<tr><td>4.0</td><td>4.75*</td><td>6**</td><td>4</td><td>4.5*</td><td rowspan="2">5.5**</td></tr>
<tr><td>4.5</td><td rowspan="2">3.25</td><td>4.5*</td><td>5.75**</td><td rowspan="2">3</td><td>3.75*</td><td>4.25*</td><td>5.2**</td></tr>
<tr><td>5.0</td><td rowspan="3">4.25**</td><td>5.5**</td><td rowspan="3">3.5*</td><td>4*</td><td>5**</td><td rowspan="2">5.25**</td></tr>
<tr><td>5.4</td><td>3</td><td rowspan="2">5.25**</td><td rowspan="2">2.75</td><td rowspan="2">3.75*</td><td rowspan="2">4.75**</td></tr>
<tr><td>5.6</td><td>3*</td><td>5**</td></tr>
<tr><td>6.3</td><td rowspan="2">2.75*</td><td>4**</td><td>4.75**</td><td>2.5</td><td>3.25*</td><td>3.5**</td><td>4.5**</td><td>4.75**</td></tr>
<tr><td>7.2</td><td>3.75**</td><td>4**</td><td>2.5*</td><td>3**</td><td>3.25**</td><td>4**</td><td>4**</td></tr>
</table>

* Reduction of the anchoring distance of the circumferential perimeter profiles to 0.75 m

** Reduction of the anchoring distance of the circumferential perimeter profiles to 0.50 m

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

*Table C.19 Permissible wall heights of radiation protection walls with single stud frame, spacing of studs **a = 60 or 62.5 cm** with **3 mm lead sheet lamination per wall side** gypsum boards in dependence on the seismic load*

S • a$_g$	**Permissible wall heights in m**									
	Single-layer cladding, 1x12.5 mm					Double-layer cladding, 2x12.5 mm				
m/s²	CW50	CW75	CW100	2xCW75	2xCW100	CW50	CW75	CW100	2xCW75	2xCW100
0	2.75	3.75	4.25	4.25	5.5	3.5	5	5.75	6	7.25
1.8	2.75	3.75	4.25	4.5	5.75 *	3	4 *	5.25 **	4.75 *	5.75 *
2.3	2.5	3.5	4.25 *	4 *	5.25 **	2.5	3.5 *	4.5 **	4.25 **	5.25 **
2.7	2.3	3.25	4 *	3.75 *	4.75 **	2.5 *	3.25 **	4.25 **	4 **	4.5 **
3.2	-	3	3.5 **	3.5 *	4.5 **	2.3 *	3 **	3.75 **	3.5 **	3.75 **
3.6	-	2.75	3.25 **	3.5 **	4.25 **	-	2.75 **	3.25 **	3.25 **	3.25 **
4.0	-	2.5	3 **	3.25 **	3.75 **	-	2.5 **	3 **	3 **	3 **
4.5	-	2.3	2.75 **	3 **	3.5 **	-	2.3 **	2.75 **	2.75 **	2.75 **
5.0	-	-	2.75 **	3 **	3 **	-	2.3 **	2.5 **	2.5 **	2.5 **
5.4	-	-	2.5**	2.75 **	2.75 **	-	-	2.3 **	2.3 **	2.3 **
5.6	-	-	2.5 **	2.75 **	2.75 **	-	-	-	-	-
6.3	-	-	2.3 **	2.5 **	2.5 **	-	-	-	-	-

* Reduction of the anchoring distance of the circumferential perimeter profiles to 0.75 m

** Reduction of the anchoring distance of the circumferential perimeter profiles to 0.50 m

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

C2.2.2 Furrings and Installation Shaft Walls

The following permissible wall heights are valid for installation shaft walls and independent furrings made up of metal stud frames with single or double studs.

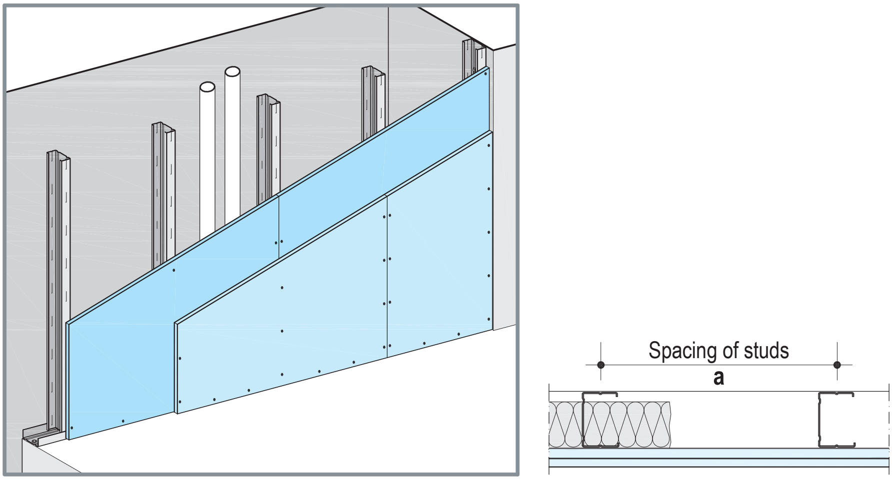


Figure C.11 Installation shaft wall with single studs

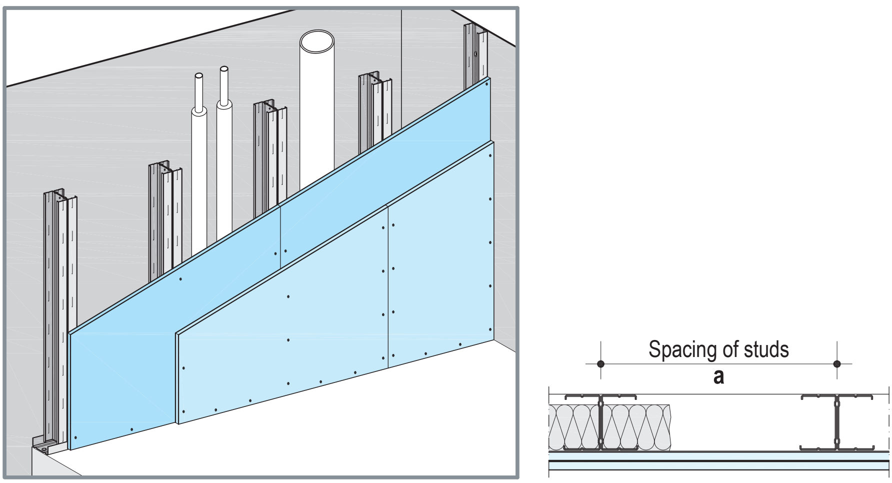


Figure C.12 Installation shaft wall with double studs

*Table C.20 Permissible wall heights of metal stud furrings and installation shaft walls, spacing of studs **a = 30 or 31.25 cm**, single-layer cladding with **1x12.5 mm** gypsum boards in dependence on the seismic load*

S • a_g m/s²	**Permissible wall heights in m** CW75	CW100
Single studs Metal gauge 0.6 mm		
0 - 7.2	3.5	4

*Table C.21 Permissible wall heights of metal stud furrings and installation shaft walls, spacing of studs **a = 40 or 41.7 cm**, single-layer cladding with **1x12.5 mm** gypsum boards in dependence on the seismic load*

S • a_g	**Permissible wall heights in m**						
m/s²	CW75	CW100	M48	M62	M70	M90	M100
Single studs Metal gauge 0.6 mm							
0 - 7.2	3	3.5	-	2.75	3	3.45	3.65
Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm							
0 - 7.2	4.5	5.5	2.75	3.25	3.55	4.1	4.3

*Table C.22 Permissible wall heights of metal stud furrings and installation shaft walls, spacing of studs **a = 60 or 62.5 cm**, single-layer cladding with **1x12.5 mm** gypsum boards in dependence on the seismic load*

S • a_g	**Permissible wall heights in m**						
m/s²	CW75	CW100	M48	M62	M70	M90	M100
Single studs Metal gauge 0.6 mm							
0 - 7.2	2.5	3	-	2.5	2.7	3.1	3.3
Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm							
0 - 7.2	3.5	5	2.5	2.95	3.2	3.7	3.9

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

Table C.23 Permissible wall heights of metal stud furrings and installation shaft walls, spacing of studs ***a = 30 or 31.25 cm****, double-layer cladding with* ***2x12.5 mm*** *gypsum boards in dependence on the seismic load*

***S • a**$_g$* m/s²	**Permissible wall heights in m** CW50	CW75	CW100
Single studs Metal gauge 0.6 mm			
0 - 6.3	-	4	4.5
7.2		3.75	
Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm			
0 - 3.2	4.5	6	7.5
3.6	4.25	5.75	7.25
4.0		5.5	7
4.5	4	5.25	6.75
5.0	3.75		6.5
5.4		5	6.25*
5.6			
6.3	3.5	4.75	6*
7.2	3.25	4.5	5.5*

* Reduction of the anchoring distance of the circumferential perimeter profiles to 0.75 m

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

*Table C.24 Permissible wall heights of metal stud furrings and installation shaft walls, spacing of studs **a = 40 or 41.7 cm**, double-layer cladding with **2x12.5 mm** gypsum boards in dependence on the seismic load*

<table>
<tr><th>$S \cdot a_g$</th><th colspan="7">Permissible wall heights in m</th></tr>
<tr><th>m/s²</th><th>CW75</th><th>CW100</th><th>M48</th><th>M62</th><th>M70</th><th>M90</th><th>M100</th></tr>
<tr><td colspan="8">Single studs Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 5.6</td><td rowspan="2">3.5</td><td rowspan="3">4</td><td>2.55</td><td rowspan="2">3.05</td><td rowspan="2">3.25</td><td rowspan="3">3.75</td><td rowspan="3">4</td></tr>
<tr><td>6.3</td><td rowspan="2">-</td></tr>
<tr><td>7.2</td><td>3.25</td><td>2.8</td><td>3</td></tr>
<tr><td colspan="8">Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 4.0</td><td>5</td><td rowspan="2">6</td><td rowspan="5">3.05</td><td rowspan="6">3.6</td><td rowspan="6">3.85</td><td rowspan="7">4.5</td><td rowspan="7">4.75</td></tr>
<tr><td>4.5</td><td rowspan="2">4.75</td></tr>
<tr><td>5.0</td><td rowspan="2">5.75</td></tr>
<tr><td>5.4</td><td rowspan="2">4.5</td></tr>
<tr><td>5.6</td><td>5.5</td></tr>
<tr><td>6.3</td><td>4.25</td><td>5.25*</td><td>2.9</td></tr>
<tr><td>7.2</td><td>4</td><td>5*</td><td>2.75</td><td>3.4</td><td>3.7</td></tr>
</table>

* Reduction of the anchoring distance of the circumferential perimeter profiles to 0.75 m

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

Table C.25 Permissible wall heights of metal stud furrings and installation shaft walls, spacing of studs ***a = 60 or 62.5 cm****, double-layer cladding with* ***2x12.5 mm*** *gypsum boards in dependence on the seismic load*

<table>
<tr><th rowspan="2">$S \cdot a_g$
m/s²</th><th colspan="8">Permissible wall heights in m</th></tr>
<tr><th>CW50</th><th>CW75</th><th>CW100</th><th>M48</th><th>M62</th><th>M70</th><th>M90</th><th>M100</th></tr>
<tr><td colspan="9">Single studs Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 5.0</td><td rowspan="5">-</td><td rowspan="3">3</td><td rowspan="5">3.25</td><td rowspan="5">-</td><td>2.75</td><td rowspan="2">2.95</td><td rowspan="4">3.4</td><td rowspan="5">3.6</td></tr>
<tr><td>5.4</td><td rowspan="4">2.5</td></tr>
<tr><td>5.6</td><td rowspan="2">2.8</td></tr>
<tr><td>6.3</td><td rowspan="2">2.75</td></tr>
<tr><td>7.2</td><td>2.6</td><td>3.2</td></tr>
<tr><td colspan="9">Double studs, each stud made of 2 profiles, connected on the web Metal gauge 0.6 mm</td></tr>
<tr><td>0 - 2.3</td><td rowspan="3">3.5</td><td rowspan="2">5</td><td>6.5</td><td rowspan="7">2.75</td><td rowspan="9">3.25</td><td rowspan="9">3.5</td><td rowspan="11">4.05</td><td rowspan="11">4.3</td></tr>
<tr><td>2.7</td><td>6.25</td></tr>
<tr><td>3.2</td><td>4.75</td><td>6</td></tr>
<tr><td>3.6</td><td rowspan="2">3.25</td><td>4.5</td><td>5.75</td></tr>
<tr><td>4.0</td><td rowspan="2">4.25</td><td>5.5</td></tr>
<tr><td>4.5</td><td rowspan="2">3</td><td>5.25</td></tr>
<tr><td>5.0</td><td>4</td><td>5</td></tr>
<tr><td>5.4</td><td rowspan="3">2.75</td><td rowspan="3">3.75</td><td rowspan="2">4.75</td><td rowspan="3">2.5</td></tr>
<tr><td>5.6</td></tr>
<tr><td>6.3</td><td>4.5</td><td>3</td><td rowspan="2">3.25</td></tr>
<tr><td>7.2</td><td>2.5</td><td>3.5</td><td>4.25</td><td>2.4</td><td>2.75</td></tr>
</table>

The standard anchoring distance of the circumferential perimeter profiles amounts to 1 m.

C2.2.3 Wood stud partitions

Table C.26 Permissible wall heights of wood stud partitions wih single- or double-layer cladding with ***12.5 mm*** *gypsum boards in dependence on the seismic load*

<table>
<tr><th rowspan="2">$S \cdot a_g$
m/s²</th><th colspan="2">Permissible wall heights in m, stud spacing a = 60 cm</th></tr>
<tr><th>Stud cross-section 60 x 60 mm</th><th>Stud cross-section 60 x 80 mm</th></tr>
<tr><td colspan="3">Single studs</td></tr>
<tr><td>0 - 7.2</td><td>3.1</td><td>4.1</td></tr>
<tr><td colspan="3">Double stud frame, studs arranged staggered</td></tr>
<tr><td>0 - 7.2</td><td>4.1</td><td>4.1</td></tr>
</table>

C2.2.4 Structural Wood Frame Wall Panels (Diaphragm Walls)

On account of their load-transferring function, wood frame walls panels are dimensioned by the structural engineer.
In doing so, e. g. DIN 1052:2004-08 serves as a basis.
However, it is common to base the dimensioning on load bearing values from constructional supervisory permissions, which are based on tests. As the bracing effect of wood frame wall panels primarily transfers horizontal loads, then loads from earthquakes are also transferred in this manner. This must be taken into account when establishing the loads.
In this book, the maximum permissible loads should be listed in accordance with authorizations. Refer to /4.9/ for more information on dimensioning in accordance with DIN 1052:2004-08.
If wood frame wall panels are to take loads in a vertical direction, then the load bearing structure must be designed in such a way that it does not cause major movement of the head point in laterals direction of the walls plane in the event of an earthquake, otherwise this can cause the structure to become off-center, and this is not taken into consideration in the establishment of the load bearing values in the constructional supervisory authorizations.
Bracing cladding must be continuous throughout the wall height without joints. A horizontal joint is only permissible if the cladding has been used exclusively to brace against the buckling of the ribs.
Figure C.13 shows the differences between bracing and non-bracing cladding..

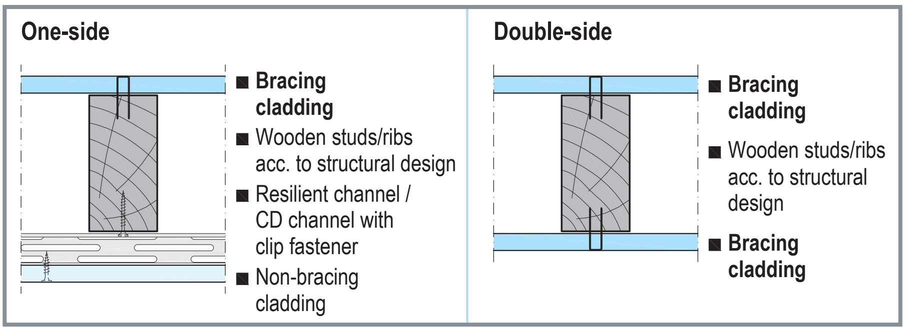
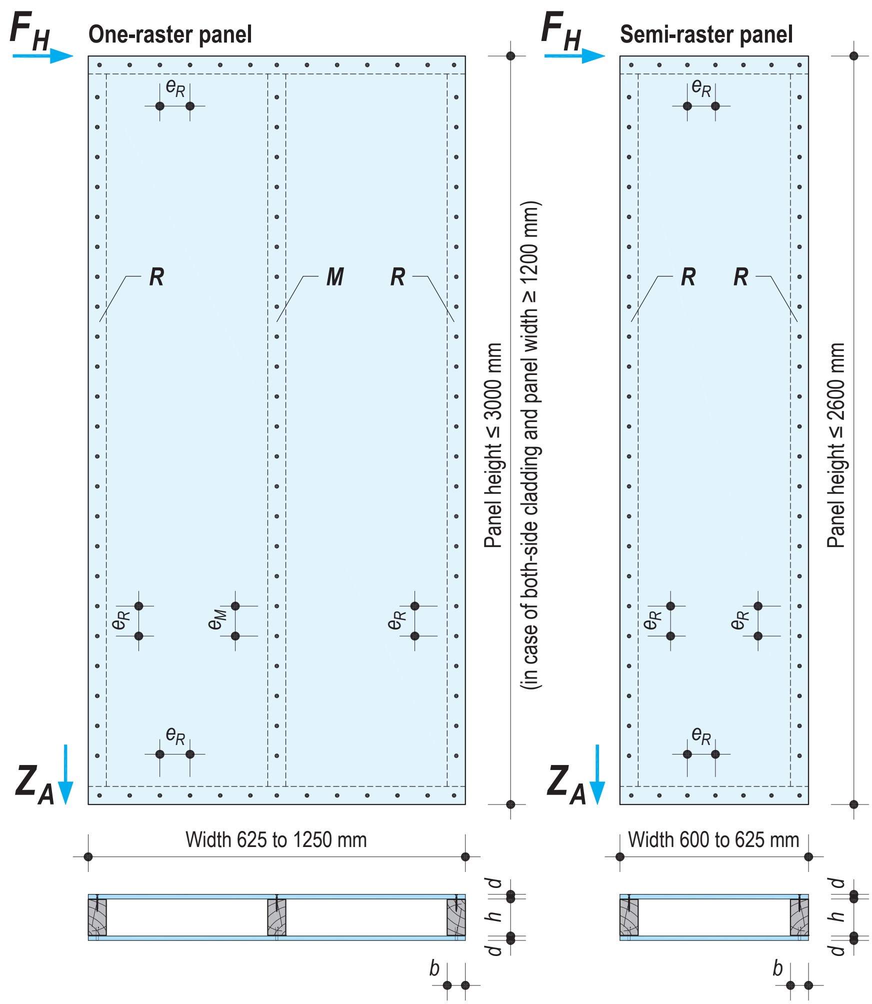


Figure C.13 Bracing and non-bracing cladding

The following Table C.27 shows the permissible horizontal loads for wood frame wall panels with bracing cladding made of gypsum or gypsum boards per panels width.



F_H	= Horizontal load	Z_A	= Anchorimg tension force
$e_{R/M}$	= Spacing of fasteners	b	= Width of rib
R	= Edge rib	h	= Depth of rib
M	= Middle rib	d	= Thickness of cladding

Figure C.14 Drawing for Table C.27

Table C.27 Permissible horizontal diaphragm load of wood frame wall panels in accordance with the general constructional supervisory permissionss Z-9.1-339 (Knauf gypsum fiber boards) and Z-9.1-199 (Knauf gypsum boards)

Cladding	**Raster width** Standard b_s **mm**	**Spacings for nails / staples e_R mm**	**Gypsum fiber boards *max.* F_H in kN for panel height h in m**		**Gypsum boards *max.* F_H[1] in kN for panel height h in m**	
			≤ 2.60	**≤ 3.00**	**≤ 2.60**	**≤ 3.00**
Double-side	600 to 625	min. 50			3.3	
		max. 75	3.3			
		max. 150			1.3	
	1200 to 1250	min. 50			6.0	5.5
		max. 75	7.5	6.3		
		max. 150			2.7	2.7
One-side	1200 to 1250	min. 50			3.3	
		max. 75	4.4	2.8		
		max. 150			1.5	

[1] For *max.* F_H the values e_R = 50 mm and 150 mm can be interpolated in a linear manner, the same applies to the values for h = 2.60 m and 3.0 m.

The values of the Table C.27 apply to cladding thickness = 12.5 mm.

The following information and specifications are excerpts from the constructional supervisory permissions that include additional information that must be observed.

Notes

The following reductions must be taken into account:

- In the case of on-site assembly: Reductions in the values for *max.* F_H according to Table C.27 by 20 %. If GKBI or GKFI boards are being used for the external cladding of external walls, a further reduction of 10 % applies.
- In the case of factory-produced external wall panels with external cladding of Knauf boards GKBI or GKFI: Reductions in the values for *max.* F_H according to Table C.27 by 10%.
- In the case of a rib cross-sectional area less than 40 cm² (only permissible with factory production): Reduction in the values for *max.* F_H by the ratio smallest rib cross-section/reference rib cross-section of 40 cm².
- In the case of one-raster panels with grid widths b_S less than 1200 mm: Reduction in the values for *max.* F_H according to Table C.27 by the ratio panel width b_s to the reference panel width of 1200 mm.

Wooden ribs

- Made of softwood, dry-sorted, strength class C24 as defined by DIN 1052: 2004-08 Appendix F (corresponds to sort class S10 as defined by DIN 4074-1)
- Axial spacing: ≤ 625 mm
- Cross-section:
  The following criteria must be taken into consideration when choosing the rib cross-sections:
  - Cross-sectional area ≥ 30 cm² for factory production
  - Cross-sectional area ≥ 40 cm² for on-site assembly
  - Width b ≥ 40 mm,
  - Thickness h ≥ 50 mm (gypsum boards) or h ≥ 80 mm (gypsum fiber boards)
  - Minimum edge distances of the fasteners

Fasteners

- Steel staples must be resinated
- In the case of on-site assembly, the minimum dimensions for the edge distances must be increased in each case by 5 mm

Additionally required proof by the structural engineer

Proof of buckling:

- Is considered fulfilled for buckling in wall plane direction with cladding on both sides
- Is considered fulfilled for buckling in wall plane direction for single-sided cladding, if the wood cross-section has a lateral ratio of h / b ≤ 4 : 1
- Must be proven for buckling perpendicular to wall plane direction, in doing so, a possible vertical load must be taken into consideration

Proof of sill compression:

- The proof of sill compression under the edge rib must also be provided

Proof of anchoring:

- The anchoring (Z_A) of wood frame construction walls must be proven

Proof of load transfer:

- The connecting elements for the transfer of forces F_H to the wall panels must be proven separately

C3 Floor Constructions

C3.1 Prefabricated Screed

Table C.28 Live loads for prefabricated screed constructions as defined by /4.16/

Live loads		Load bearing layer		Optional configurations below load bearing layer Thickness in mm				
Area load [kN/m²]	Single load [kN]	Thickness in mm	**Material** Brio = Gypsum fiber boards Vidifloor = Gypsum fiber boards TUB = Special gypsum boards	Minera wool	Dry bulk leveler 1)	Dry bulk leveler 1) + covering board	Wood fiber WF	Expanded polystyrene (EPS)
2	1	18	Brio 18	10-20	20-100	20-100 +12.5	10-20	10-100
		20	Vidifloor 20					
		23	Brio 23					
		25	Vidifloor 25 / TUB 2x 12.5					
2	2	18	Brio 18	-	20-30	20-100 +12.5	10-20	10-100
		20	Vidifloor 20					
		23	Brio 23					
		25	Vidifloor 25 / TUB 2x 12.5					
3	2	18	Brio 18	-	-	20-100 +12.5	10-20	10-100
		20	Vidifloor 20					
		23	Brio 23					
		25	Vidifloor 25					
3	3	23	Brio 23	-	-	-	10-20	10-100
		25	Vidifloor 25					
		30.5	Brio 18 + TUB 12.5					
		32.5	Vidifloor 20 + TUB 12.5					
4	3	35.5	Brio 23 + TUB 12.5	-	-	-	10-20	10-100
		37.5	Vidifloor 25 + TUB 12.5					
		36	2x Brio 18					
		40	2x Vidifloor 20					
		37.5	3x TUB 12.5					
4	4	36	2x Brio 18	-	-	-	10-20	10-100
		40	2x Vidifloor 20					
		46	2x Brio 23					
		50	2x Vidifloor 25					
5	4	46	2x Brio 23	-	-	-	10-20	10-100
		50	2x Vidifloor 25					

[1] 1) The load bearing capacity of the dry bulk leveler must be proven

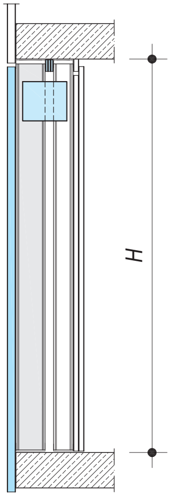
C3.2 Hollow Floors

Table C.29 Live loads for hollow floors as defined by /4.17/

Live loads		Load bearing layer (hollow floor unit)		Pedestal grid in mm
Area load [kN/m²]	Single load [kN]	Thickness [mm]	Material [1]	The choice of suitable hollow floor pedestals should be made taking into consideration the height of the cavity space in connection with the live load.
1	1	25	GIFAfloor FHB	600 x 600
2	1	25		600 x 600
2	2	25		600 x 600
3	3	25		600 x 600
3	4	25		425 x 425
		28		600 x 600
4	4	25		425 x 425
		28		600 x 600
5	4	25		425 x 425
		28		600 x 600
5	5	32		600 x 600
		28 + 13	GIFAfloor FHB + GIFAfloor LEP	600 x 600
5	6	28 + 13		425 x 425
6	7	32 + 13		425 x 425
6	7	32 + 18		600 x 600
6	10	32 + 18		425 x 425

[1] The values in Table C.29 apply exclusively for the gypsum fiber boards GIFAfloor FHB and GIFAfloor LEP specified in /4.17/ whilst taking into account the constructional specifications. Application is also possible in a line-supported design. Always observe the manufacturer's instructions if using other board makes.

C4.1 Double-Shell External Wall Constructions



In the case of double-shell external wall constructions, the specifications apply only for the outer shell. The inner shell must be dimensioned as a furring in accordance with C.2.2.

*Table C.30 Permissible wall heights of Aquapanel external walls, spacing of studs **a = 40 cm**, double-shell construction (dead weight of the outer shell **up to 39 kg/m²**) in dependence on the seismic load*

<table>
<tr><th rowspan="2">S • a_g
m/s²</th><th colspan="5">Permissible wall height H in m</th></tr>
<tr><th>CW100</th><th>CW125</th><th>CW150</th><th>2xCW125</th><th>2xCW150</th></tr>
<tr><td>0 - 3.6</td><td>3.4</td><td>4</td><td rowspan="3">4</td><td rowspan="4">4</td><td rowspan="5">4</td></tr>
<tr><td>4.0 - 4.5</td><td>3.1</td><td>3.7</td></tr>
<tr><td>5.0 - 5.6</td><td>3</td><td>3.6</td></tr>
<tr><td>6.3</td><td>2.9</td><td>3.4</td><td>3.9</td></tr>
<tr><td>7.2</td><td>2.7</td><td>3.2</td><td>3.7</td><td>3.8</td></tr>
</table>

*Table C.31 Permissible wall heights of Aquapanel external walls, spacing of studs **a = 60 cm**, double-shell construction (dead weight of the outer shell **up to 39 kg/m²**) in dependence on the seismic load*

<table>
<tr><th rowspan="2">S • a_g
m/s²</th><th colspan="5">Permissible wall height H in m</th></tr>
<tr><th>CW100</th><th>CW125</th><th>CW150</th><th>2xCW125</th><th>2xCW150</th></tr>
<tr><td>0 - 3.6</td><td>3</td><td>3.5</td><td>4</td><td rowspan="2">4.</td><td rowspan="5">4</td></tr>
<tr><td>4.0 - 4.5</td><td>2.7</td><td>3.2</td><td>3.7</td></tr>
<tr><td>5.0 - 5.6</td><td>2.6</td><td>3</td><td>3.5</td><td>3.8</td></tr>
<tr><td>6.3</td><td>2.5</td><td>2.9</td><td>3.4</td><td>3.7</td></tr>
<tr><td>7.2</td><td>-</td><td>2.8</td><td>3.2</td><td>3.5</td></tr>
</table>

C4.2 Single-Shell External Wall Constructions

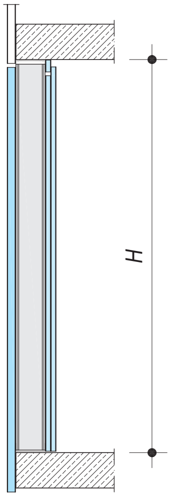


*Table C.32 Permissible wall heights of Aquapanel external walls, spacing of studs **a = 40 cm**, single-shell construction (dead weight of the wall **up to 48 kg/m²**) in dependence on the seismic load*

<table>
<tr><th rowspan="2">$S \cdot a_g$
m/s²</th><th colspan="5">Permissible wall height H in m</th></tr>
<tr><th>CW100</th><th>CW125</th><th>CW150</th><th>2xCW125</th><th>2xCW150</th></tr>
<tr><td>0 - 2.7</td><td>3.4</td><td>4</td><td rowspan="3">4</td><td rowspan="5">4</td><td rowspan="6">4</td></tr>
<tr><td>3.2 - 3.6</td><td>3.1</td><td>3.7</td></tr>
<tr><td>4.0 - 4.5</td><td>3</td><td>3.6</td></tr>
<tr><td>5.0</td><td>2.9</td><td>3.4</td><td>3.9</td></tr>
<tr><td>5.4 - 5.6</td><td>2.8</td><td>3.3</td><td>3.8</td></tr>
<tr><td>6.3 - 7.2</td><td>2.6</td><td>3</td><td>3.5</td><td>3.8</td></tr>
</table>

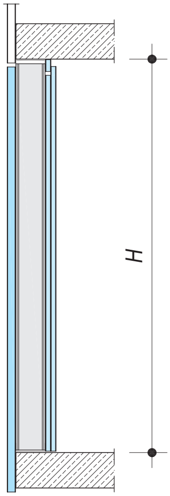


*Table C.33 Permissible wall heights of Aquapanel external walls, spacing of studs **a = 60 cm**, single-shell construction (dead weight of the wall **up to 48 kg/m²**) in dependence on the seismic load*

<table>
<tr><th rowspan="2">$S \cdot a_g$
m/s²</th><th colspan="5">Permissible wall height H in m</th></tr>
<tr><th>CW100</th><th>CW125</th><th>CW150</th><th>2xCW125</th><th>2xCW150</th></tr>
<tr><td>0.0 - 2.7</td><td>3</td><td>3.5</td><td>4</td><td rowspan="2">4</td><td rowspan="6">4</td></tr>
<tr><td>3.2 - 3.6</td><td>2.7</td><td>3.2</td><td>3.7</td></tr>
<tr><td>4.0 - 4.5</td><td>2.6</td><td>3</td><td>3.5</td><td>3.8</td></tr>
<tr><td>5.0</td><td>2.5</td><td rowspan="2">2.9</td><td>3.4</td><td>3.7</td></tr>
<tr><td>5.4</td><td rowspan="4">-</td><td>3.3</td><td>3.6</td></tr>
<tr><td>5.6</td><td>2.8</td><td>3.2</td><td>3.5</td></tr>
<tr><td>6.3</td><td>2.7</td><td>3.1</td><td>3.4</td><td>3.9</td></tr>
<tr><td>7.2</td><td>2.6</td><td>3</td><td>3.3</td><td>3.8</td></tr>
</table>

D Standards

In order to gain some clarity, despite the broad bandwidth of normative regulations in the individual countries, this book essentially deals with the European EN or German DIN standards, as these are applied and recognized in a multitude of countries also beyond the actual countries of destination.
The following specifies the standards for earthquakes, load assumptions and drywalling for selected countries.

D1 Standards and Directives on Earthquake Resistance

EN 1998 Eurocode 8	Design of structures for earthquake resistance [Countries of the European Union and others that adopt the EN standards]
OENORM B 1998	Design of structures for earthquake resistance - National specifications concerning EN 1998 [Austria]
DIN 4149	Buildings in german earthquake areas - Design loads, analysis and structural design of buildings [Germany]
SNiP II-7-81	Construction in seismic regions [Russia]
NF P06-013	Rules for earthquake-resistant construction Rules to protect buildings against earthquakes Règles PS 92 – Règles de construction parasismiques applicables aux bâtiments [France]
NF P06-013/A1	Rules for earthquake-resistant construction Rules to protect buildings against earthquakes [France]
OENORM B 4015	Design loads in building - Accidental actions - Seismic actions - General principles and methods of calculation [Austria]
D.M.L.P.	Technical standards for earthquake-resistant buildings [Italy]
ABYYHY	Specifications for Structures to be Built in Disaster Areas Part III - Earthquake Disaster Prevention [Turkey]
P100	Cod de proiectare seismică - Seismic design code [Romania]
SNI 03-1726-2002	Guideline for Seismic Design Construction - Building Regulation [Indonesia]
NCh 3	Seismic Intensity Scale [Chile]
NCh 433	Earthquake Resistant Resign of Buildings [Chile]

NCh 2369	Seismic Design of Structures and Industrial Installations [Chile]
NCh 2745	Analysis and Design of Buildings with Seismic Insulations [Chile]
IBC`06	International Building Code [USA]
UBC`97	Uniform Building Code [USA]

D2 Standards Regarding Design Loads

ISO 9194	Bases for design of structures; Actions due to the self-weight of structures, non-structural elements and stored materials; Density
EN 1991	Eurocode 1 Actions on structures [Countries of the European Union and others that adopt the EN standards]
OENORM B1991	Eurocode 1 Actions on structures - National specifications concerning EN 1991 and national supplements [Austria]
DIN 1055	Action on structures [Germany]
NF P06-001	Bases for design of structures – dead loads Base de calcul des constructions – Charge d'exploitation des bâtiments [France]
P06-002, P06-006	Snow and wind loads on constructions Règles NV65 et N84 modifiées. Règles définissant les effets de la neige et du vent sur les constructions et annexes. [France]
OENORM B 4000	Actions on structures - General principles of calculation for building construction and application rules for self-weight, stored materials, imposed loads for buidlings, snow and ice loads [Austria]
OENORM B 4014-1	Design loads on structures - Static wind loads (non-sway structures) [Austria]

OENORM B 4014-2	Design loads on structures - Dynamic wind actions (sway structures) [Austria]
SIA 261	Actions on structures [Switzerland]

D3 Drywalling Standards

Countries of the European Union and others that adopt the EN standards

EN 520	Gypsum plasterboards - Definitions, requirements and test methods
EN 13162	Thermal insulation products for buildings - Factory made mineral wool (MW) products - Specification
EN 13950	Gypsum plasterboard thermal/acoustic insulation composite panels - Definitions, requirements and test methods
EN 13963	Jointing materials for gypsum plasterboards - Definitions, requirements and test methods
EN 13964	Suspended ceilings - Requirements and test methods
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DIN 1052	Design of timber structures - General rules and rules for buildings

DIN 4103-1	Internal non-load bearing partitions; requirements, testing
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DIN 18168-1	Ceiling linings and suspended ceilings with gypsum plasterboards - Part 1: Requirements for construction
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E Manufacturer's Addresses

Albania	Knauf Sh.p.k Tirana Rr. Jeronin Derada, Kulla 2, Kat II Tirana Tel.: (+355) 4248714 info@knauf.com.al
Algeria	K.P.F. - Knauf Plâtres Fleurus Bp n° 2, Boufatis Wilaya 31000 Oran Tel.: (+213) 41-521050
Argentina	Yesos Knauf GmbH - Sucursal Argentina Bartolomé Cruz 1528 2° Piso B1638BHL Vicente López Pcia de Buenos Aires Tel.: (+54) 11-48370700 knauf@knauf.com.ar
Armenia	Knauf Armenia Zavaryan # 1 apt. 35 Yerevan Tel. (+374) 10-610433 r.movahed@knaufir.com
Austria	Knauf Ges. m.b.H. Wien Strobachgasse 6 1050 Wien Tel.: (+43) 1 / 580 68-0 info@knauf.at
Azerbaijan	Knauf GmbH Sahibtaj Business Center, 2nd floor A. Zeynalli Street 31 - 33 1000 Baku Tel.: (+994) 12-4977908 info@knauf.az

Bosnia and Herzegovina

Knauf d.o.o. Sarajevo
Kolodvorska 11 a
71000 Sarajevo
Tel.: (+387) 33-711090
knauf@knauf.ba

Bulgaria	Knauf EOOD Angelov Vrach 27 1618 Sofia Tel.: (+359) 2-9178910 info@knauf.bg
Chile	Knauf de Chile LTDA. San Ignacio 181 Quilicura Santiago de Chile Tel.: (+56) 2-5849400 info@knauf.cl
China	Knauf Plasterboard Tianjin C0. Ltd. North Side of Yinhe Bridge East Side of Jinggjin Highway, Beichen District Tianjin, P.R.C. Tel.: (+86) 22-26972777 info@mail.knauf.com.cn
Croatia	Knauf d.o.o. Ulica grada, Vukovara 21 10000 Zagreb Tel.: (+385) 1 3035-400 knauf@knauf.hr
Cyprus	Knauf Cyprus Limited 28th October Str., Lophitis Business - Centre 3035 Limassol Tel.: (+357) 25-343371 knauf@knauf.com.cy

France	Knauf SAS Zone d'Activites 68600 Wolfgantzen Tel.: (+33) 389-72-1100 info@knauf.fr
Georgia	Knauf Marketing Tbilisi Ges.m.b.H. Gamrekeli 19 0160 Tbilisi Tel.: (+995) 32242502 info@knauf.ge
Greece	Knauf Gypsopiia ABEE Euripidou 10 17674 Kallithea / Athens Tel.: (+30) 210-9310567 knauf@knauf.gr
Hungary	Knauf Kft. Lejtö út. 5 1124 Budapest Tel.: (+36) 1 248 2430 info@knauf.hu
Indonesia	PT Knauf Gypsum Indonesia The Landmark CentreTower A - 19th Floor Jl. Jend. Sudirman No.1 12910 Jakarta Tel.: (+62) 21 522 9558 sales@knauf.co.id
Iran	Knauf Iran P.J.S.C. No. 31 Shahid Naghdi St. North Mofateh Ave. 15766 Teheran Tel.: (+98) 21-88751680 knaufiran@hotmail.com

Italy	Knauf di Lothar Knauf s.a.s. Loc. Podere Paradiso 56040 Castellina Marittima (PI) Tel.: (+39) 050 692-201 / 202 info@knauf.it
Jordan	Knauf Jordan Wasfi AL-Tal-Str. 111 AL-Bakri Commercial Center P.O. Box 143649 11814 Amman Tel.: (+962) 6-5521721 knauf@co.com.jo
Kazakhstan	TOO Knauf Gips Kaptschagaj Almaty area 040800 Kaptschagaj, Saretschnij Tel.: (+7) 32772 23496 kaptchagaja@knauf.kz
Kosovo	Knauf sh.p.k. Prishtina Qamil Hoxha, Blok I 10000 Prishtina Tel.: (+381) 38-243926 knaufprishtina@hotmail.com
Lebanon	Knauf Lebanon S.A.R.L. Sadeco Square Block-C 12th Floor Achrafieh Beirut 2062-1212 Tel.: (+961) 1-612209 knauf@intracom.net.lb
Marocco	Knauf Marocco 30, rue Abou Faris al Marini, Ap.nr.3 MA-10000 Rabat-Maroc Tel.: (+212) 7-736029
Macedonia	Knauf dooel Skopje ul. Aleksandar Makedonski bb 1000 SKOPJE Tel.: (+389) 23-235750 info@knauf.com.mk

Portugal	Knauf Lisboa GmbH Rua Poeta Bocage, 14 D-1°C 1600-581 Lissabon Tel.: (+351) 21 7112750
Romania	Knauf Gips S.R.L. Str. Nicolae Iorga - Hr. 13 - Sector 1 010432 Bukarest Tel.: (+40) 21 65000-40 office@knauf.ro
Russian Federation	
	Knauf Gips, Krasnogorsk Ul. Zentralnaja 139 143400 Krasnogorsk Oblast Moskau, Russland Tel. (+7) 495 937 95 95
Switzerland	Knauf AG Kägenstraße 17 4153 Reinach BL Tel.: (+41) 61 71610-10 info@knauf.ch
Serbia	Knauf d.o.o. Beograd Dalmatinska 65a 11000 Beograd Tel.: (+381) 11 2074500 info@knauf.co.yu
Slovakia	Knauf Bratislava s.r.o. Turčianska 1/A 82109 Bratislava Tel.: (+421) 2 58240831 info@knauf.sk
Slovenia	Knauf Ljubljana d.o.o. Dunajska cesta 115 1000 Ljubljana Tel.: (+386) 1 5682 279 info@knauf.si

Spain	Knauf GmbH Avda. Manoteras 10 Edificio “C“ 28050 Madrid Tel.: (+34) 91 3830540 info@knauf.es
Turkey	Knauf A.S. - Ankara P.K. 92 Bakanliklar 06582 Ankara Tel.: +90 312 29701-00 mailbox@knauf.com.tr
Uzbekistan	Knauf Marketing Tashkent GmbH Mirakilowa 68a 700070 Tashkent Tel.: (+998) 71 555269 knauf@uzpak.uz

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Figure

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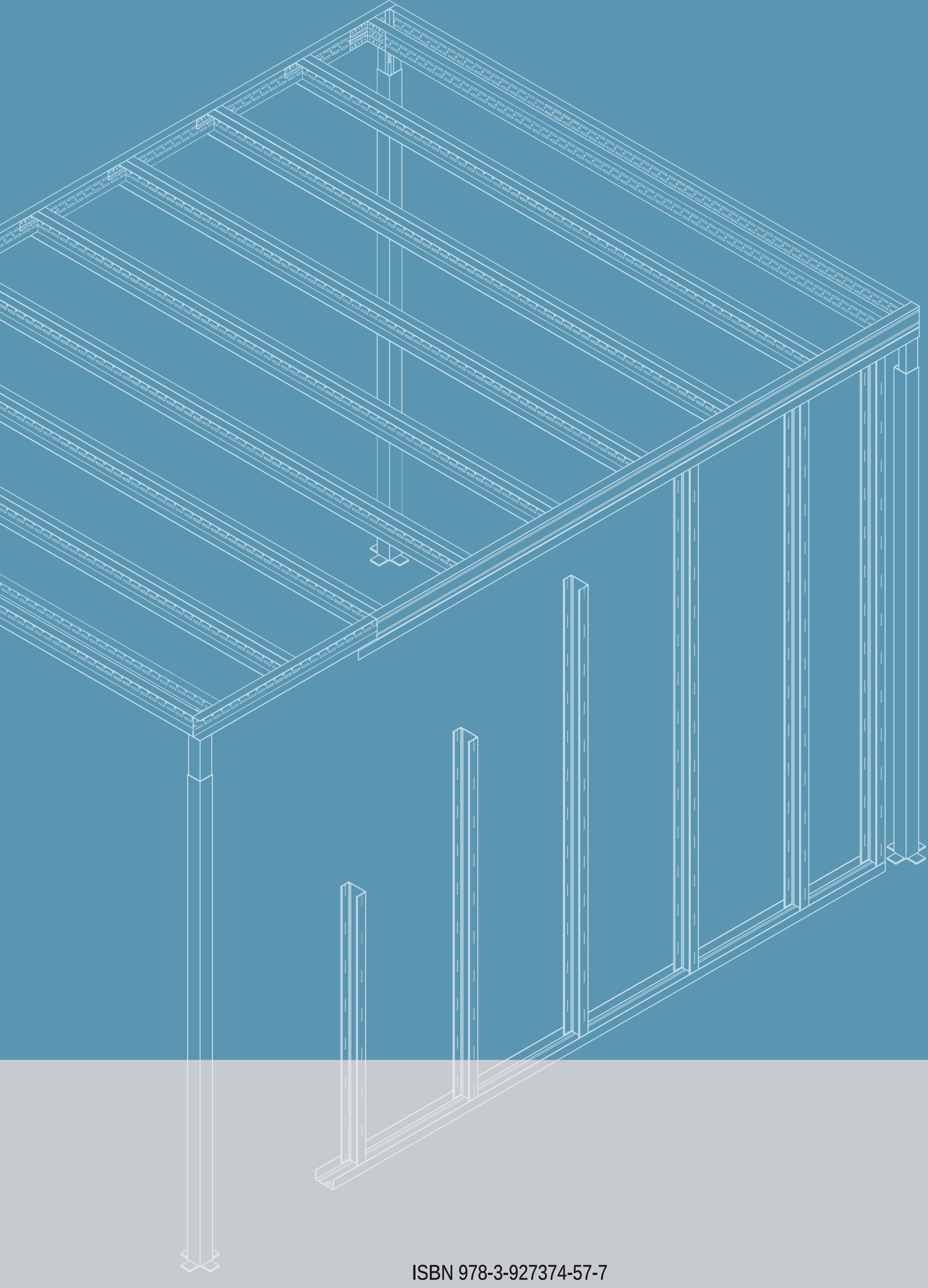

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