Editors: Raffaele Landolfo, Dennis Holl

Lightweight steel drywall constructions for seismic areas

Design, research and applications

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Editors:

Prof. Dr. Ing. Raffaele Landolfo, University of Naples Federico II

Dipl.-Ing. Dennis Holl, Knauf Gips KG, Germany

Team of authors:

Luigi Fiorino, PhD, University of Naples Federico II, Italy

Prof. Maria Founti, PhD, National Technical University of Athens, Greece

Dipl.-Ing. Dominik Herfurth, Knauf Gips KG, Germany

Dipl.-Ing. Dennis Holl

Ornella Iurio, PhD,University of Naples Federico II, Italy

Dr.-Ing. Georg Krämer

Prof. Dr. Ing. Raffaele Landolfo

Vincenzo Macillo, PhD, University of Naples Federico II, Italy

Tatiana Pali, PhD,University of Naples Federico II, Italy

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1 Foreword

Raffaele Landolfo, Dennis Holl

Necessity of seismic design

The increase in the exploitation of the natural resources of our planet - also due to the exponential growth of emerging countries - presents new challenges for society. With respect to the construction industry, these lead to the endless research of new and innovative solutions, which must also be competitive in terms of sustainability, i.e. with energy conservation demands, reduced CO_2 emissions, limited production of waste materials, etc.

In addition to sustainability requirements, a fundamental issue for structural assets is in meeting traditional performance requirements in terms of safety, especially with reference to seismic actions.

As a matter of fact, earthquakes caused approximately 2.5 million deaths and over 2.9 trillion US dollars of damage since 1900. Indeed, it is clear that the question of seismic risk reduction becomes one of the most important concerns for engineers.

From a structural point of view, this means concentration on the development of vulnerability reduction strategies, above all through the development of seismic efficient systems.

Lightweight steel drywall systems

The objective is to analyse the traditional building methods and, where necessary, to set out and pursue new approaches. Lightweight steel construction is one of the innovative building methods steadily increasing in significance due to its economic efficiency and ecological performance, and which solves a range of "built-in" problems arising for common building methods with earthquake safety relevant properties and practices, without making any adverse compromises on the performance requirements of the structures.

Over the last few decades, many research projects and applications have undoubtedly demonstrated the good response of lightweight steel drywall systems in terms of structural safety, for both gravity and seismic loads and for either non-structural and structural elements. This is possible thanks to the load bearing skeleton, which is characterized by high structural efficiency due to the highest capacity-to-weight ratio offered by cold-formed steel profiles, with respect to the more traditional constructive products currently on the market.

In addition, an even higher structural performance may be reached when a cladding-braced approach is used, i.e. if the positive effect of interaction between steel elements and cladding is taken into account.

As a result, in terms of seismic response, lightweight steel drywall systems are extremely efficient and competitive with other common constructive solutions such as masonry, reinforced concrete and steel, e.g., elastic behaviour under design earthquakes (with 2 % probability of exceedance in 50 years), with consequential damage reduction also in the case of severe seismic conditions.

Integral approach

But as stated beforehand, the current trend on the construction market leads toward integrated solutions that must satisfy multiple requirements in terms of eco-efficiency (energy-saving, preservation of resources, recycling, pollutant emissions), safety (structural performance, fire protection), health and comfort (sound insulation, hygrothermal issues) without neglecting the economic aspects. In this perspective, the lightweight steel drywall elements represent reference products for both structural and non-structural applications.

In fact, the benefits of lightweight steel drywall systems in terms of environmental, structural and economic characteristics facilitate a full response to the old and new demands through integrated solutions.

Fig. 1.1: Earthquake damage of traditional constructions

Fig. 1.2: Earthquake damage of traditional constructions

Many strategies are used by the lightweight steel drywall industry to face the increasing environmental needs, like the reduction of material amount and waste, the increase of product's life, the adoption of lean productions with reduced use of energy.

These strategies, together with the own characteristics of used products (lightness, recyclability, reuse, etc.), the constructive aspects (dry constructions, short building times, reduced overall transport requirements, etc.), and the features of the system (flexibility, building accuracy, easy integration of services such as plumbing, cabling, ducting inside of elements, etc.) make lightweight steel drywall solutions a successful sustainable alternative compared to more traditional constructive systems.

By following a similar integral approach, after a general introduction on the current trends to seismic design and the basics of lightweight steel constructions, this book addresses the specific issues related to the seismic design of structural and non-structural drywall systems together with the basics of building physical aspects, principles of sustainability and integral design.

2 Current approach to seismic design

Vincenzo Macillo, Luigi Fiorino, Raffaele Landolfo

Earthquake occurrence represents a frequent natural phenomenon that, in the case of stronger events, can cause catastrophic effects such as the loss of human life and considerable damage to buildings and infrastructure with subsequent significant economic costs. The recent seismic events highlighted the importance of promoting and developing new strategies aimed at mitigating the damage related to earthquakes. This chapter presents an historical overview of the effects of catastrophic seismic events. A topical issue is the definition of the seismic risk as the possibility of different types of losses caused by the occurrence of a seismic event. It represents the complex combination of the seismic hazard, vulnerability and exposure of the element at risk. The definition of these concepts together with the information related to the mitigation of the seismic risk is provided. This chapter also deals with the principles for seismic design. In particular, the philosophy of performance-based design is illustrated. It consists of achieving different design objectives in terms of building performance for different stages of seismic hazard. In addition, information about the modern procedure for design, such as displacement-based design, and innovative solutions for structural systems are shown and discussed.

2.1 Earthquakes

2.1.1 Basic concepts

The incessant sequence of mass movements underneath the Earth's surface continuously causes earthquakes. In fact, every day, the seismic stations located all over the world record thousands of ground movements. It is estimated that about 9,000 small or imperceptible earthquakes not exceeding magnitude 3 occur every day. Furthermore, about one thousand destructive earthquakes, corresponding to magnitudes higher than 5, and a couple of significant events causing human and economic losses occur every year. Fig. 2.1 shows the geographical location of the 1,547 seismic events exceeding magnitude 5 that occurred world-wide during the year 2013 /2.1/.

Tab. 2.1 /2.1/ shows the number of world-wide destructive earthquakes from 2000 to 2012 divided by intensity and year. It is noticable that the annual number of seismic events with similar intensity is substantially constant, while the number of estimated deaths is highly variable, because it depends on the urbanization level of the zone affected by the earthquake and possible secondary effects such as tsunamis.

2.1.2 Historical overview of seismic events and their effects

In ancient times, the literature and the mythology left traces of strong earthquakes and their aftermaths. Greek mythology narrates that the cause of the earthquakes was Poseidon, the God of the sea, who shook the ground with his trident for revenge or when he was in a bad mood. Also in Japan, before the scientists started to study the phenomenon, mythological and divine explanation prevailed. According to Japanese mythology, the earthquakes are generated by a giant catfish named Namazu, which lives restrained underground controlled by the God Kashima. When the God lets down his guard, the fish starts to shake himself causing earthquakes.

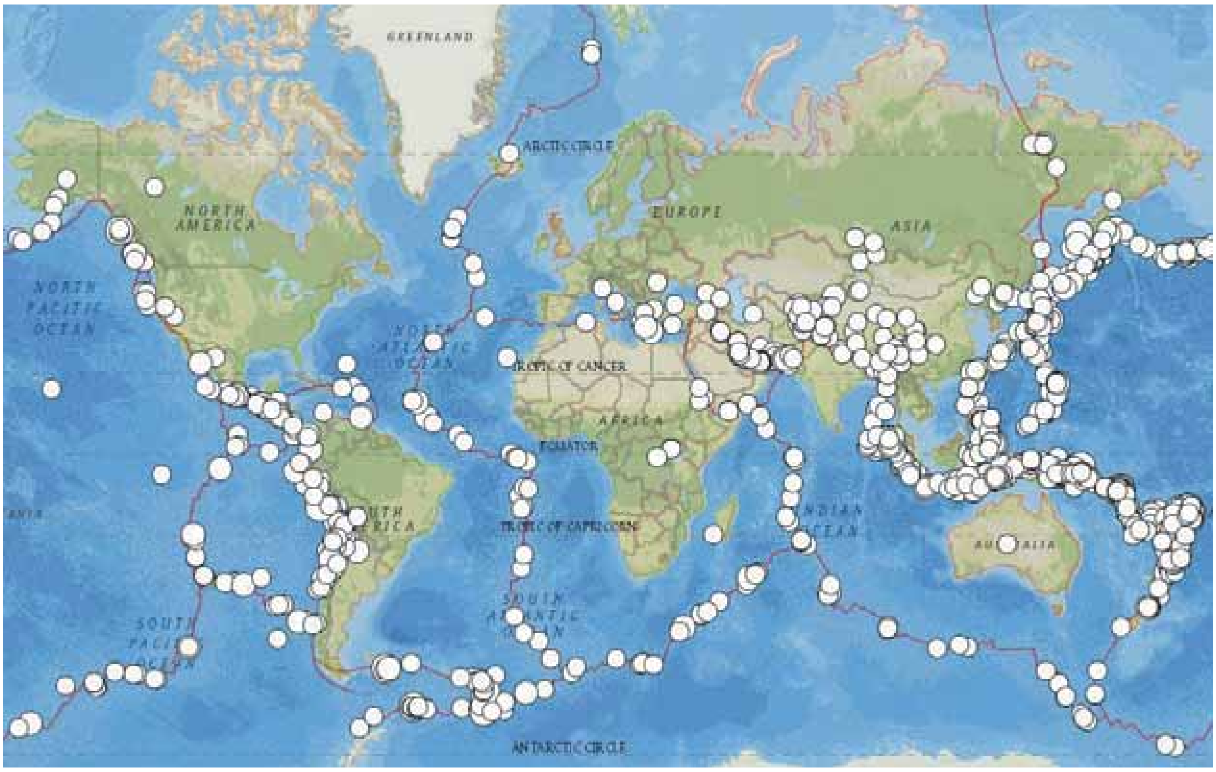


Fig. 2.1: World-wide earthquakes (M>5) in 2013 /Department of the Interior, U.S. Geological survey/

Tab. 2.1: Number of world-wide earthquakes (M>5) from 2000 to 2013 /USGS/

Year	**Magnitude**				**Total**	**Estimated deaths**
	5.0 – 5.9	**6.0 – 6.9**	**7.0 – 7.9**	**8.0 – 8.9**		
2000	1,344	146	14	1	1,505	231
2001	1,224	121	15	1	1,361	21,357
2002	1,201	127	13	0	1,341	1,685
2003	1,203	140	14	1	1,358	33,819
2004	1,515	141	14	2	1,672	228,802
2005	1,693	140	10	1	1,844	88,003
2006	1,712	142	9	2	1,865	6,605
2007	2,074	178	14	4	2,270	712
2008	1,768	168	12	0	1,948	88,011
2009	1,896	144	16	1	2,057	1,790
2010	2,209	150	23	1	2,383	320,120
2011	2,276	185	19	1	2,481	21,953
2012	1,401	108	12	2	1,523	768

Among the historical earthquakes, the 226 BC Rhodes earthquake is famous for having destroyed one of the seven wonders of the ancient world, the Colossus. The 17 AD Lydia earthquake, which destroyed several cities in the Roman province of Asia Minor, was also recorded by the Roman historian Tacitus, in his "Annales", and Pliny the Elder, who defined it as "the greatest earthquake in human memory". Seneca dealt with earthquakes in the sixth book of his "Naturales quaestiones", and he provided an account of the 62 AD Pompeii earthquake,

Tab. 2.2: Main earthquakes that occurred in the 20th century /Gioncu, V., Mazzolani, F.M. (2011)/

Year	Location	Magnitude	Death toll
1905	India, Kangra	8.6	20,000
1906	USA, San Francisco	7.8	1,000
1906	Chile, Valparaiso	8.2	20,000
1906	Ecuador, Esmeralda	8.8	1,000
1908	Italy, Messina and Reggio Calabria	7.5	83,000
1920	China, Gansu	8.6	220,000
1923	Russia, Kamchatka	8.5	-
1923	Japan, Kanto	8.3	143,000
1927	China, Xining	8.3	200,000
1932	China, Gansu	7.6	70,000
1935	Pakistan, Quelta	7.5	60,000
1938	Indonesia, Banda Sea	8.5	-
1939	Chile, Conception	8.3	25,000
1939	Turkey, Erzincan	7.9	25,000
1940	Romania,Vrancea	7.4	-
1940	USA, El Centro	7.1	9
1948	Turkmenistan, Ashgabat	7.3	110,000
1950	India, China border	8.6	-
1952	Russia, Kamchatka	9.0	-
1957	USA, Aleutian Islands	9.1	-
1960	Morocco, Agadir	5.9	12,000
1960	Chile, Valdivia	9.5	6,000
1963	Kuril Islands	8.5	-
1964	USA, Alaska, Anchorage	9.2	116
1970	Peru, Ancash	8.1	66,000
1976	China, Tangshan	8.0	250,000
1977	Romania, Vrancea	7.2	1,600
1980	Algeria, El Asnam	7.3	9,700
1985	Mexico, Mexico City	8.1	2,000
1988	Armenia, Spitak	7.1	25,000
1989	USA, Loma Prieta	7.1	70
1990	Iran, Manjil	7.7	40,000
1993	India, Killari	6.3	23,000
1994	USA , Northridge	6.7	63
1995	Japan, Kobe	6.9	5,600
1997	Iran, Ardebil	7.1	1,600
1998	Afghanistan, Rostaq	7.1	5,000
1999	Turkey, Izmit	7.4	20,000
1999	Taiwan, Chi-Chi	7.3	2,500

Fig. 2.2: 1906 San Francisco earthquake /Department of the interior/U.S. Geological Survey/

Fig. 2.3: 1908 Messina earthquake /Museo dell'Osservatorio Vesuviano - INGV/

which caused great damage in the towns of Pompeii and Herculaneum.

The 1556 Shaanxi earthquake (China) is in all likelihood the deadliest earthquake of all times with an estimated death toll of 830,000 people and an involved area 840 kilometres wide. The high loss of life was caused by the collapse of entire villages of yaodongs, which are artificial caves made of soft rocks.

The birth of the science of Seismology, as we know it, corresponds to the occurrence of the Great Lisbon earthquake (Portugal) in 1755 /2.2/. In fact, the first studies about wave propagation were carried out after this event. It was one of the most dramatic natural events of European history, with at least 60,000 deaths. The earthquake, having a magnitude approaching 9.0, together with the subsequent tsunami and fires destroyed about two-third of Lisbon.

The main earthquakes that occurred in the last century having a magnitude greater than 7 or with high fatalities are listed in Tab. 2.2 /2.3/, together with information about date, magnitude and death toll. It has to be noted that the very strong earthquakes, i.e. 1964 Alaska (M 9.2), 1957 Aleutian Island (M 9.1) and 1952 Kamchatka (M9.0), did not cause a great number of deaths, because they fortunately occurred in less populated areas. Only the 1960 Valdivia (Chile) earthquake, whose magnitude (M 9.5) is the most powerful ever recorded, caused about 6,000 deaths. Several cities were heavily damaged, and many small villages were completely destroyed. However, the death toll was not enormous, because the population was alerted by several previous tremors. Instead, the 1964 Great Alaskan earthquake caused only 115 deaths mainly due to the effects of a frightening tsunami with waves up to 67 m.

The engineering approach to earthquakes during the 20th century can be divided into three different periods /2.3/. In the first period until 1950, earthquake engineering was not yet developed, and there were no rules for guaranteeing the seismic protection of buildings in earthquake prone areas. Therefore, the earthquakes of this period caused large damage in urban zones with many fatalities. The effect of these earthquakes led to the development of the practice of earthquake engineering and studies about the nature of earthquakes.

The start of a scientific conception of structural design against earthquakes in the United States corresponds to the 1906 San Francisco earthquake (M 7.8), which was remembered as one of worst disasters in the US history. It occurred close to a very urbanized area, and it sparked a devastating fire that lasted several days. It destroyed large areas (over 80 %) of San Francisco city, especially the older buildings that were not structurally prepared, but the greatest part of the destruction was a consequence of the resulting fire (Fig. 2.2 /2.1/). After this event, the Seismological Society of America was established.

The 1908 Messina and Reggio Calabria earthquake (M 7.5) can be considered the origin of earthquake engineering in Italy /2.4/. In fact, after this event, the first recommendation for the design of seismic-resistant structures was defined. The earthquake, with about 83,000 fatalities, is considered the deadliest that occurred in Europe. The main reason for the deaths and

Fig. 2.4: 1994 Northridge earthquake /Department of the interior/U.S. Geological Survey/

Fig. 2.5: 1995 Kobe earthquake /Kobe City/

destruction was the subsequent tsunami that struck nearby on the coast with 15 m waves (Fig. 2.3 /2.5/).
Another important seismic event occurred at the beginning of the 20th century with the 1923 Kanto earthquake (M 8.3) in Japan. It struck the metropolitan area of Tokyo-Yokohama, where more than half of the buildings were heavily damaged or destroyed. The building damage was mainly due to the subsequent fires. Furthermore, the earthquake caused a 12 m tsunami on the Atami coast. After this event, the first research group aimed at studying both seismology and earthquake engineering was formed in Japan.
The 1940 El Centro earthquake (M 7.1), which occurred in Southern California, is very important from a scientific point of view, because it was the first major earthquake recorded by a strong-motion seismograph. The event caused significant damage to buildings of several towns in Imperial Valley and killed nine people.
During the second period of the 20th century (1950-1980), wide-ranging theoretical studies were developed, but little information about the characteristics of the ground motion for the earthquakes that occurred in this period is available, because there were only a small number of seismic instruments. In this period, the 1970 Peru Ancash earthquake (M 8.1) was one of the most frightening seismic events of the century with 66,000 deaths and several cities significantly damaged.
The most deadly earthquake of the century was the 1976 Tangshan earthquake (M 8.0) in China. It struck and completely destroyed the densely populated industrial coal-mining city of Tangshan. This event was followed by a 7.8 magnitude aftershock a few hours after, which increased the number of deaths (250,000).
In the last period of the 20th Century (1980-2000), the wide-ranging advances in the science of seismology and the greater number of records available were used to develop the anti-seismic concepts for supporting engineers in the practical application. In addition, the building damage due to earthquakes were cause for reflection in improving the structural codes.
In the last period of the century, the most important event in the United States was the 1994 Northridge earthquake (M 6.7). It was a near-fault ground motion that occurred in a heavily populated zone, and it provided an opportunity to investigate the effects of this type of event. Many reinforced concrete structures (buildings and bridges) were severely damaged, and this earthquake caused the highest economic losses in the US history (Fig. 2.4). After this event, the building codes were revised, and many changes were introduced in terms of design and construction practice.
The 1995 Kobe earthquake (M 6.9) was a very devastating earthquake that occurred in Japan with 5,600 fatalities. The causes of its high level of destruction are related to the location of its epicentre, very near to the highly populated city of Kobe, and the subsequent fire and the soil liquefaction. Major damage and collapse of buildings and infrastructures were recorded (Fig. 2.5).
The 1999 Izmit earthquake (M 7.4) in Turkey was the most important event in the Eastern Mediterranean Basin, in terms of the number of deaths and damage. A large number of collapses of traditional constructions were

Tab. 2.3: Main earthquakes that occurred in the period 2000-2014

Year	Location	Magnitude	Death toll
2001	Peru, Atico	7.9	20,000
2001	Peru, Atico	8.4	2,000
2001	El Salvador, San Miguel	7.9	1,100
2002	Algeria, Boumerdès	6.8	2,266
2003	Iran, Bam	6.7	35,000
2004	Indonesia, Sumatra	9.0	283,000
2005	Indonesia, Sumatra	8.7	1,520
2005	Pakistan, Kashmir	7.6	79,000
2006	Indonesia, Banda sea	7.6	-
2006	Russia, Koryakia	7.6	-
2006	Pacific, Tonga	7.9	-
2006	Indonesia, Java	7.7	-
2006	Russia, Kuril Islands	8.3	
2006	Taiwan, Hengchun	7.2	2
2007	Russia, Kuril Islands	8.1	-
2007	Solomon Islands	8.1	28
2007	Peru, Chincha Alta	8.0	650
2007	Chile, Tocopilla	7.7	2
2007	Martinique Islands	7.4	1
2007	Fiji Islands	7.8	
2008	China, Sichuan	7.9	67,180
2009	Italy, L'Aquila	6.3	281
2009	Samoa Islands	8.1	189
2009	Indonesia, Sumatra	7.6	1,115
2010	Haiti, Léogâne	7.0	316,000
2010	Chile, Bio-Bio	8.8	525
2010	China, Quingai	6.9	2,700
2011	Japan, Tohoku	9.0	20,000
2012	Italy, Emilia	5.9	27
2012	Indonesia, Sumatra	8.6	11
2013	Solomon Island	8.0	18
2014	Chile, Iquique	8.2	6

recorded due to poor structural design and execution.

The last devastating event of the century was the Chi-Chi earthquake (M 7.3) in Taiwan. It was a near-fault ground motion that caused severe damage to infrastructure (bridges, highways and dams) and the toppling of high buildings. Also in this case, the poor quality of the execution resulted in building collapses, clearly indicating that poor construction is responsible for people's deaths.

The first years of the 21st century were characterized by many devastating earthquakes that caused a large number of fatalities, as shown in Tab. 2.3. The most significant events are summarized in the following.


Fig. 2.6: Bam citadel before and after the earthquake /Auroville Earth Institute/

Fig. 2.7: 2009 L'Aquila earthquake /Luigi Innocenzi, INGV/

Fig. 2.8: 2012 Emilia earthquake /Adriano Cavaliere, INGV/

The 2003 Bam earthquake (M 6.7) was a very catastrophic event that occurred in Iran. It caused a very large number of victims (35,000) and destroyed 70 % of the city, which was the most popular tourism area of the country. In fact, the ancient citadel of Bam was the biggest adobe construction in the world and it was irreparably destroyed with terrible cultural damage (Fig. 2.6).

The 2004 Sumatra earthquake (M 9.0), known also as the Indian Ocean earthquake, was one of the strongest earthquakes ever recorded. It was one of the deadliest natural disasters in history, with about 280,000 deaths. The tremendous damage was not caused by the earthquake but by a major tsunami, which affected Indonesia, Sri Lanka, India, Thailand, the Maldives and also part of the East African coast.

The 2008 Sichuan earthquake (M 8.0), which occurred in China, was a very shallow earthquake, with an epicentre depth of 19 km. This fact, together with the high density of population, resulted in a high number of fatalities. It has to be noted that the losses were increased by the presence of many buildings unprepared to resist earthquakes.

The Italian 2009 L'Aquila earthquake was a highly destructive earthquake, although the magnitude was not very high (M 6.3). The causes of this high damage are the near-fault conditions and the unprepared constructions. In fact, many reinforced concrete buildings were seriously damaged, and some small villages with masonry houses were destroyed (Fig. 2.7). Significant damage was also recorded in downtown historical buildings /2.6/.

The 2010 Chile earthquake was a very strong event (M 8.8). The main building damage occurred in residential buildings, generally made of masonry or reinforced concrete. However, considering the high magnitude of the event, the damage was limited thanks to a conservative building code and good code supervision /2.7/.

The 2011 Tohoku earthquake, with a very high magnitude (M 9.0), was the strongest earthquake that ever struck Japan. The event was followed by a powerful tsunami, which caused nuclear accidents. It also caused severe damage to infrastructure, such as bridges, transport networks, dams and ports.

In 2012, the Italian region of Emilia was shaken by a series of earthquakes with magnitudes up to 5.9. A wide area including many villages and towns was affected by the events. The main damage was recorded in historical buildings, and many industrial buildings made of precast concrete collapsed (Fig. 2.8).

During the preparation of this book, a devastating seismic event with a magnitude of 7.8 occurred in Nepal (April 2015). The earthquake caused about 9,000 deaths mainly in Nepal, but casualities were recorded also in China, India and Bangladesh. In the days following the main event, several strong aftershocks with magnitudes up to 7.3 occurred. After the earthquake, the International Airport of Kathmandu was temporarily closed and reopened, on the same day, to facilitate relief operations. Many historical buildings of Kathmandu Durbar Square, a UNESCO world heritage site, were destroyed as also occurred for the Dharahara tower and Manakamana temple in the Gorkha district. The earthquake also caused avalanches on Mount Everest, which killed 19 people.

2.1.3 Seismic risk

In general, the risk can be defined as the probability of damage and expected losses of a given element exposed to the hazard of a particular natural or man-made phenomenon, over a specified future period of time. Together with the concept of risk, a fundamental role is played by the concept of hazard, related to the probability of occurrence of a given natural phenomenon which can potentially cause harm /2.8/.

Clearly, these concepts can be also adopted for earthquakes, and it is possible to speak of seismic risk. The seismicity represents a physical characteristic aspect of the territory, related to the frequency and the intensity of earthquakes. The knowledge of these parameters, together with the probability of occurrence of a seismic event for a given geographical area, is defined as the seismic hazard. It is higher when, for a given temporal interval, the occurrence of an earthquake with high magnitude is more likely. The consequences of an earthquake do not depend only on the earthquake intensity, but also on the resistance of buildings to the seismic actions. The susceptibility of a building to be damaged is defined as vulnerability, and more vulnerable buildings are more inclined to be harmed during the earthquake. In addition, the presence of assets at risk, which is related to the possibility of economic loss and loss of human life, is defined as exposure.

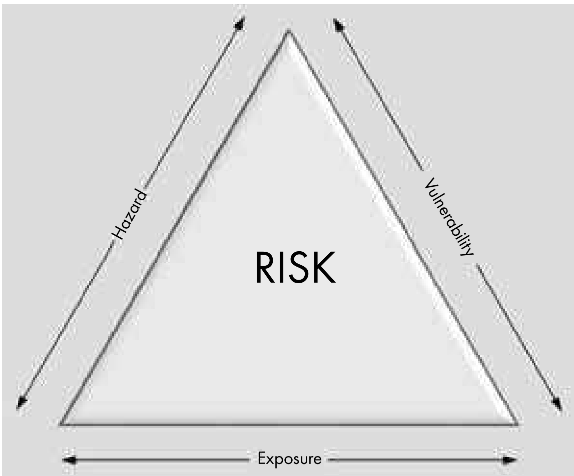


Fig. 2.9: The risk triangle /Crichton, D. (1999) "The Risk Triangle" /2.9//

Therefore, the seismic risk, defined as the probability of loss and damage due to an earthquake, is a complex combination of hazard, vulnerability and exposure and can be qualitatively expressed through the well-known pseudo-equation (Fig. 2.9 /2.9/):

Seismic Risk = Hazard × Vulnerability × Exposure

In this pseudo-equation, the mathematical symbol of multiplication is used because of the different terms combined with each other. In fact, it can be noted that high seismic hazard does not always mean high seismic risk and vice versa. In fact, in a non-populated area, the exposure is zero, and also a very high seismic hazard cannot generate any seismic risk. On the other hand, a densely populated area with poor constructions (high exposure and vulnerability) can have a high seismic risk even in a low seismic hazard zone. Therefore, hazard itself cannot generate risk, while development without correct planning does.

The control of the seismic risk depends on the knowledge of its three components. The seismic hazard of an area is defined as the probability of occurrence, in a certain time

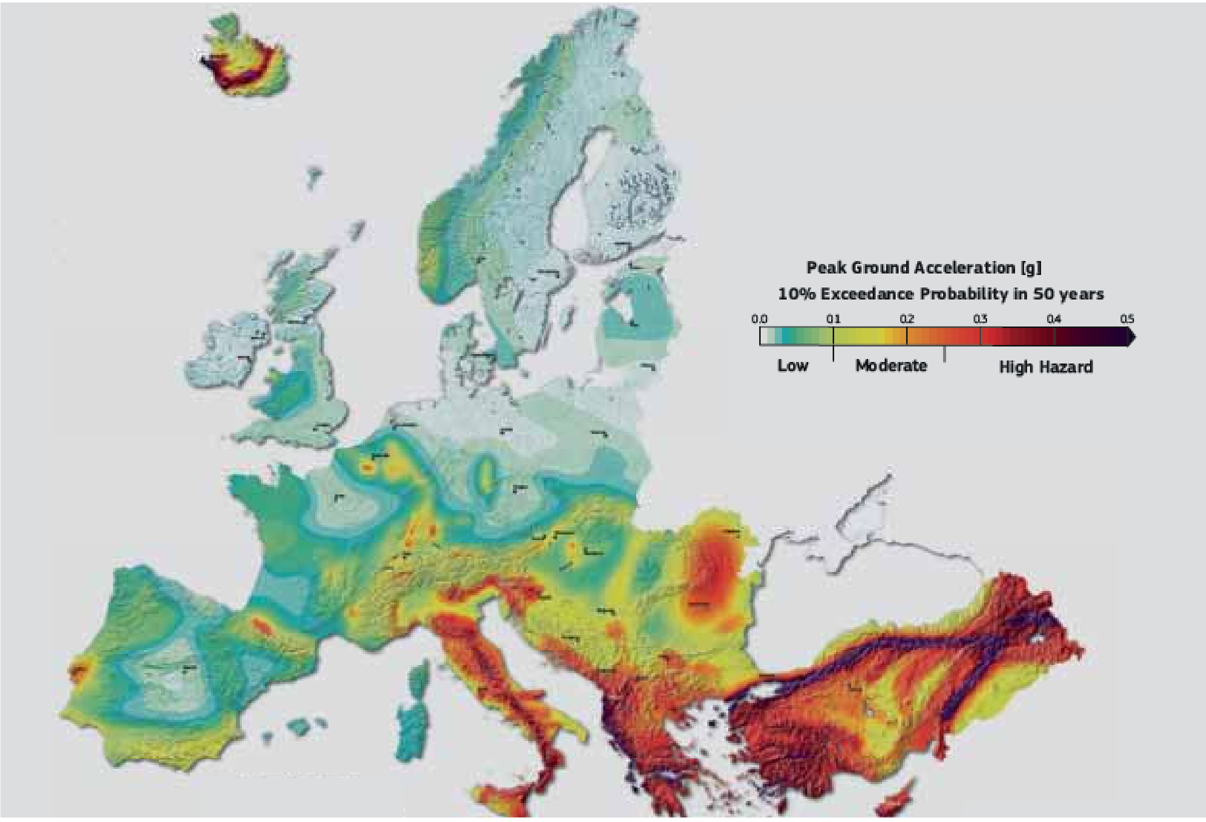


Fig. 2.10: European seismic hazard map /Giardini, D., Woessner, J., Danciu, L., Crowley, H., Cotton, F., Grünthal, G., Pinho, R., Valensise, G., the SHARE consortium (2013)/

interval, of an earthquake exceeding a given threshold of magnitude or peak ground acceleration (PGA). Nowadays, the study of seismic hazard is in continuous development, and the results are often used for the analysis aimed at defining the seismic classification of the territory (seismic zonation). The approach to the assessment of the hazard may be deterministic or probabilistic. The deterministic method is based on the study of the damage observed during seismic events that occurred in the past in a given site by recreating the damage scenarios in order to evaluate the frequency of events with equal intensity. The limit of this approach is the necessity for complete information on local seismicity and its effects. The probabilistic approach is preferable for the analysis of hazard. In this case, the hazard is expressed as the probability that in a given period of time an event with given characteristics occurs. Cornell's method (1968) /2.10/, which is the mostly used probabilistic method, consists of the identification of seismic active source for the studied area (genetic seismic zones), by quantifying the seismic activity level and computing the effects caused in relation to the distance from the epicentre. Fig. 2.10 shows the seismic hazard map of Europe, recently assessed with a time-independent probabilistic approach based on the history of earthquakes of the past 1,000 years, the knowledge of active faults mapped in the field, the style and rate of deformation of the Earth's crust from GPS measurements, and the instrumental recordings of strong ground shaking generated by past earthquakes /2.11/.

The seismic vulnerability is the aptitude of a structure to suffer a given level of damage from the effect of a seismic event of a given intensity. After an earthquake, the assessment of the building vulnerability consists in the observation of the occurred damage and associating it to the event intensity. On the other hand, the assessment of the vulnerability before an earthquake is more complex, and to this end statistical and mechanical methods have been developed. The statistical methods classify buildings according to their materials and construction techniques, by relating them to the damage observed in previous earthquakes on similar buildings. Therefore, this method

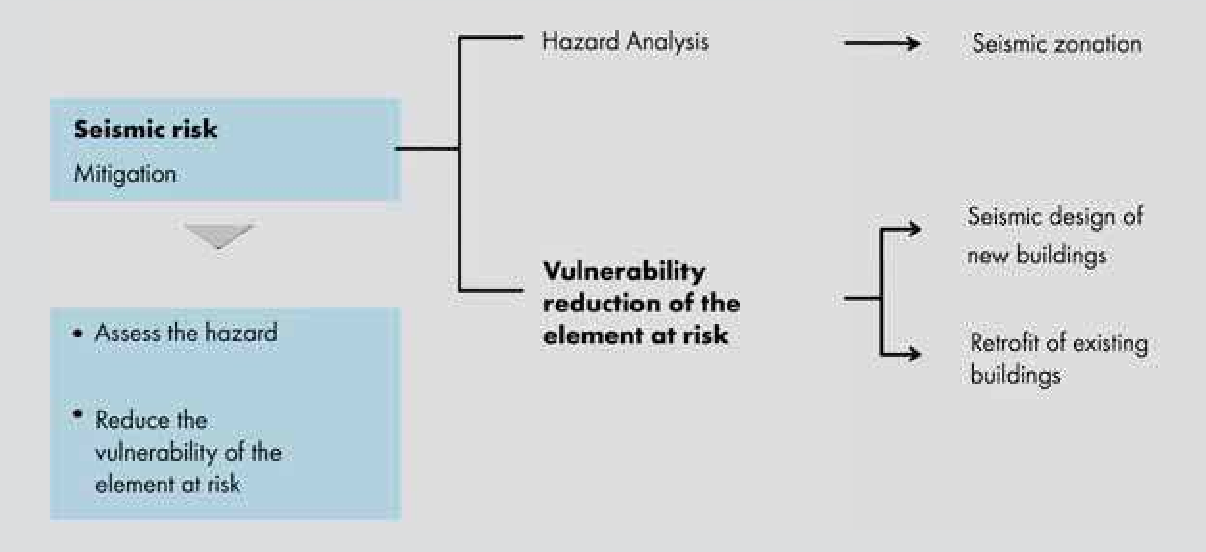


Fig. 2.11: Seismic risk mitigation strategies /Landolfo, R. (2011)/

requires information about the incurred damage of past earthquakes, which are not always available. The mechanical methods use theoretical models reproducing the main features of the buildings. The damage is assessed on the basis of simulated earthquakes.

The preservation of human life is the first objective in the control of the seismic risk, and it is very important to assess the number of people, deaths and injured involved. The reasons for loss of human life during the earthquake are not only related to the collapse of constructions, but also to phenomena triggered by the earthquake, such as landslides, soil liquefaction, tsunamis and fires. These factors are related to the concept of exposure of the elements to the seismic risk. The casualties caused by earthquakes depend heavily on the number of people present in stricken areas, and losses are also related to the quantity of the buildings, infrastructure, and other property in those areas. The exposure to the seismic risk increases as earthquake-prone regions become more densely populated and urbanized without proper local planning.

Conclusively, in some areas, earthquakes are inevitable and beyond human control, thus the seismic hazard cannot be reduced, and it is only possible to improve relevant knowledge. On the other hand, vulnerability and exposure can be supervised, and this issue should be the main objective of the programs of mitigation of the seismic risk /2.3/. In fact, the modern strategies for mitigation of the seismic risk are mainly focused on a reliable characterization of the seismic demand and capacity for different typologies of constructions (Fig. 2.11). In terms of seismic demand assessment, this approach consists of the definition of the seismicity of a given area, aimed at a quantitative evaluation of the hazard, in which the element exposed to risk is located. Instead, from a structural point of view, the mitigation strategies concentrate on the promotion of measures to reduce the vulnerability of these elements, through the application of adequate seismic design methodologies for new construction and techniques of retrofitting and upgrading existing buildings /2.12/.

2.2 Seismic design principles

2.2.1 Performance-based design

Nowadays, the strategies for seismic design are under continuous evolution and improvement. Until the 1990s, the seismic design methodologies and, therefore, the majority of the seismic codes only dealt with the safety of the occupants, avoiding the injuries or loss of life in the cases of major earthquakes. Whereas the effects on buildings due to minor earthquakes, which are the most

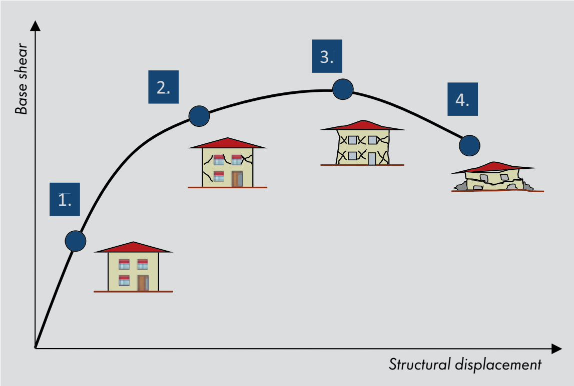


Fig. 2.12: Qualitative performance levels:
(1) Fully operational; (2) Operational;
(3) Life safety; (4) Near collapse

frequent events in the life of buildings, were generally neglected or considered in a shallow manner.

During the earthquakes that occurred in the early 1990s, such as the Northridge and Kobe earthquakes, the structures designed according to the modern seismic codes performed as expected, and the loss of human lives was minimal, compliant with the aim of the adopted methodologies. At the same time, a remarkable rise of economic losses was recorded. The reasons for these losses varied /2.3/: (a) the high density of buildings in prone seismic areas, (b) the presence of older buildings designed without any anti-seismic concept, (c) lack of knowledge about local seismic hazard and dynamic behaviour of structural systems, (d) increasingly cost related to the loss of function and business interruption in affected buildings, and (e) high damage in non-structural elements and contents.

Therefore, the effects of the earthquakes that occurred in urban areas demonstrated that the seismic risk still far exceeds socio-economically acceptable levels. Hence, there is the need to change this hazardous situation through the development of more reliable seismic provisions than those currently available and their stringent implementation for the design of new constructions and for the vulnerability assessment and upgrading of existing provisions. In order to implement a comprehensive approach, these advanced seismic provisions should not refer only to the design aspects, but must also consider all the aspects involved in the complete engineering process of construction, such as detailing, execution, monitoring and maintenance /2.13/. For these reasons, in the recent years, a new philosophy for the seismic design of construction, named "Performance-based Design", has been the object of discussion and development within the engineering community.

The birth of this new philosophy for the seismic design coincides with the proposal published by the Structural Engineers Association of California (SEAOC) in the Vision 2000 document /2.14/, which defines the main concepts of this approach. The basic concept of the performance-based design philosophy is to obtain structures that are able to achieve stated performance objectives for stated levels of seismic hazard. In particular, the approach concentrates on the selection of the performance objectives, in terms of acceptable damage levels, corresponding to different levels of earthquake intensity that could affect the structure.

Another important characteristic of this design approach is the participation of owners and users in the design process. In fact, the definition of the performance levels cannot be determined only by structural design issues, but depend also on the demands of owners, users and society. There is a minimum level of protection demanded by society, which corresponds to the safeguarding of human life. In addition to this minimum objective, society has other responsibilities, such as the continuous functionality after major seismic events of strategic structures, i.e., hospitals, communications centres, police and fire stations or safety critical facilities, i.e., nuclear plants, explosive or toxic material storages. While, in the case of important companies, the owners can demand enhanced levels of performance that allow avoidance of business interruption, with subsequent economic losses or cost reduction when repairing the sustained damage. Therefore, through this design approach, the entity of losses and damage after a seismic event of given intensity can be controlled and, when required, minimized according to the owners' demand.

Therefore, the performance-based approach is not completely new, because it can be considered as the natural evolution of limit states design that derives from the need to define, in addition to life safety requirements, the acceptable damage levels for intermediate levels of

Tab. 2.4: Earthquake design levels

Earthquake design level	Probability of exceedance	Return period
Frequent	50 % in 50 years	43 years
Occasional	20 % in 50 years	75 years
Rare	10 % in 50 years	475 years
Very Rare	5 % in 50 years	970 years

seismic intensity, by combining economic issues with the protection of human life and ensuring that, in the case of an earthquake, human life is protected, the damage is limited, and the main civil protection structures remain in operation /2.12/.

In general, a comprehensive performance-based seismic design should involve the definition of three steps /2.3/:

- Definition of the performance objective, with relevant acceptable damage levels
- Definition of multi-level appropriate design criteria, with relevant levels of seismic intensity
- Definition of the appropriate design concept and a suitable structural analysis method for each level

The first step of the performance-based design approach consists in defining an allowable damage level for a given seismic intensity. The generally accepted criteria for determining these performance levels is the adoption of a limited series of standard behavioural states /2.15/ as defined in the Vision 2000 project (Fig. 2.12):

- Fully operational: Only very minor structural or non-structural damage occurred. The building retains its original stiffness and strength. Non-structural components operate, and the building is available for normal use without any service interruption. Repairs, if required, may be done at the convenience of the building users. The risk of life threatening injury during the earthquake is negligible
- Operational: Only minor structural damage occurred. The building structure retains nearly its original stiffness and strength. Non-structural components are secured and, if utilities are available, most of them function. Life safety systems are operational. Repairs may be done at the convenience of the building users. The risk of life threatening injury during the earthquake is very low. The service interruption is less than 3 days
- Life safe: Significant structural and non-structural damage occurred. The lateral strength has still a margin against collapse. Non-structural components are secure but cannot operate. The building may not be safe for occupancy until repaired. The risk of life threatening injury during the earthquake is low. The service interruption is less than 3 months
- Near collapse: Substantial damage occurred. The building has lost most of its original stiffness and strength, having very little margin against collapse. Non-structural components may become dislodged and present a falling hazard. In case the experts decide that the building can be repaired, the service interruption is longer than 3 months. But in many cases repair is not practical

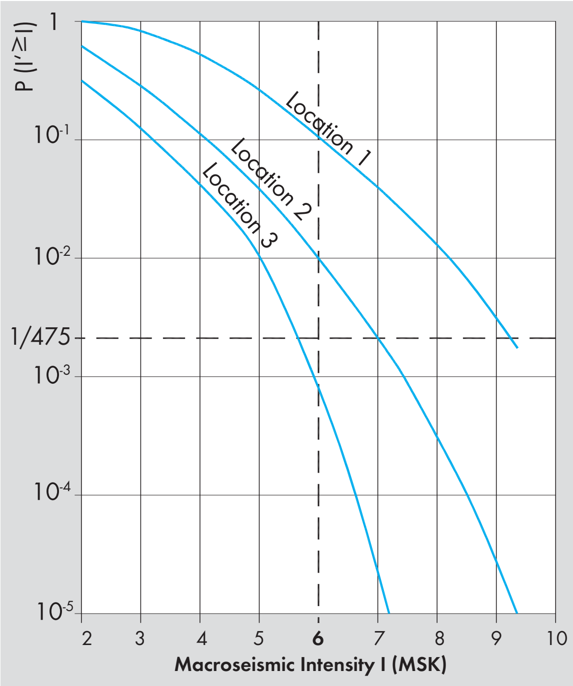


Fig. 2.13: Correlation between the probability of occurrence and intensity of earthquakes for three different locations

Similar standard behavioural states, together with useful matrices providing detailed information and descriptions

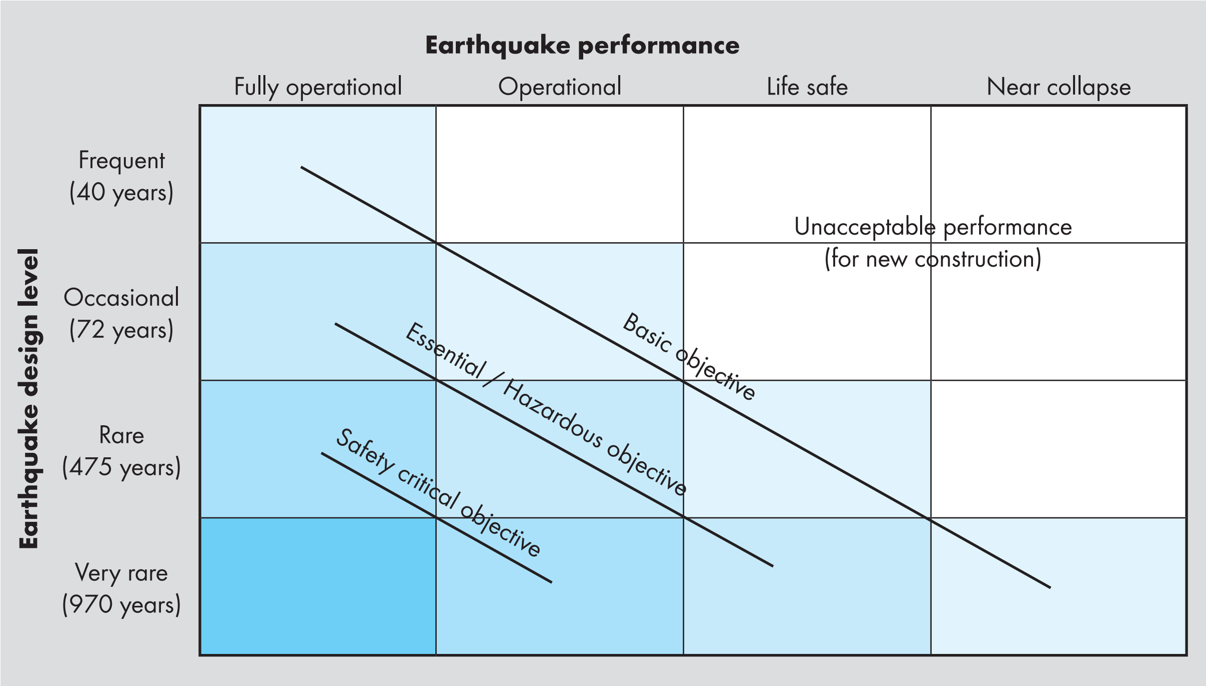


Fig. 2.14: Performance objective matrix, recommended in SEAOC /SEAOC (1995)/

of the different damage levels for structural and non-structural components of typical building systems, are also proposed by ATC-33.03 /2.16/, FEMA 273 /2.17/ and FEMA 356 /2.18/ documents.

The second step concentrates on defining the multi-level appropriate design criteria for the different performance objectives. Different earthquakes with different intensities (low, moderate and severe) can affect a building inflicting slight, moderate or heavy damage or, in many cases, the collapse depending on the earthquake intensity. The intensity of earthquakes is generally related to the probability of occurrence and, in particular, rarer events correspond to more severe earthquakes. The earthquake design levels proposed by Vision 2000 defined through their probability of exceedance and their return period are listed in Tab. 2.4.

These four earthquake design levels are related to the four performance levels defined above through the performance objective matrix shown in Fig. 2.14 /2.19/. The diagonal lines on the matrix represent the performance target for different levels of building importance. In particular, levels of building importance are "basic", "essential" (hospitals and civil protection facilities), "hazardous" (containing hazardous materials, but of confined impact), and "safety critical" (containing explosives and radioactive materials). The minimum required objective for seismic design is represented by the "basic" level. All the performances below this minimum objective have to be considered unacceptable. Enhanced performance objectives are reached in the case of more important structures or for specific owners' requirements. Obviously, more stringent requirements and enhanced objectives entail more expensive or uneconomic structures. In general, the performance level, as well as the expected damage, increases for more severe and less likely earthquakes. On the contrary, for the same probability of earthquake occurrence, a more critical or important building suffers a minor amount of damage.

The third step consists in selecting a coherent strategy for seismic design, through suitable structural analysis methods and verifications for each performance level /2.20/.

Elastic analyses of the structure are generally adopted for serviceability limit states (fully operational and operational) under minor earthquakes, for which the integrity of the structure should be assured and non-structural components could suffer minor damage. Therefore, the interaction between the main structure and the non-structural elements should be considered, while rigidity verifications are carried out in terms of inter-storey drifts compatible to non-structural elements.

In the case of damageability limit states (life safety) under moderate earthquakes, an elasto-plastic analysis must be performed. The non-structural elements are generally damaged, and their interaction with the structure is not considered. The verifications are performed in terms of members' strength, while rigidity and ductility verifications are optional.

For survivability limit states (near collapse) under severe earthquakes, the optimal strategy consists in performing kinematic analysis. Thus, it is possible to consider the global structural behaviour due to the formation of a plastic mechanism. The verifications refer to the ductility in terms of mechanism control and rotation capacity of plastic hinges. Strength verifications are optional.

Therefore, in the design approach of performance-based design, the fundamental aspect consists in identifying and evaluating the performance capability of a building. This aspect also influences many design decisions. The key phases of the performance-based design process are depicted in the flowchart of Fig. 2.15 /2.21/. The design process is iterative, and it starts with the selection of the performance objectives. After this, a preliminary design is developed, and the building performances are assessed and compared to the objectives. If performance does not meet the objective, the building is redesigned and reassessed until the desired performance is achieved.

2.2.2 Codification

The seismic design concepts introduced by the performance-based design philosophy are attractive, but their implementation has a long way to go. The promise of engineered structures whose performance can be qualified and conformed to the owner's desires may be fully unrealistic, because it is not possible to confidently predict all important seismic demands and capacities /2.22/. In addition, the behavioural performance parameters are meaningful to owners and users, but they are not useful for a practical design. Therefore, a relationship between the behavioural parameters and design procedure is introduced in the codes through the definition of a minimum level of protection, aimed at safeguarding human life and reducing the economic losses /2.3/.

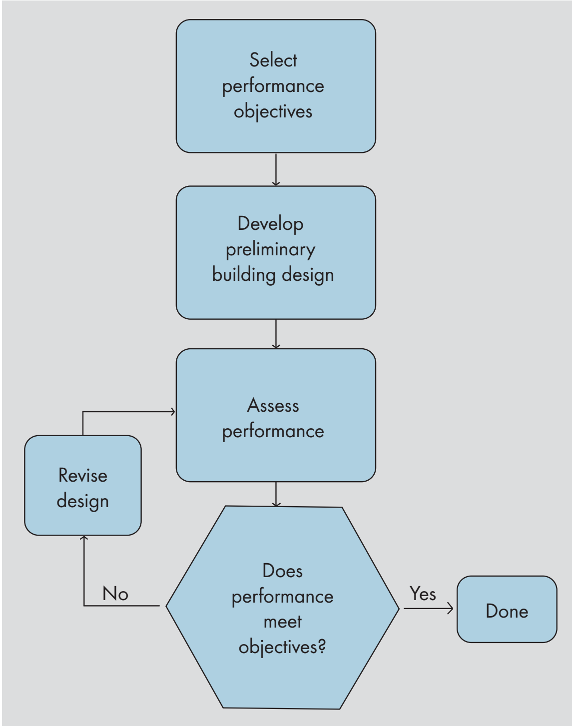


Fig. 2.15: Performance-based design flowchart /FEMA/

In this perspective, the Vision 2000 report /2.14/ suggested four minimum levels, but, in practice, this methodology can be followed only for special cases, such as important buildings or special owners' demands. For current cases, the good seismic behaviour of a building should be ensured through a multi-level design that considers two or maximum three levels. This is because a specific analysis has to be performed for each level, and an excessive supplementary design effort may not be accepted by the design professional community. In order to take into account different limit states or performance levels of the structure (Fig. 2.16), a different multi-level approach is possible /2.23/:

- One design level. The structures are designed only for an ultimate limit state (ULS), that is for the prevention of the structural collapse. This approach was adopted, for a long time, in many codes provisions
- Two design levels. The structures are designed for a damage limit state (DLS) and an ultimate limit state (ULS). For DLS, the structures are designed to remain elastic, and the non-structural elements are undamaged or suffer minor damage. While, for ULS, the plastic capability of the structure is exploited, and

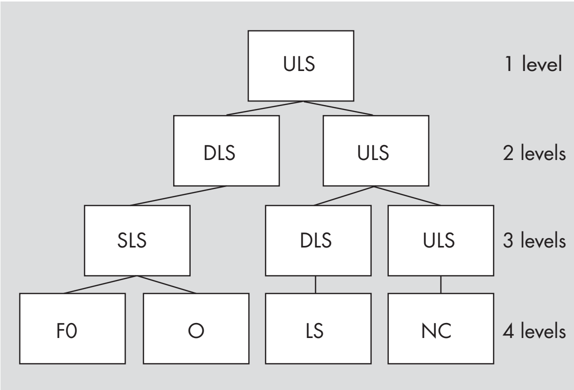


Fig. 2.16: Evolution of multi-level approach /Truta, M., Mosoarca, M., Gioncu, V., Anastasiadis, A. (2003)/

the non-structural elements are partially or completely damaged. This is the methodology currently adopted by seismic codes

- Three design levels. The structures are designed for a serviceability limit state (SLS), a damageability limit state (DLS) and an ultimate limit state (ULS). For SLS, structural parts are undamaged, and non-structural parts could suffer minimum damage. For DLS, the damage can occur in non-structural elements, while the structures suffer moderate damage that can be repaired without technical difficulties. This is the objective of new generation codes
- Four design levels. The serviceability is divided into a full operational limit state (FO), in which the facilities remain in operation, and an operational limit state (O), in which the facilities can be immediately resumed. Life safety (LS) corresponds to damage of structural and non-structural parts ensuring the accessibility for emergency activities, while for near collapse (NC), the collapse is prevented and the structure can support only the gravity loads. This is the approach of Vision 2000, and it can be noted that it substantially corresponds to the three design levels, with the serviceability limit state split into two (FO and O)

The development process of performance-based design guidelines started in the United States in response to seismic design problems highlighted by earthquakes of the early 1990s, with particular reference to the defined assessment criteria for existing structures. During that period, several efforts were made, more or less in parallel, with the aim of facing up to the challenge of this new design philosophy. The result was the publication of several documents, such as Vision 2000 Report, Performance-based seismic engineering of buildings /2.14/, FEMA 273 NEHRP guidelines for the seismic rehabilitation of buildings /2.17/, together with its companion document FEMA 274 NEHRP commentary on the guidelines for the seismic rehabilitation of buildings /2.24/ and ATC-40 Seismic Evaluation and Retrofit of Concrete Buildings /2.16/. The Vision 2000 Report describes the framework for the design of buildings according to multi-performance objectives. The FEMA 273 and 274 are documents related to the seismic upgrade of existing buildings, while the ATC-40 document provides guidelines for retrofitting existing concrete buildings.

These documents can be considered as the first generation of performance-based procedures. They present only some differences in notation and terminology, but the basic framework is conceptually very similar. The proposed procedures introduce the basic concepts of discrete defined performance levels by linking these levels to a specific level of seismic hazard for both structural and non-structural elements. Furthermore, these documents also provide information about the different possible analytic procedure for simulating the seismic response of buildings, with particular reference to non-linear procedures and acceptance criteria. Accordingly, these documents represented the first important step towards performance-based design and improvement of the procedure provided by the old seismic codes.

The second and current generation of performance-based design procedure is basically contained in the FEMA 356 Pre-standard and commentary for the seismic rehabilitation of buildings /2.18/. The main objective of this document is to encourage and develop the wider application of the FEMA 273 contents, by converting it into mandatory language, in order to provide a more specific document for earthquake resistant buildings to design practitioners. The main developments introduced by the FEMA 356 are technical updates to the analytical procedure and acceptance criteria of previous documents. These updates are carried out on the basis of the information obtained by the practical design application of these procedures and the advances in the research studies.

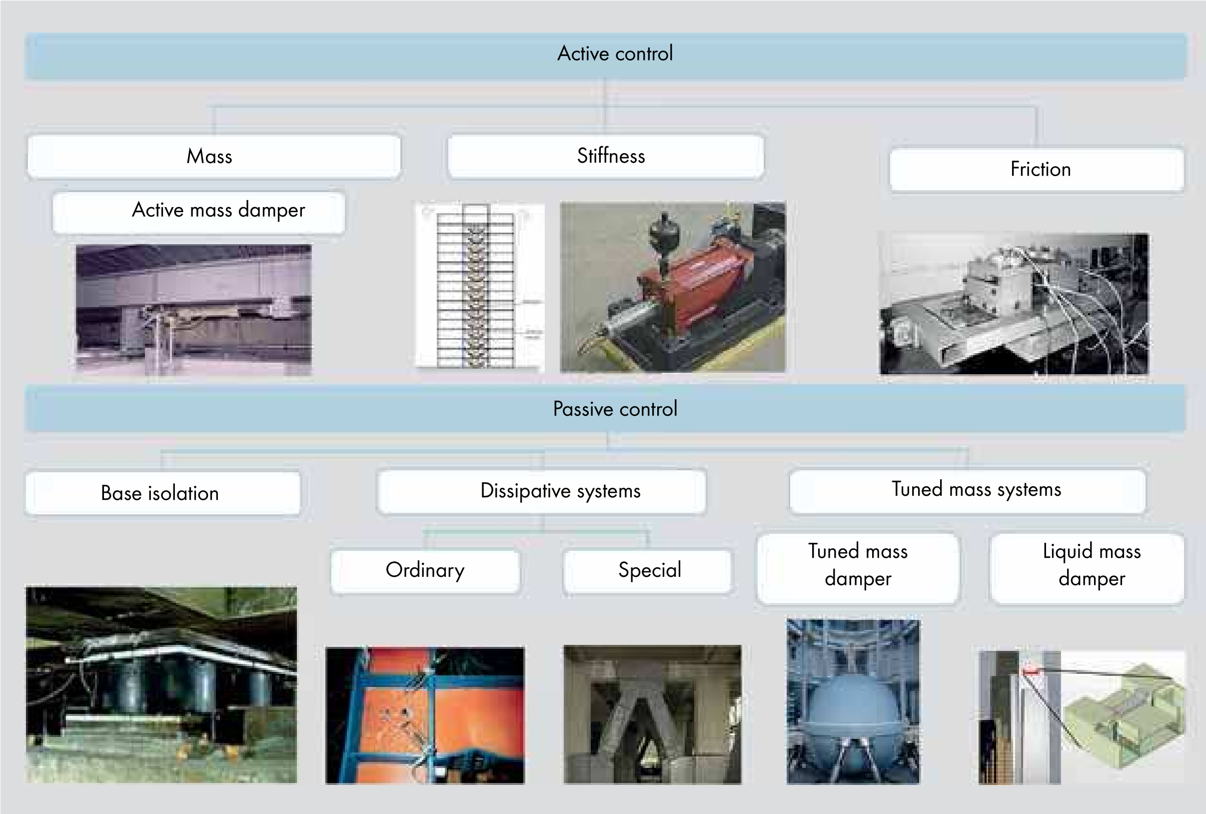


Fig. 2.17: Seismic design strategies /Landolfo, R. (2009)/

The development of these second generation procedures entailed more familiarity of the engineering professionals with the performance-based design concepts, making very common the use of advanced non-linear techniques for the evaluation of the seismic response of buildings. Nowadays, the performance-based procedures are under continuous development and, on the basis of the knowledge and experience achieved by researchers and designers, the studies for the next generation of procedures are in progress. These studies are focused on the more accurate analytical procedure for predicting a building's response, the reduction of conservatism in the actual acceptance criteria and the improvement for a more reliable and economical design for new buildings /2.21/.

The introduction of the performance-based procedures caused a very large change in the approach to seismic codes. The old seismic codes are prescriptive and provide specific provisions and methods that must be strictly followed to achieve the final result. Therefore, the old prescriptive codes are concerned with "how the building is built", providing a series of restrictions, while modern performance-based codes are concerned with "how the building behaves", allowing any possible solution provided that compliance with the performance requirements can be demonstrated /2.25/.

2.2.3 Life safety limit state

The verification of ultimate limit state (ULS) involves performing a series of structural checks, generally in terms of strength, with the aim of avoiding the structural collapse of the building and ensuring the safety of inhabitants for seismic events having a low probability of occurrence. In this case, it has to be noted that, for zones of medium to high seismic hazard, the structure can be subjected to a maximum horizontal acceleration, as provided by the elastic response spectrum, and an even greater acceleration of gravity. In order to face up to such high intensity horizontal acceleration, the modern design strategies consider several design solutions for achieving seismic-resistant structural systems. The various possible systems basically differ in how they behave under the dynamic excitation induced by the earthquake.

In general, it is possible to classify the seismic-resistant

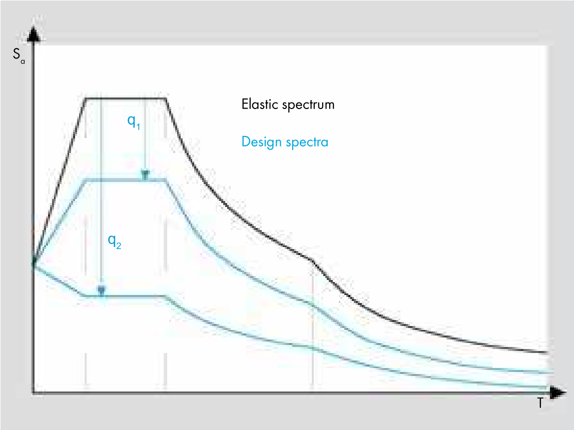


Fig. 2.18: Behaviour factor q

systems as follows /2.12/: Active control systems, which essentially operate on the dynamic properties of the structure by changing the structural response in an artificial way; passive control systems, which are designed by increasing the energy dissipation capacity of the structure; and iper-resistant systems, which are designed to withstand the earthquake actions remaining in the elastic range, without suffering any structural damage (Fig. 2.17).

Leaving aside the active control systems and particular passive systems, such as base-isolation and tuned mass, which represent innovative and/or more sophisticated solutions (see Section 2.3.2), the ordinary seismic-resistant structures can be essentially divided into two families:

- Non-dissipative structures
- Dissipative structures

The non-dissipative structures, also known as iper-resistant, are designed to remain in the elastic range, without suffering any structural damage, not only during frequent seismic events, having a return period comparable with the service life of the structure, but also in the case of destructive earthquakes with a low probability of occurrence. Such types of structures react for mass and are characterized by very resistant members, which are subject to an elastic regime of stress also under major earthquakes. This behaviour entails that the accumulated elastic strain is completely released without any trace of residual deformations. To this end, it is also necessary to provide the structure with a high rigidity involving, in most cases, oversized structural elements and thus uneconomic solutions. Non-dissipative structures are generally used in the case of buildings and plants intended for particular strategic functions, where the non-damageability for the ultimate limit state is also clearly a design requirement. This is also the case with structures, for which the capacity of dissipation in the plastic range is not exploited, either by design choice or for the inability of the structural system to ensure that behaviour. Therefore, the design of such systems for the ultimate limit state has to be conducted by carrying out only strength verifications, because it is not necessary to satisfy any ductility requirements, and the evaluation of the seismic actions is referred to the elastic response spectrum.

In the case of dissipative structures, the design is based on the principle for which part of the seismic input energy is dissipated by hysteresis due to the plasticization of some specific elements dedicated to this purpose, by avoiding the brittle fractures and the occurrance of unexpected instable mechanisms. The zones of the structure devoted to the absorption of the seismic energy and intended to undergo plastic deformations are concentrated in specific elements or parts of elements, by retaining all the other structural parts under an elastic regime of stress.

In order to achieve dissipative seismic-resistant systems, the structure has to be able to exploit available resources beyond its elastic limit or, in other words, to ensure a global ductile behaviour. The concept of structural ductility plays a very important role, and, together with the resistance, it represents a fundamental requirement to ensure and to be pursued at different levels, as explained in the following. Therefore, the design of such systems can be conducted by considering a reduced value of the seismic actions in proportion to the potentially available ductility of the structure. The reduction of the design seismic forces with respect to the elastic ones is achieved through the introduction of a reduction factor named "behaviour factor" (q) or "response modification factor" (R), according to the European or American terminology, respectively (Fig. 2.18).

This factor, which represents a quantitative measure of the energy dissipation capacity of the structure, is a key parameter for the design, and it can be obtained through conditions of kinematic or energetic equivalence. In addition, it should be noted that the apparent advantage of

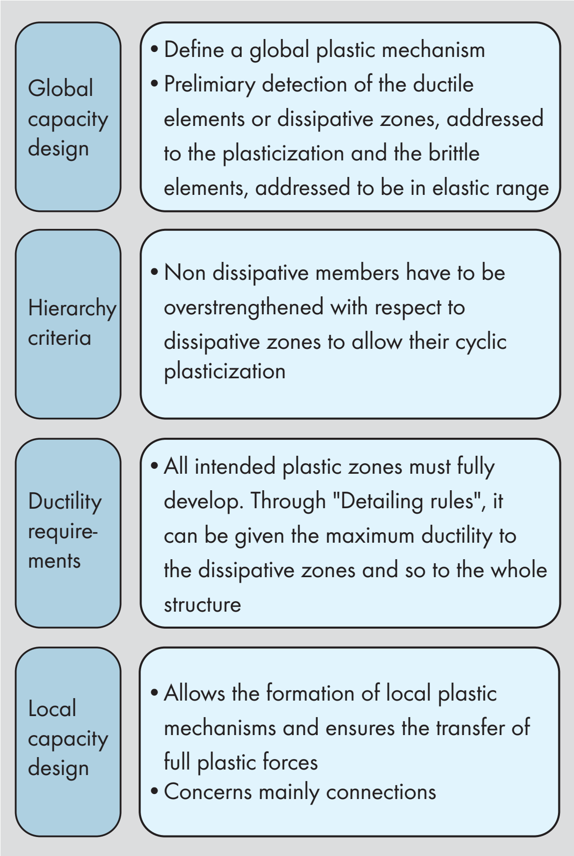


Fig. 2.19: The main steps of the design procedure in accordance with the capacity design

the reduction of seismic design forces is counterbalanced by more complex criteria and design rules different than those related to the elastic design of a structure. In fact, in this case, the performance target to be ensured is defined in terms of ductility rather than strength.

From the practical point of view, in the case of analysis of dissipative structures at ULS, the main international seismic codes provide that the design spectral acceleration, and thus the seismic action, is obtained by reducing the ordinates of the elastic response spectrum by the behaviour factor (q). In general, the values of this factor are provided by codes in tabulated format, and they depend on the construction systems (e.g. reinforced concrete, steel or masonry buildings), the structural typology (e.g. MR frames, walls or bracings), the building regularity and the local ductility (ductility class).

The basic objective in the design of dissipative structures concentrates on realizing several zones able to ensure a reliable dissipative behaviour. The design process can be articulated in three main phases (Fig. 2.19):

- Define a global plastic mechanism (selection of the structural typology and the relevant global collapse mechanism)
- Design ensuring the adequate ductility and reliability to the selected dissipative zone
- Avoid any plastic deformation, brittle failure or elastic buckling in the structural elements which do not belong to dissipative zones

Therefore, the ULS design is performed by an appropriate pre-identification of the dissipative zones (or ductile elements), which are devoted to plasticization. For these zones, it is necessary to ensure the fulfilment of appropriate performance requirements in such a way as to give a good response in terms of dissipative behaviour. Thereafter, for the remaining parts of the structure (non-dissipative zones or brittle elements), an adequate overstrength with respect to dissipative zones must be ensured, so that these parts can react to the actions transmitted during an earthquake remaining in the elastic range. According to this principle, in the design phase, a differentiated resistance is assigned to the different structural elements, so that the failure of ductile elements can prevent that of fragile elements. This principle is internationally known as capacity design (Fig. 2.20). Basically, the ductile elements must be less resistant than the brittle ones, acting as structural fuses and protecting the whole structure. On the other hand, in order to allow the development of cyclic plasticization in dissipative zones, the non-dissipative members have to possess an adequate overstrength with respect to the dissipative ones.

It is clear that the design methodology is based on two stages. The first one focuses on the zones identified as responsible for the hysteretic dissipation and, for this reason, these zones have to satisfy the requirements of strength, stiffness and ductility to allow the development of large excursions in the plastic range. At the same time, through the capacity design requirements, the non-dissipative members have to be sufficiently over-resistant, by ensuring the plasticization of the zones devoted to the dissipation. Therefore, if from one side the formation of collapse mechanisms with a large number of plasticized zones is encouraged (global mechanisms) with the aim of dissipating the greater part of the seismic input energy, on

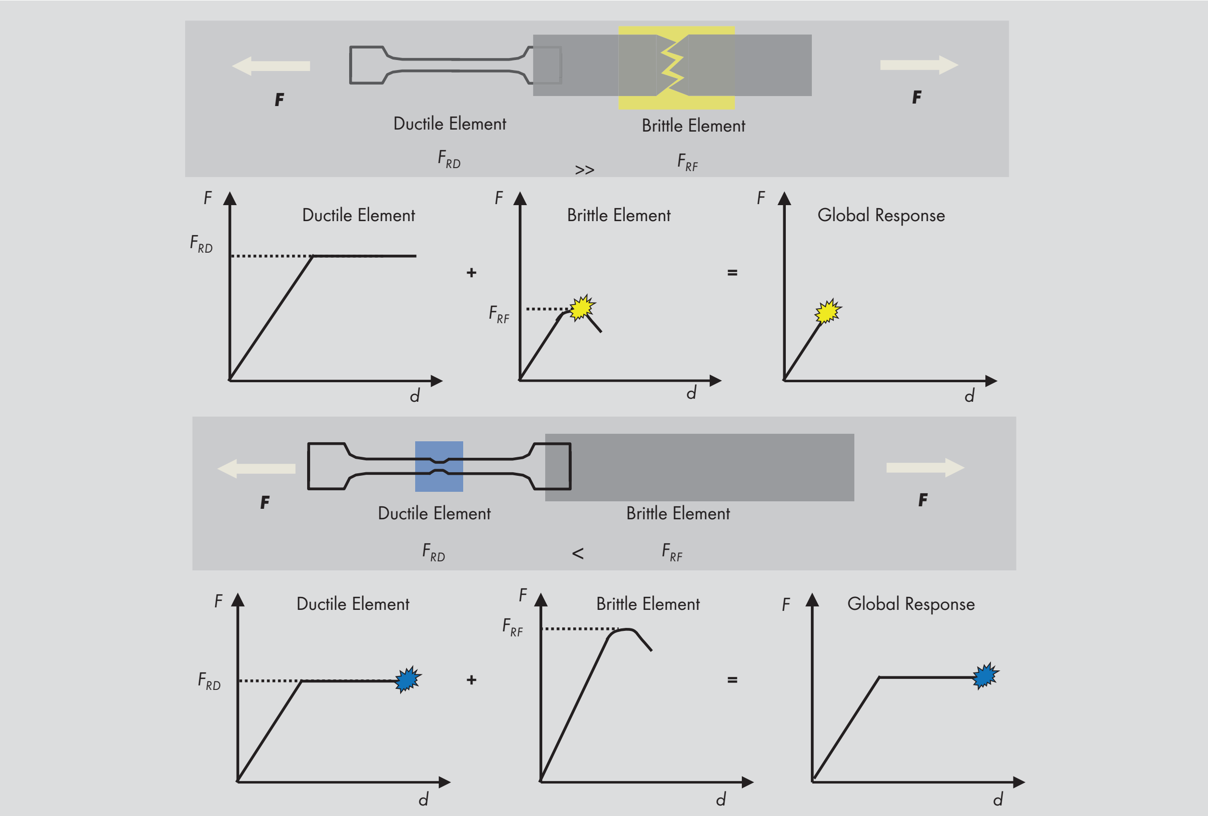


Fig. 2.20: The concept of capacity design /Landolfo, R. (2009)/

the other hand, a stable global response has to be also ensured in the presence of local plasticization, buckling or other phenomena related to the hysteretic behaviour of the structure.

Although a dissipative structure represents a competitive and rational solution for withstanding earthquake actions, it is important to highlight that its seismic design inevitably entails more complicated procedures, involving greater care in the selection and verification of the structure. For each structural level, it is necessary to identify all the possible failure mechanisms and, among them, the ductile and brittle ones, in such a way as to provide an adequate overstrength for the fragile mechanisms. From a practical point of view, the concept of capacity design is expressed in structural codes through a series of provisions and requirements. In the following, the basic approach for the capacity design of typical solutions for reinforced concrete and steel structures are briefly illustrated.

The typical seismic-resistant reinforced concrete structure is made with frame systems. The most ductile mechanism for this typology basically consists in the development of flexural plastic hinges at the ends of the beams (Fig. 2.21). At the material level, the ductility is provided by the steel reinforcement bars and, then, the failure mechanisms that involve this material have to be promoted. Therefore, seismic codes prescribe the use of ductile steels having the ratio between the actual and nominal yielding strength within a given limit. Also, the cross-sections, which are substantial in bending, have to be designed to promote a ductile behaviour. In particular, lightly reinforced cross-sections, which fail with large deformations of rebars and low stresses in concrete, are preferable because of the higher values of curvature corresponding to the ultimate condition. The codes generally obtain these conditions by imposing an upper limit of reinforcement ratio of the tension zone, and, in order to avoid brittle failures due to the cracking of very lightly reinforced cross-section, also a lower limit is defined for the reinforcement ratio of the compression zone. In general, the rupture of a beam or a column member can occur due to bending or shear. If the rebars are correctly designed, the failure of the members is ductile and due to bending, while the shear failure is brittle and should be avoided according to the capacity design. This implies that the members should be designed

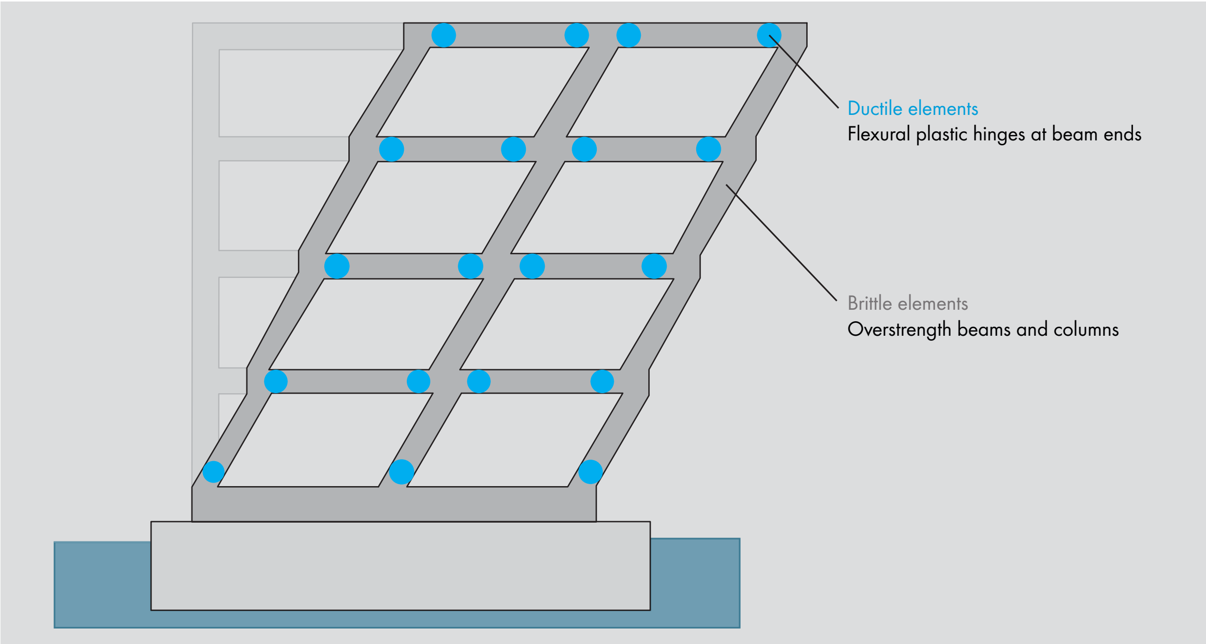


Fig. 2.21: Capacity design for reinforced concrete frames

for the maximum possible shear, corresponding to the vertical loads and the presence of the resisting moment of the member at the ends. The capacity design at a whole structural level promotes a global failure mechanism, in which the plastic hinges formation occurs at all the beam ends and at the column bases. The codes attempt to achieve this mechanism by prescribing that the columns and the beam-column joints have to have an adequate overstrength with respect to the bending failure of the beam /2.26/.

In the case of the design of steel structures, there are different possibilities to obtain seismic resistant systems, which are classified with reference to their behaviour against horizontal actions and to the specific elements designated for dissipating seismic energy. One of the most common structural solutions is the use of concentrically braced frames. In this earthquake resistant system, the dissipation of seismic energy input is assigned only to the diagonal braces, which can be arranged in X or V configuration. The system dissipates energy through the braces plasticization under an axial regime of stress (Fig. 2.22). Also in this case, the capacity design has to be ensured at different levels. For the material, the codes generally provide particular requirements for the ductility and the hardening of the used steel material. At the cross-section level for dissipative elements, the use of sections belonging to "ductile" or "compact" classes is required, which do not suffer from local stability (e.g. buckling) effects, by allowing a greater deformation capacity. Stability issues also affect the choice of the dissipative members (braces). In fact, due to the occurrence of stability issues, the dissipation capacity of the diagonals in compression is certainly lower than those in tension, and depends on the global slenderness of the diagonal. Therefore, the codes provide limitations on the slenderness values of the diagonal members. In order to attempt the achievement of the desired global mechanism, non-dissipative members (beams, columns and connections) have to be designed to remain in the elastic range, by providing a sufficient overstrength. In particular, the capacity design engages in designing beams and columns, so that axial resistances are higher than the axial forces corresponding to the plasticization of the diagonals. Instead, the resistance of the connections have to be at least equal to those of the weakest members connected /2.27, 2.28/.

2.2.4 Damage limit state

The damage limit state (DLS) is generally verified, after the structure has been preliminarily designed at the ULS. The verification for the DLS is usually performed by checking that the seismic actions corresponding to

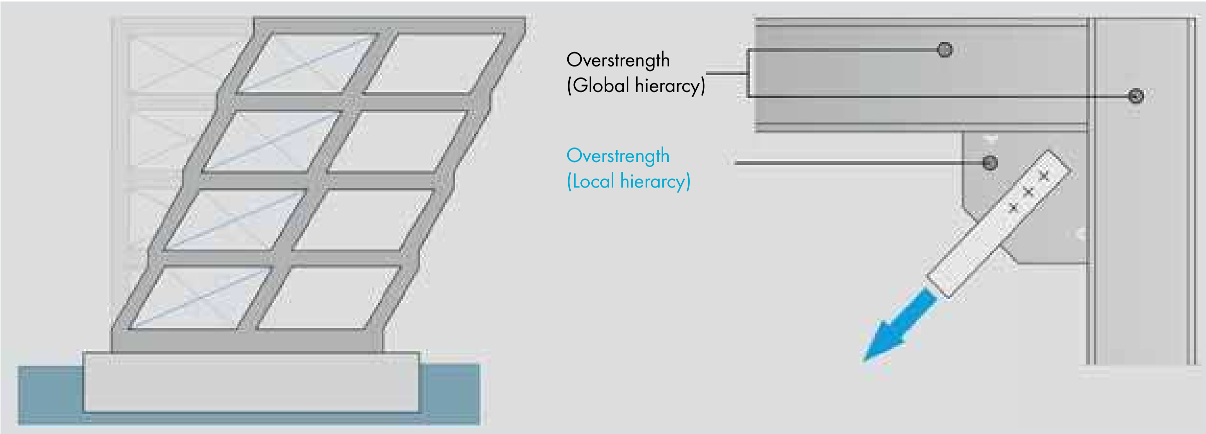


Fig. 2.22: Capacity design for steel concentrically braced frame

frequent low-intensity earthquakes induce displacements compatible with the normal functionality of the structure. Therefore, the seismic actions corresponding to frequent or occasional earthquakes must cause very limited damage in the elements without structural function, such as linings, partitions, ceilings, elevators and plants.
The required verification generally consists of checking that the building lateral deformations, in terms of inter-storey drift, are contained within given limits under earthquakes with return periods in the range between 40 and 75 years. In particular, for DLS verifications the effects of seismic actions are evaluated on the basis of elastic spectra. It should be noted that for some structural typologies characterized by a high deformability (e.g. steel frames), the verification of the DLS can influence the design more than the ULS. In this case, it should be noted that specific values of the inter-storey drift limit, according to the type and the properties of the non-structural element and its connection with the structure, can be advantageous to achieve reliable design criteria. The strength verification of structural elements for this limit state is usually prescribed only for strategic structures.

2.3 Modern strategies for seismic design

2.3.1 Modern design methods

The main codes for seismic design currently apply force-based design procedures for the calculation of the structures. These procedures are in widespread use in the engineering community, but, although they have been significantly improved in recent years, force-based design approaches present several basic deficiencies in their application. An important problem related to the force-based design is the interdependency of strength and stiffness, which corresponds to the impossibility to assess the stiffness of the structure, and hence also the fundamental period and the distribution of the design forces, until the structure is fully defined (Fig. 2.23). Other problems include invalid assumptions for the relationship between elastic and inelastic displacements, simplistic definition of behaviour factors for whole structural types without considering the variability of ductility capacity within a structural class and inadequate representation of the response of dual systems /2.29/. Despite these problems and critical points, the force-based design method is able to provide safe design, when it is applied together with the capacity design principles, and the structural details are carefully studied.
Furthermore, force-based design methods do not seem the most suitable approach to modern philosophy as performance-based design, which is based on the achievement of given limit states derived on the displacement capacities of structural and non-structural components. In fact, force-based design approach gives a secondary importance to the displacement limit states, which are checked only as a final step of the design procedure /2.30/.

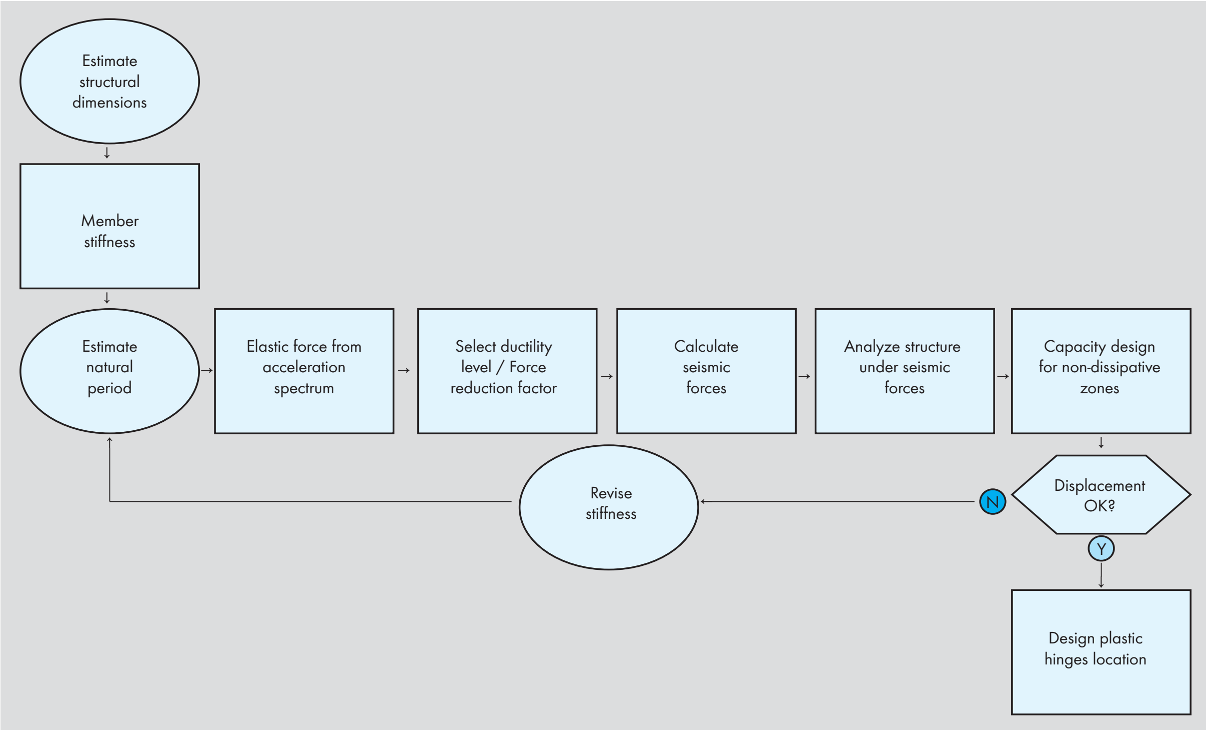


Fig. 2.23: Procedure for force-based design /Priestley, M.J.N., Calvi, G.M., Kowalski, M.J. (2007)/

An alternative to the conventional force-based design methods is the direct displaced-based design /2.31/, which is conceptually aimed to the modern performance-based philosophy and appropriate to address the problems related to the existing approach. The direct displacement-based design fundamentally differs from the traditional force-based design for the concept of substitute structure. In practice, the structure to be designed is simulated by a single-degree-of-freedom (SDOF) system representative of the performance at peak displacement, instead of the initial elastic condition as used for traditional methods. This approach attempts to design in such a way as to achieve the desired performance for a given seismic intensity, perfectly in line with the performance-based philosophy. The application of the design procedure allows calculation of the required strength for the zone designated for the development of the plastic hinges corresponding to the attainment of the design displacement objective. Then, in order to ensure the development of the plastic hinges only in the desired zone, the capacity design procedure must be applied. In this case, the requirements for capacity design are less onerous than the traditional approach, with consequent more economical resulting structures.

The key steps of direct displacement-based design procedure are summarized in Fig. 2.24. The structural system to be designed is schematized by an equivalent SDOF system with equivalent mass (m_e) and height (H_e) (Fig. 2.24a). The SDOF system response is characterized by the secant or effective stiffness (k_e) at peak or design displacement (Δ_d) (Fig. 2.24b). The latter is set according to the deformation limit state corresponding to the required performance level. At this point, in order to define the expected displacement ductility demand (Δ_d/Δ_y), the yield displacement (Δ_y) of the SDOF system is calculated on the basis of the yield curvature of the structural elements. Starting from the displacement ductility demand, the equivalent viscous damping (ξ) is estimated from the appropriate relationship calibrated for the expected hysteretic response of the system typology. The equivalent viscous damping takes into account both elastic and hysteretic energy dissipation of the system and, for a given ductility demand, higher damping values correspond to systems with broader hysteresis response characteristics (Fig. 2.24c). Knowing the design displacement (Δ_d), it is possible to evaluate the effective period (T_e) at maximum displacement response on the displacement capacity spectrum corresponding to relevant equivalent viscous

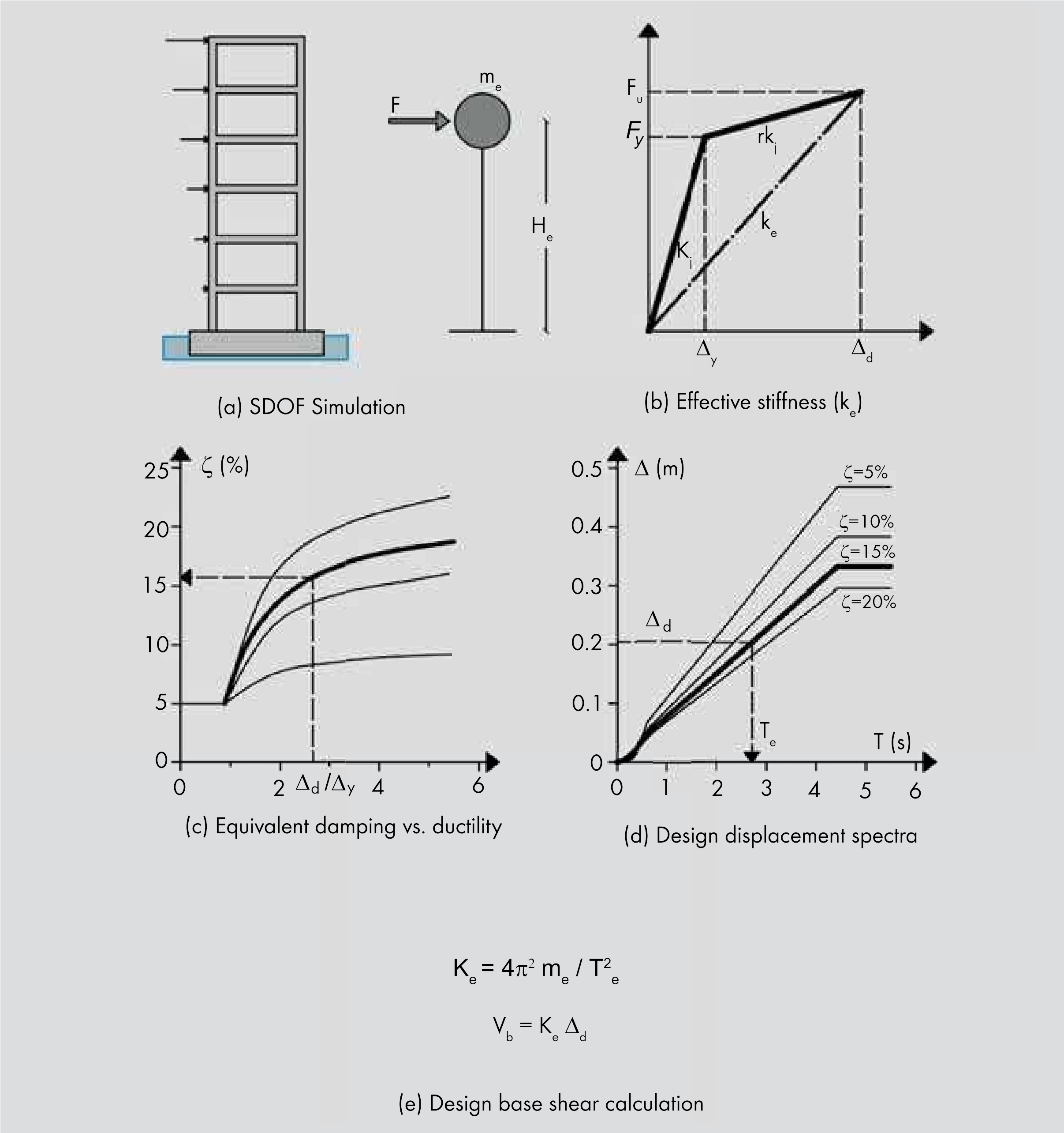


Fig. 2.24: Procedure for direct displacement-based design /Priestley, M.J.N., Calvi, G.M., Kowalski, M.J. (2007)/

damping (Fig. 2.24d). In this way, the effective stiffness (k_e) can be calculated through the classical equation for the period of an SDOF oscillator, and the design shear base force (V_b) is obtained by multiplying this stiffness by the design displacement (Fig. 2.24e). The resulting design base shear can be considered as the system strength required to achieve the objective displacement for the considered seismic intensity.

As a conclusion, the design concept is thus very simple. The complexity of the method is related to the definition of the equivalent SDOF system, the determination of the design displacement, the development of design displacement spectra and the distribution of the design base shear force to the different systems masses.

2.3.2 Innovative system solutions

Structures located in prone seismic areas have the task of absorbing and dissipating the energy input caused by the earthquake through the damping and the development of inelastic deformations. This requirement has led to the development of specific seismic resistant structural typologies /2.3/. With this in mind, the researchers

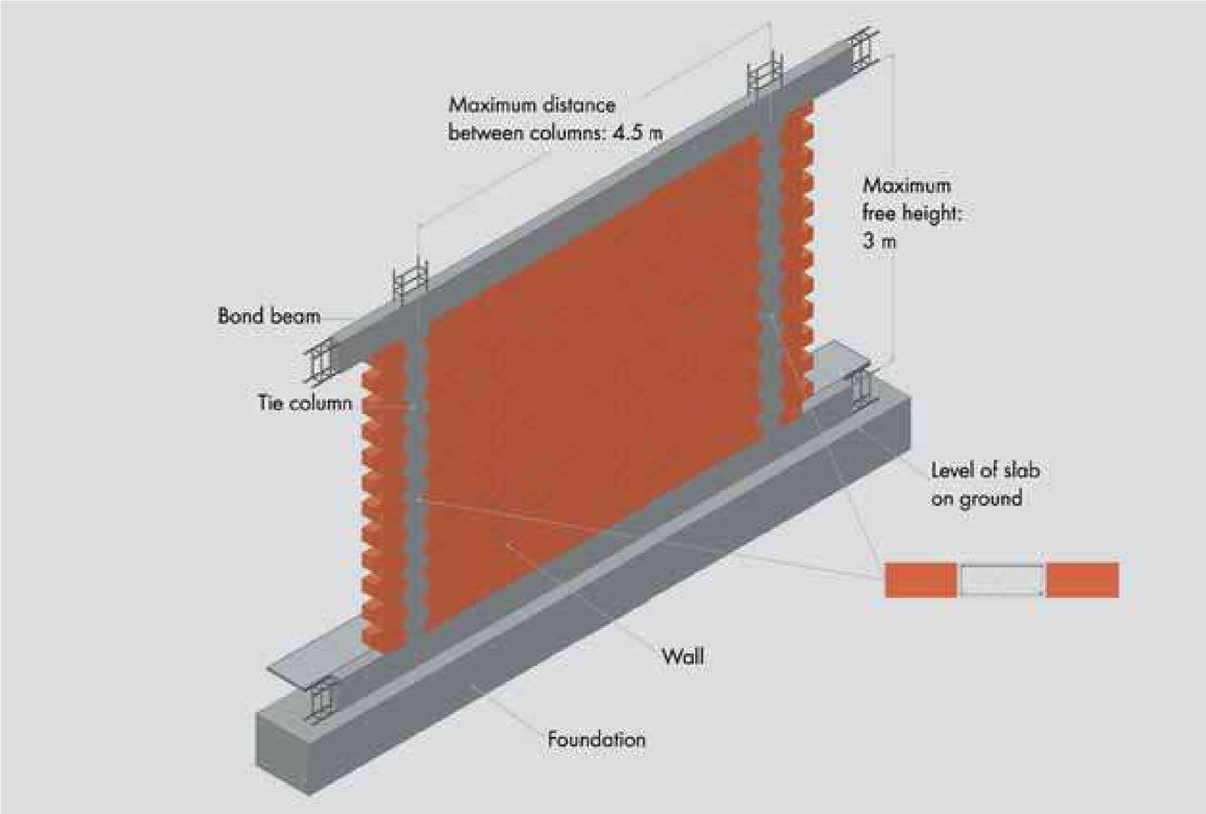


Fig. 2.25: Confined masonry

and the industry are constantly and actively involved in the improvement and optimization of the performance of traditional seismic structural typologies and in the development of more innovative solutions /2.32/.

In general, a good seismic response is essentially related to materials having adequate ductility and dissipative systems able to exploit this ductility. In the following, the main development of the last decades in terms of materials and innovative structural systems are discussed. Among the various materials, masonry buildings are the most ancient system, and their seismic vulnerability is well known. In order to improve their behaviour, modern buildings are based on the concept of reinforced masonry. A common typology of reinforced masonry used in seismic applications is the reinforced grouted cavity masonry. The system consists of two masonry walls built with a small clearance to produce a cavity, in which a steel rebars grid is placed and then filled with cast concrete. Another typology is the confined masonry, which is a masonry wall confined on all edges by reinforced concrete elements, i.e. bond-beams and tie-columns. Tie columns are always located at wall intersections and corners to achieve an effective confinement of the masonry walls (Fig. 2.25).

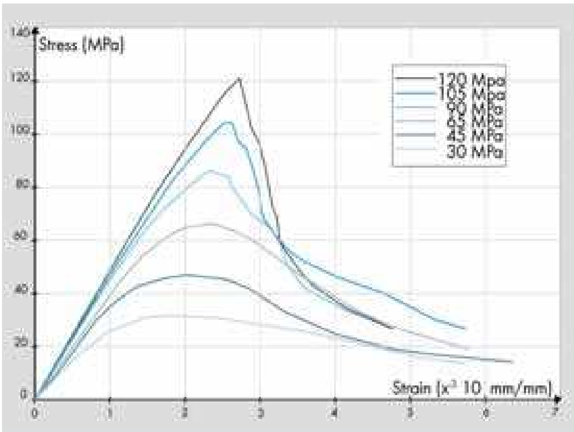


Fig. 2.26: Stress-strain curves for high-strength concrete

The use of concrete in seismic resistant structures has two important limitations represented by the low structural efficiency (strength to weight ratio) and the very low tensile strength. These deficiencies can be overcome through the introduction of high-performance concrete having enhanced mechanical properties. As an example, high-strength concrete is often used in seismic applications. The strength limits for high-strength concrete

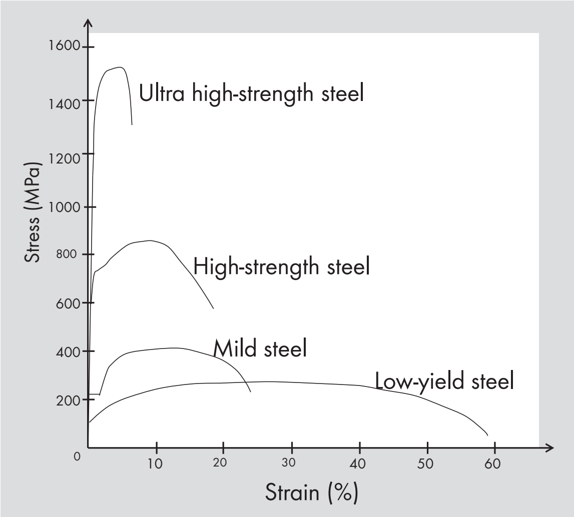


Fig. 2.27: High-strength and low-yield steel material curves

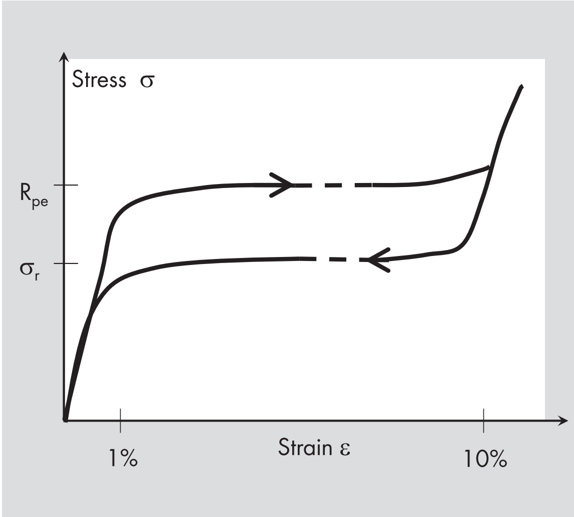


Fig. 2.28: Shape memory alloys constitutive curve

have changed during the years with the development of the technology (Fig. 2.26). A high-strength concrete has a compression strength significantly greater than those used in normal practice, with the minimum values actually equal to about 70 MPa /2.33/. The typical use of high-strength concrete in seismic applications is in the columns of high-rise buildings, which allow a reduction of structural weight. The weak point of the high-strength concrete is the brittleness, but several research projects showed that well confined columns, with adequate details of reinforcements, can provide a satisfactory ductile behaviour /2.34, 2.35/. Increasing tensile strength can be obtained through the high-performance fibre-reinforced concrete. It allows important increases of shear strength, ductility and dissipation in members subjected to reversal cyclic loading cycles /2.36/.

Steel is generally considered as an excellent material for seismic structures, due to its ductility and strength. The research development involving this material over the last decades tends toward the use of high-strength and low-yield steels (Fig. 2.27). The high-strength steel has a yield strength beyond the limit of 350 MPa of common mild steel and is generally used for the structural members that must remain in the elastic range during an earthquake (non-dissipative members). In recent years, the development of steel manufacturing made it possible for the production of steel to have a tensile strength up to 1500 MPa, called ultra high-strength steel, which can also be used for over-resistant elements. On the other hand, the low-yield steel is used for dissipative elements as shear wall panels. Because of the low-yield stress (about 120 MPa), the panels made with this steel can undergo large plastic deformations at the first stage of loading cycles with a consequent increase of dissipated energy /2.37/.

The shape memory alloys are innovative materials that have the ability to remember their initial shape when subjected to a given thermo-mechanical stimulus (Fig. 2.28). Their capacity to recover from a large deformation dissipating a great amount of energy, together with the high ultimate strength, allow the use of shape memory alloys for the production of special devices for seismic resistant structures. An example of seismic application is the use of a Nickel-Titanium shape memory alloy for braces in the retrofit of an existing reinforced concrete building /2.38/.

In recent years, the research put important efforts into the study of innovative solutions, aimed at improving the seismic safety for both existing and new buildings. These new solutions are generally based on different design strategies, such as the weakening of some specific elements to promote the plasticization of specific parts of the structure, the use of suitable special devices or the adoption of alternative structural typologies.

As an example, the need to improve the performance of beam-to-column joints in the torsion resisting frame of steel buildings was a clear result of the catastrophic

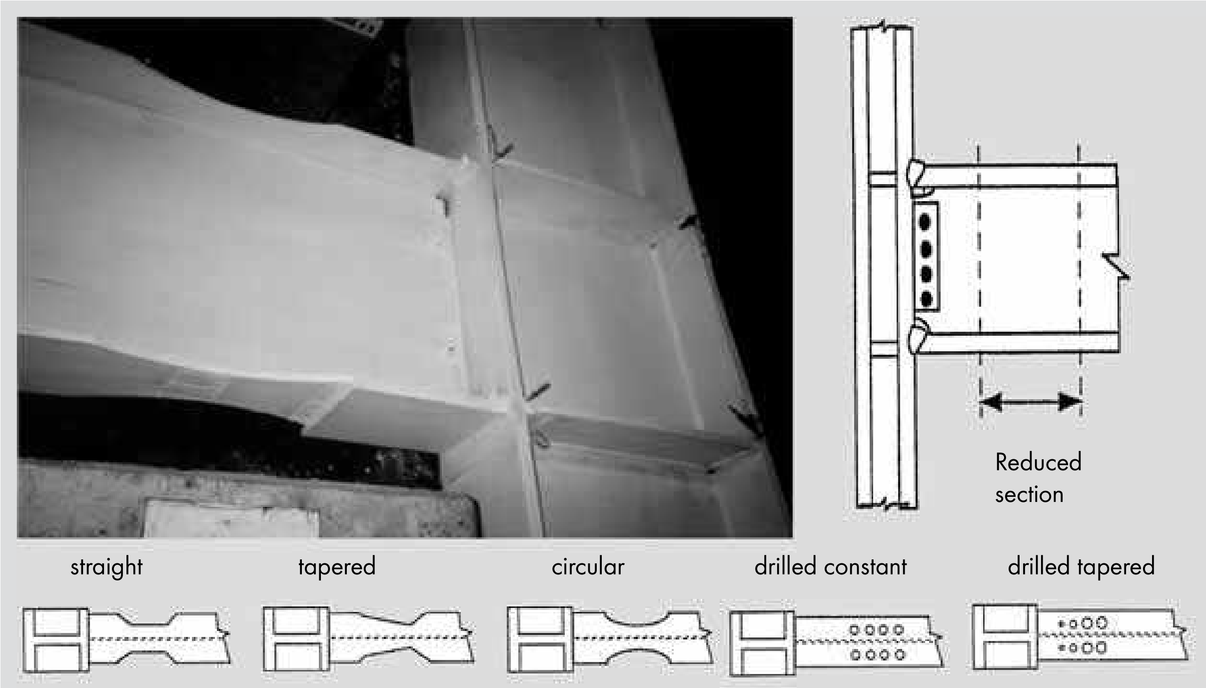


Fig. 2.29: Typical shapes for dog bones /Landolfo, R. (2009)/

earthquakes in Northridge (1994) and Kobe (1995), in which these type of joints exhibited considerable damage and brittle failures. For this purpose, an alternative solution for steel torsion resisting frames, consisting of a weakened beam section at a given distance from the column flange was patented for the first time in 1998 /2.31, 2.39/. This system, named "dog-bone" due to the characteristic shape (Fig. 2.29), allows the definition of the position of plastic hinges in the ductile fuse by the reduction of the beam cross-section. The cross-section reduction protects the integrity of the beam-to-column joint and the column itself, without significant reduction of the overall lateral stiffness of the seismic resistant system. In addition, the "dog-bone" solution facilitates the attainment of a more dissipative collapse mechanism, because of the localization of the plastic hinges. A disadvantage of this solution is represented by the large inelastic deformations affecting the reduced sections after the earthquake, which may be very difficult to repair. Therefore, a possibility to overcome this disadvantage is the use of a dismountable element for the weakened part of the beam /2.40/.

Another possible optimization strategy for torsion resisting frames is the use of "special" joints able to exhibit the function of energy dissipation (dissipative connections). Modern structural codes allow design of dissipative partial strength and/or semi-rigid joints, provided that their rotational capacity is properly assessed and compatible with the deformation demands at a global level. This approach presents several computational difficulties, which complicate the practical application. This problem could be overcome by proposing standard pre-qualified joint solutions /2.41/, from which the designer can choose the most suitable joint for the specific case (Fig. 2.30).

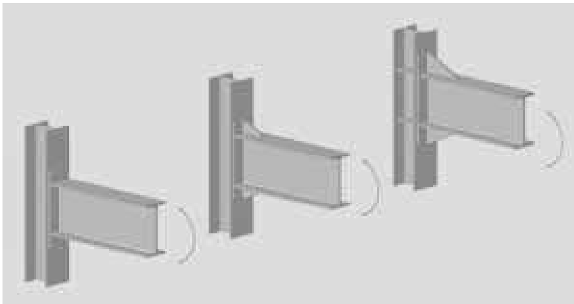


Fig. 2.30: Pre-qualified joint solutions, /AISC (2010), ANSI/AISC 358-10/

As far as innovative solutions for braced structures are concerned, one of the most important systems developed in the recent years is represented by buckling restrained braces /2.35/. Unlike the common steel members, these seismic dissipation devices do not show any type of degradation of strength and stiffness under reversed cyclic

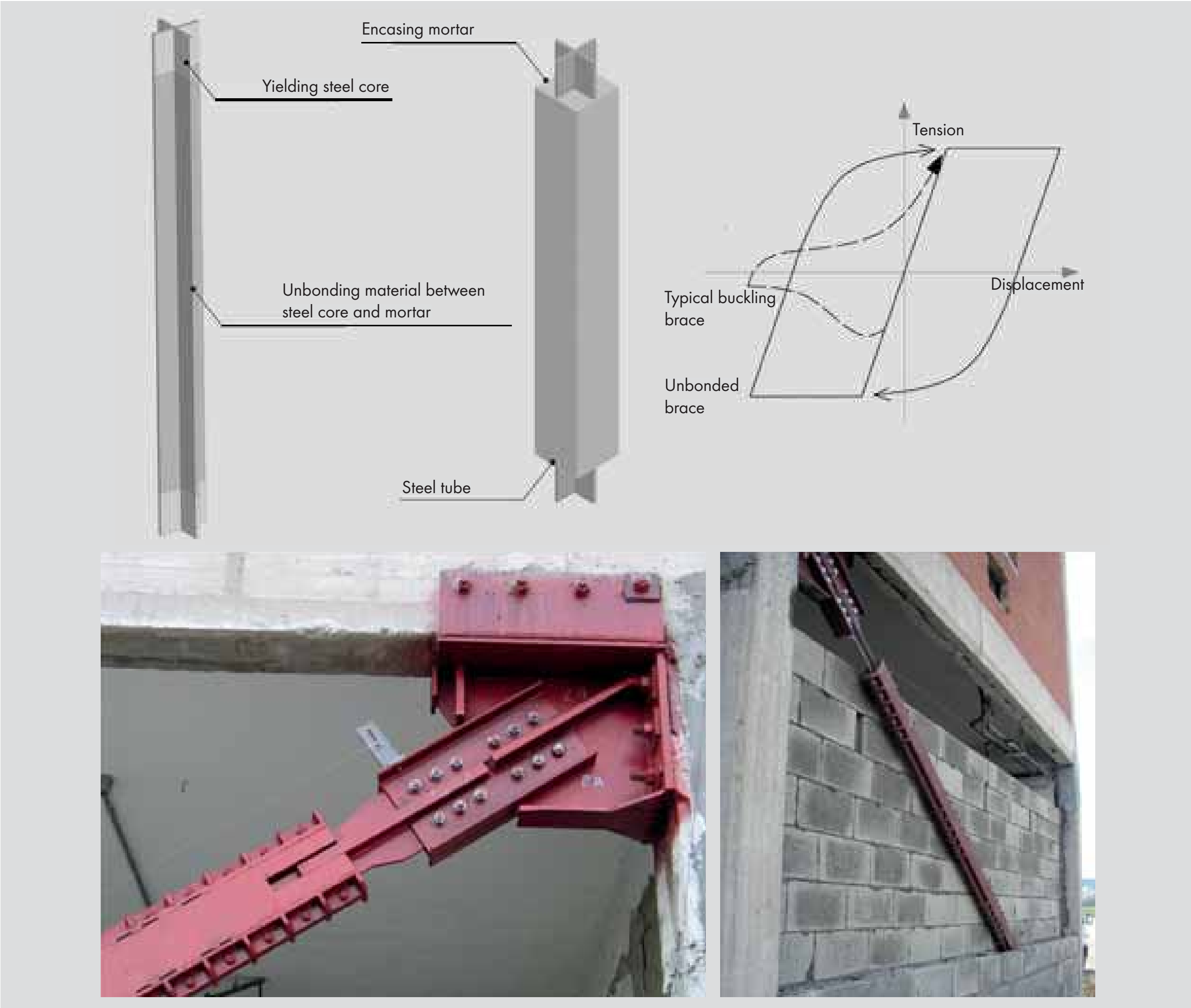


Fig. 2.31: Buckling restrained braces

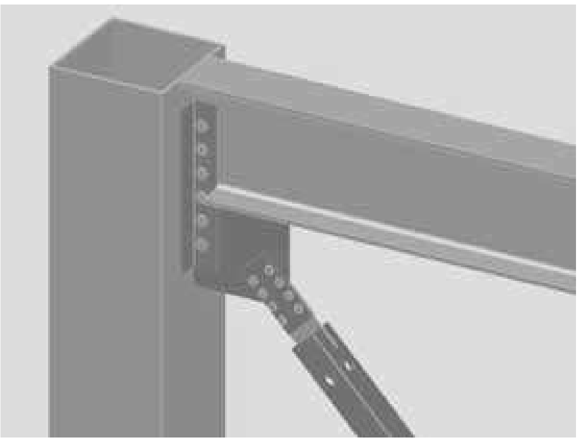


Fig. 2.32: Reduced brace section

loadings. These braces are composed of two distinct and separate parts: a central core made of ordinary steel, devoted to energy dissipation, and an external tube designed to restrain the lateral displacements in order to allow only the axial deformations of the core. In this way, it is possible to decouple the axial resistance, provided by the core, from the flexural buckling resistance provided by the external tube (Fig. 2.31). This system presents a stable hysteretic behaviour, and it allows an independent design of strength, stiffness and ductility. There are several techniques to fabricate a buckling restrained brace. The most common technique consists of an inner core inside a steel tube filled with concrete. In order to reduce the friction between the two materials, a layer of elastomeric material is interposed between steel core and concrete. Alternatively, there are also all steel solutions, obtained simply by inserting a gap between the inner core and the external restraint composed of two or more steel tubes /2.34/. This system can be used for both new constructions and retrofitting of existing buildings.

Also in the case of braced steel structures, the concept of introducing a ductile fuse by means of the weakening of

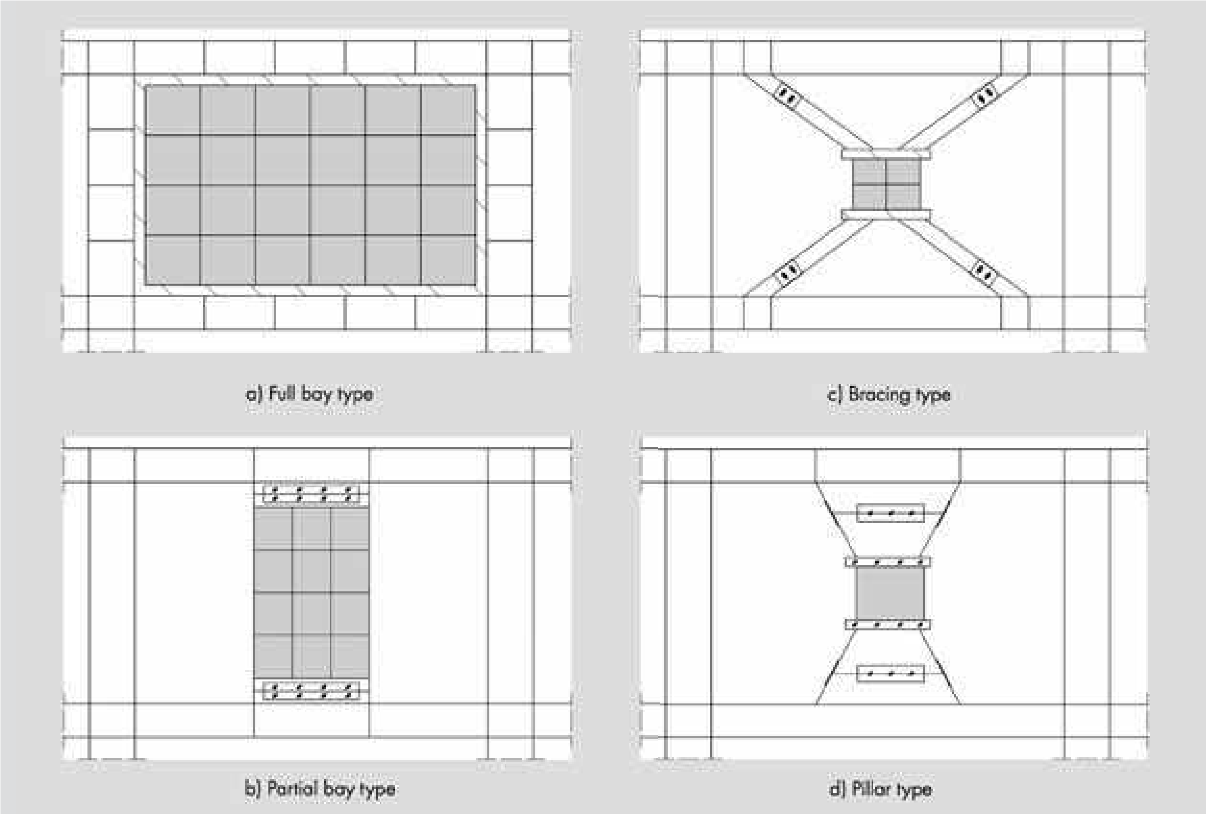


Fig. 2.33: Type of arrangements for metal infilled panels

some sections can be used. In this case, the end sections of the diagonals are weakened through a reduction of the cross-section. This strategy starts from the need to release the code limitations on the slenderness of the diagonals, which often involve oversized diagonal members, especially at the upper storeys. This design solution is called "Reduced Brace Section" (Fig. 2.32), and facilitates preservation of the brace-to-column connections and promotes a global behaviour by avoiding the development of a soft-storey mechanism.

In addition to the framing systems, a modern and innovative system able to resist horizontal actions is represented by the use of metal infilled panels, made of steel or aluminium alloys. These panels are arranged within the structural mesh with different possible arrangements (Fig. 2.33) and are able to absorb a large amount of the seismic energy input /2.42/. In this system, the hysteretic dissipation is essentially based on the principle of shear yielding, activated by the relative displacements of the floors diaphragm. These devices are characterized by lower construction costs and rapid execution and present several mechanical advantages. The infilled panels are able to provide a high ductility and energy dissipation capacity, limiting, at the same time, the rate of the inter-storey drift. In addition, compared to reinforced concrete walls, the metallic panels are lighter and less bulky with a higher structural performance and obvious benefits for the structural members and foundations. This system can be also used for retrofitting existing reinforced concrete buildings /2.43/.

The most recent studies are oriented towards the definition of innovative systems, whose design objective is to ensure limited damage even under strong earthquakes. The main purpose is to conceive structural systems that are able to fulfil the damageability or even the operational limit state under horizontal forces close to those corresponding to the ultimate limit state, without entailing excessive costs /2.12/. In this context, particular dissipative systems, called "self-centring systems", are in the testing phase. These systems offer, together with a good dissipative capacity due to the use of elastic-plastic behaviour or friction systems, the possibility of "re-centring" the structure. These systems restore the structure, instant by instant, to the initial configuration by means of post-tensioned

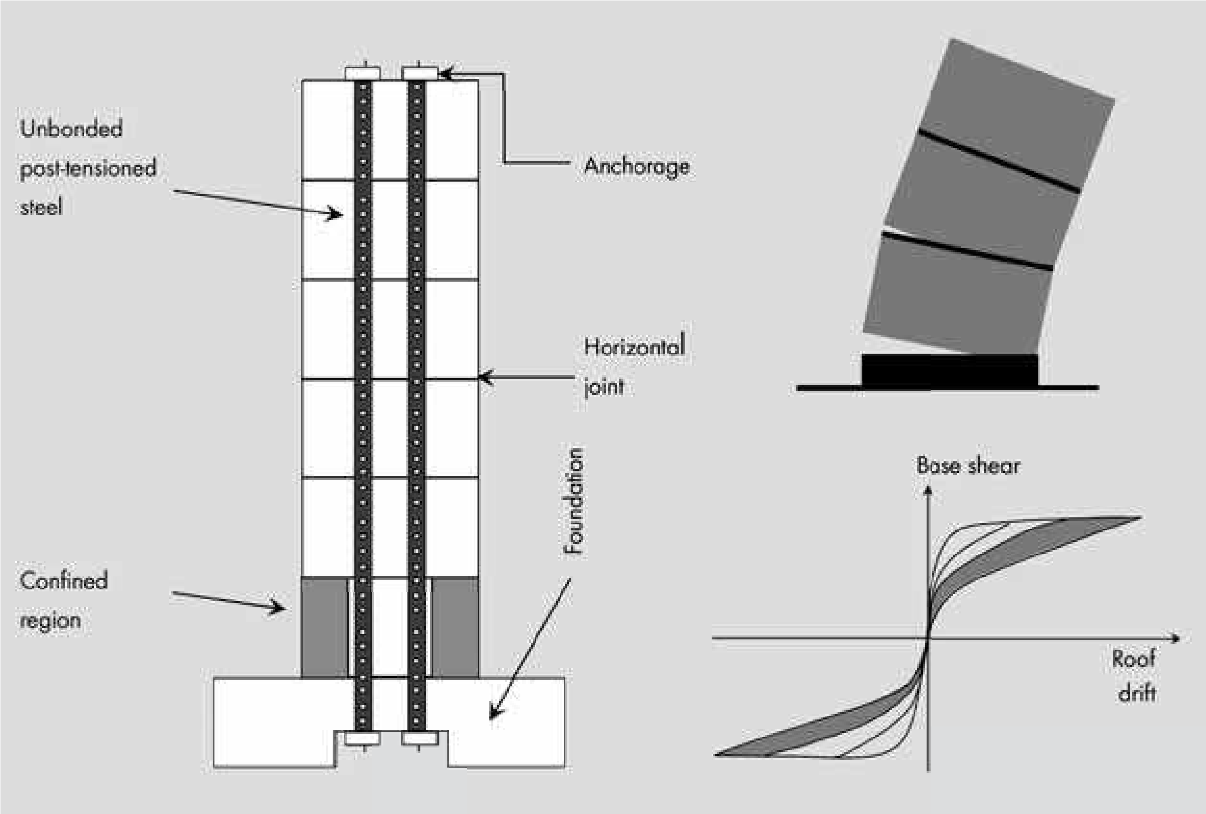


Fig. 2.34: Self-centring systems

steel cables /2.44/. Currently, the main systems under investigation are those involving the use of connections for moment-resisting frames with post-tensioned steel bars /2.45/ and those using centring systems for the whole structure. In the latter case, the advantage is the possibility of eliminating any structural and non-structural damage, because the activated mechanism consists of a rigid rotation of the structure, which is re-centred by the systems of post-tensioned cables remaining in the elastic range. This is the case for unbonded post-tensioned precast concrete walls (Fig. 2.34), which consist of post-tensioning precast wall panels across horizontal joints with unbonded cables anchored at the foundation and the top of the structure. Under earthquake loading, the horizontal joint at the foundation level decompresses due to the toppling torsion with a constant gap opening in horizontal joints, which are closed for effect of post-tensioning and gravity load after the earthquake /2.46/.
Among the various passive control technologies, the base isolation is considered one of the most efficient systems, because of the possibility to significantly reduce the damage, also for potentially devastating seismic events. Contrary to the fixed-base conventional structures being zones specifically devoted to absorb the seismic input (dissipative zones) and subjected to inelastic deformation and damage, in base-isolated systems, the superstructure is isolated from the foundation through specific devices that reduce the ground motion transmitted to the structure by adding significant damping /2.39/.
Different types of base-isolation devices using rubber, neoprene or other materials that have been developed. The most common is the lead-rubber bearing, which consists of layers of rubber attached together by steel layers with a solid lead plug in the middle. The device is connected with the superstructure and the foundation by means of two steel plates (Fig. 2.35 /2.47/). The bearing is very stiff and strong in the vertical direction, but deformable in the horizontal direction allowing displacements and, thus, dissipation during the earthquake.
Another modern strategy used to improve the seismic performance of buildings concentrates on the introduction of special energy dissipating devices in conventional structures. These devices absorb the seismic input energy reducing the demand on the other members, with a

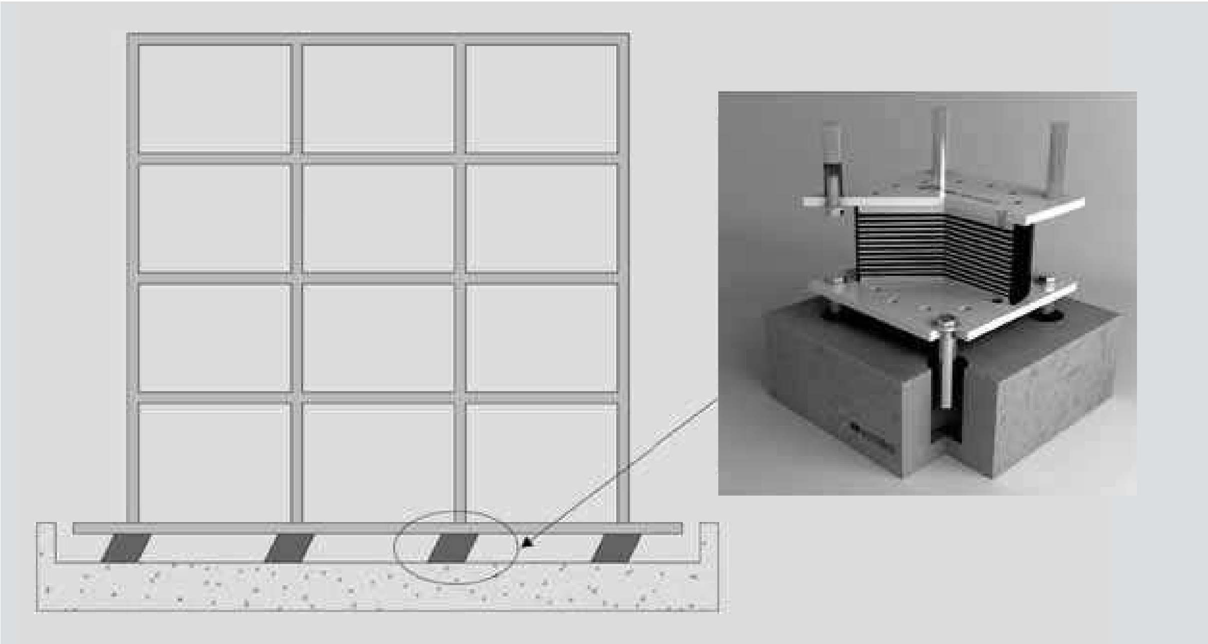


Fig. 2.35: Base-isolated systems /FIP Industriale SpA/

consequent reduction of the damage both to structural and non-structural parts. These devices, known as ADAS (Added Damping And Stiffness elements), have different features (Fig. 2.36) and can be located in the intersection of X-braced systems or along the brace of V-braced systems, or in place of the link of eccentrically Y-braced systems, in order to enhance the dissipation capacity through the shear or flexural yielding of the device /2.42/.
Another philosophy to combat earthquakes concentrates on controlling the seismic response through appropriate adjustment of dynamic properties of the structures. Among these structural control strategies, there are the tuner mass damper system and the active mass damper system. The first is a passive system that absorbs the vibrations due to high winds and moderate earthquakes through a pendulum or a mass at the top of the building that moves opposite to the resonance frequency oscillation of the structure. On the other hand, the second system reduces the effect of the earthquake on the building by means of a computer-controlled actuator that tries to suppress the oscillation of the building (Fig. 2.37). An interesting alternative solution, which proved to be very effective, is a hybrid solution that combines the passive base-isolation and the active control system.
Among the various modern strategies for earthquake resistant buildings presented in this overview, the lightweight drywall constructions, which use cold-formed steel members, represent an efficient and reliable solution for withstanding seismic actions.
Lightweight steel building systems are very competitive thanks to the advantages related to their use as lightness, system versatility, low environmental impact, short time for assembly and simplicity of execution. In particular, from a structural point of view, the lightness of these constructive systems allow achievement of high structural performance in the high seismicity zone also. In fact, the lightness entails reduced structural weight with consequent reduced seismic forces, that allow consideration of the structure as an iper-resistant system. Therefore, it is possible to design the structure in such a way so that it remains in the elastic range under seismic actions at the ultimate limit state, considering a behaviour factor (q) equal to 1.
Lightweight drywall systems are widely used in the seismic area too for non-structural elements, such as partitions and ceilings. Drywall non-structural elements are characterized by an elevated flexibility that allows achievement of low levels of damage for high values of inter-storey drift. In fact, modern seismic codes provide less stringent limits for inter-storey drift with respect to traditional non-structural elements. Therefore, the use of these systems guarantee a good seismic behaviour both for ultimate and damage limit states.

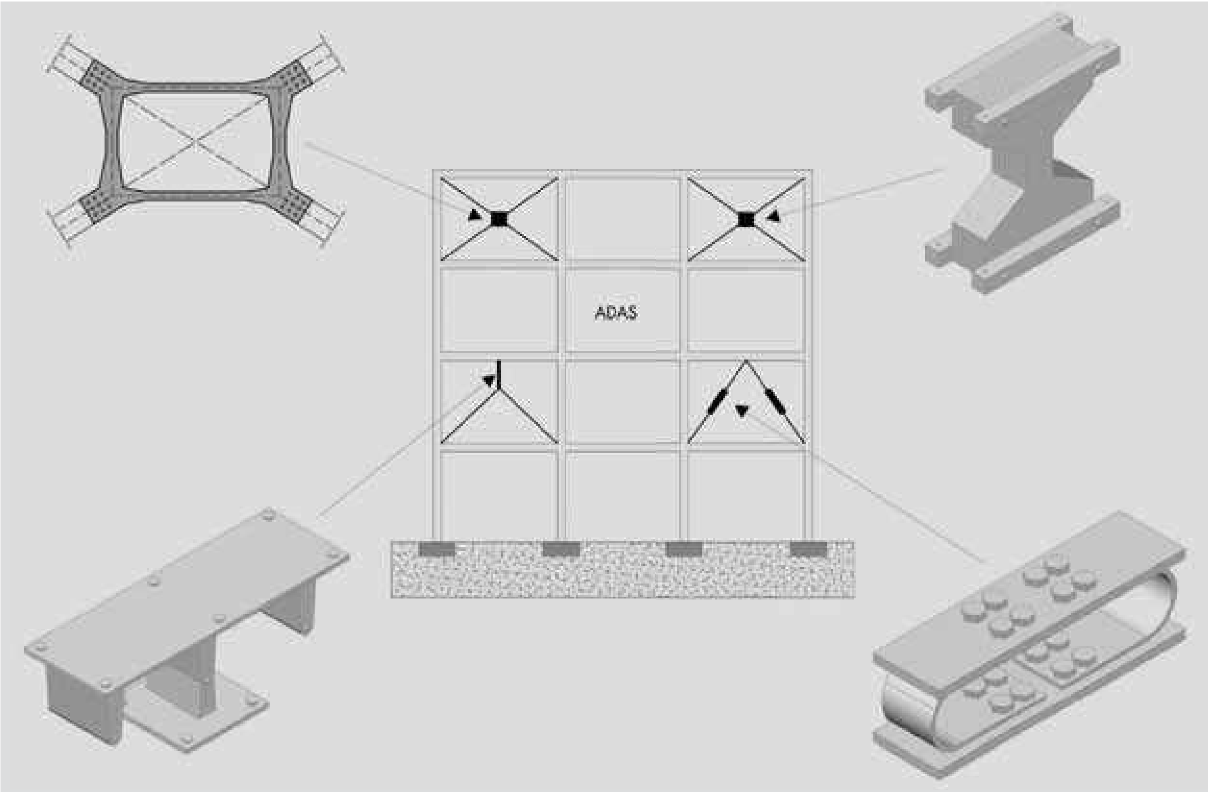


Fig. 2.36: Energy dissipation systems

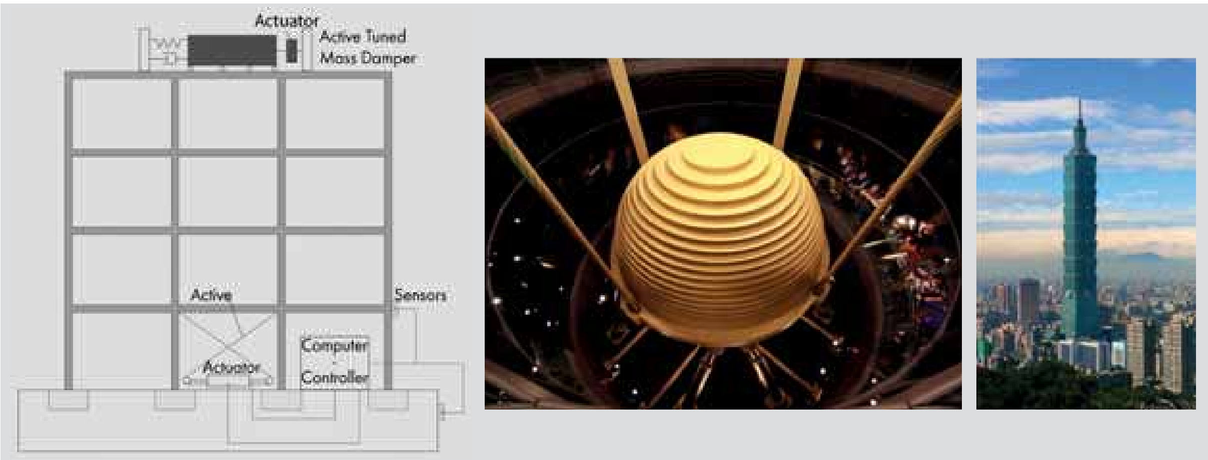


Fig. 2.37: Active control systems /Taipei Financial Center Corporation/

As a conclusion, lightweight drywall steel constructions represent a valid alternative to traditional constructive systems in seismic areas. The peculiarities and potentials of these constructive typologies are the key topic of this book. In particular, in the following chapters, the seismic design of lightweight drywall steel constructions is discussed in-depth both for structural and non-structural systems.

3 Drywalling: Fundamentals and overview

Georg Krämer

Structural and non-structural lightweight steel constructions, also known as drywalling, represent a non-traditional but modern construction method providing substantial advantages in terms of seismic safety, particularly when applying an integral approach to safe and sustainable buildings. The highest demands in terms of fire, sound and thermal / moisture protection as well as sustainable life cycle properties can be implemented while providing highly favourable behaviour and characteristics in terms of seismic design. Before analysing this construction method specifically and detailed from the earthquake engineering point of view, this chapter provides basics and an overview over the wide range of applications.

Over the last decades, drywalling has developed in addition to solid construction as an indispensable, efficient building method for interior construction. Characteristic for drywalling constructions (lightweight constructions) is the dry application of industrial semi-finished products (frames, cladding, insulation materials, etc.), while largely avoiding wet processing.

3.1 Drywalling

Typical drywall constructions used in interior fittings are:

- Wall linings and furring made of composite boards
- Non-load bearing partitions and furring with grid
- Installation shaft walls
- Ceiling linings and suspended ceilings
- Pre-fab floor screeds (dry floor units) and hollow floors
- Room-in-room systems
- Encasement for columns and beams
- Ventilation and cable ducts
- Façade constructions
- Grids and frames for built-ins

Complete buildings are increasingly being constructed with a limited number of storeys using a lightweight steel construction method combined with structural wood frame panel construction.

Drywall constructions facilitate cost-effective solutions with very effective and flexible technical properties, which also enhance the earthquake-resistance of the buildings. The highest demands in terms of fire, sound and thermal / moisture protection can be implemented. Furthermore, drywalling systems are a significant component of decorative interior design (e.g. ceiling design, frieze, mitering, integration of illumination). A large range of design and configuration options can be implemented cost-effectively thanks to the host of combination options of the industrial semi-finished products used in drywalling.

These benefits have led to the extensive spread of drywalling in many different building types and utilizations. Drywall systems can be found, for example, in office and administration buildings, hotels, hospitals, cultural centre buildings and in the construction of new residential buildings and the renovation of older buildings.

Light construction design in drywalling excels when compared to solid constructions due to its lower weight and mainly dry construction method. The main advantages compared to other common solid constructions are as outlined in the following points:

Controllable properties
Innovative lightweight construction enables precise control of the desired properties, in particular with respect to fire protection and sound insulation of the constructions, by optimum utilization of modular material components systems. Combined building physics requirements can be fulfilled by appropriate selection of the frame, the insulation and the cladding in a construction and implemented very cost-effectively.
Sound insulation
By exploiting the "spring-mass-principle", a very high level of sound insulation is possible with lightweight constructions made of flexurally ductile boards (e.g. gypsum boards) and resilient connection elements (e.g. metal studs) with low component weights. Thus, for example, a common metal stud partition with a mass of approx. 50 kg/m^2 achieves an airborne sound insulation performance of approx. 52 dB, usually achieved only with a 240 mm thick solid wall with a mass of approx. 300 kg/m^2.
The optimally sound insulation designed lightweight steel special constructions can even fulfil the enhanced requirements for cinema halls, and this with just a fraction of the weight of a comparative solid construction.
Fire resistance
By using single to multi-layer cladding for structural systems made of steel, wood etc., fire resistance requirements for constructions extending up to 180 minutes can be implemented cost-effectively. Gypsum boards are particularly suitable for application, as the gypsum contains approx. 20 % crystallized water (e.g. approx. 3 l per m^2 for a 15 mm thick gypsum board), which vaporises when exposed to fire and thus dissipates some of the energy and also forms a veil of steam between the fire and the gypsum materials effectively delaying the spread of the fire.
Cracking and structural stability
Lightweight constructions can more easily move with the motions of the flanking bracing construction due to their construction principle without cracks forming or loss of the structural stability.
Integration of HVAC equipment
Easy integration of HVAC equipment and installations in the cavities of the extension systems is possible. An opening (shaft) analogue to those on solid constructions with the corresponding noise and soiling related problems as well as the associated weakening of the structure can be avoided.
There are solutions available for the simple electrical installation right up to complete sanitary installations with the associated plumbing and shafts for pipes as well as anchoring and attachment possibilities for the sanitary fittings.
Optimization of floor space
Lightweight constructions in comparison to solid constructions facilitate the use of thinner / narrower components requiring less space while offering comparable performance (e.g. lightweight wall 150 mm thick compared to a solid wall 240 mm).
Sustainability
Installed lightweight constructions are very variable during building conversion and modernisations and when remodelling. The constructions are, depending on the system, easier to assemble, disassemble, reconfigure and dispose of. The associated noise levels and debris are low.
When required, fire protection and acoustic performance upgrading by the arrangement of additional panel cladding or by the improvement or modification of cavity insulation is very cost-effective to implement by simple disassembly of individual component layers.
Even complete disassembly of existing constructions, e.g. with modifications of the floor plan is possible with little effort. Non-load bearing walls can be generally applied at any new position independently of the floor plan.
Dry construction work
No moisture apart from the filling / jointing work is introduced into the building during drywalling work. Drying times are not a factor, and the building progress is accelerated. During remodelling, damage to existing components (e.g. wooden constructions) in particular, can be avoided.
Work time savings
The high level of pre-fabrication of lightweight systems and their components, right up to the complete pre-fabrication of wall and ceiling panelling reduces building times.

Low transport effort

The transport effort and expense involved moving to and from the building site is reduced dramatically in comparison with solid constructions due to the low weight of drywall systems and the individual components.

Savings in the structural systems

The low weight of lightweight constructions leads to a reduction in the loads on the supporting structure and possibly to a saving in materials and more cost-effective dimensioning of the load-bearing supporting components. When remodelling or converting the building, it is possible to omit support reinforcement.

3.2 Drywall systems overview

The most important drywalling basic systems are described in the following with their most important application and performance characteristics. The respective properties are determined primarily by the design of the individual constructions with respect to their cladding, insulation materials, framing and attachment to the flanking components.

3.2.1 Dry lining made of gypsum boards

Dry lining with gypsum boards /3.1/

A "dry" alternative to wet plastering is dry lining with gypsum boards.

The gypsum boards are applied without framework and "glued" directly to the basic walls (masonry, concrete) with an adhesive compound.

This is to compensate for unevenness in the walls, design-relevant straightening of walls and provision of high-quality level component surfaces that can be further processed using standard techniques (painting, wallpapering, plastering etc.).

The benefits of dry lining in comparison to wet plastering are:

- Reduction of the level of moisture entering the building due to building measures (avoidance of secondary damage, domestic hygiene concerns)
- Avoidance of delays due to setting processes of plasters, which negatively impact the building progress
- Compensation for existing wall dimensional tolerances requiring extensive work or that are impossible to level using thick layer plasters
- Very high requirements placed on the surface evenness and condition are fulfilled
- Good sorption capacity of the gypsum boards provides benefits for the room climate
- Possible improvement of the thermal insulation by internal insulation measures with composite board

With dry lining using gypsum boards, the building physics properties of a basic wall with regard to its sound insulation, thermal insulation or fire protection are generally not improved or only to a very minor degree. Resonance effects can even negatively affect the sound insulation performance of the wall.

Gypsum boards are applied and fixed using adhesive gypsum, where the method shown in Figs. 3.1 - 3.3 is dependent on the evenness of the surface /3.1/

Dry lining requires that the substrate is sufficiently stable. Wallpaper, loose plaster, decorative coats or paste, tiles and wet concrete are an unsuitable substrate for dry linings. Furthermore, the substrate of the basic wall must be dry, free of shrinkage and frost as well as protected against rising or penetrating dampness.

Dry lining with composite boards /3.1/

With a special dry lining type, instead of using standard gypsum board, boards with a laminated insulation layer are employed. They are bonded to the basic wall with an adhesive compound using a method similar to that with gypsum boards and enable the same potential for improvement of the basic wall as an additional thermal insulation layer.

The composite boards consist of gypsum boards, e.g., acc. to EN 520 that are complemented by an insulation layer made of elastified EPS hard foam or densely compressed mineral wool boards. Composite boards are mainly used for interior insulation, but are suitable for conditional improvement of the sound insulation. The thickness of the insulation material is usually in the

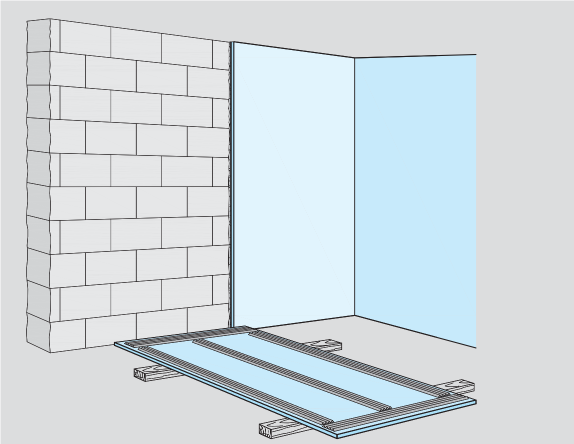


Fig. 3.1: *Thin bed application method (e.g. concrete) /Knauf Gips KG/*

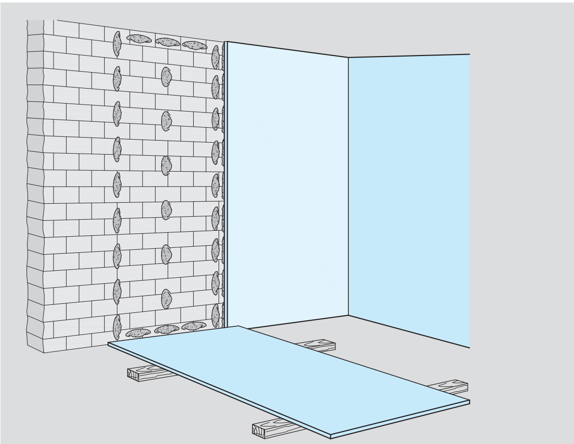
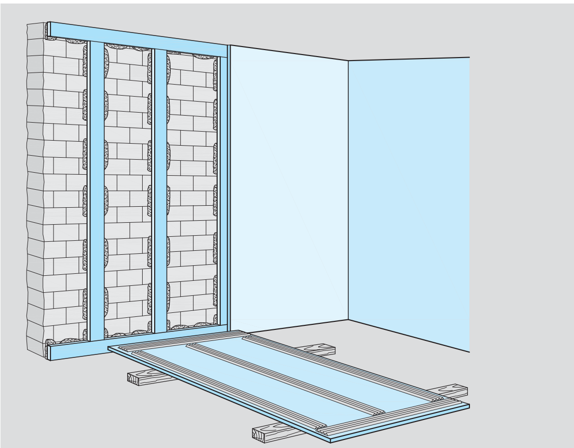


Fig. 3.2: *Application method with lumps of Perlfix (uneven surface up to ≤ 20mm) /Knauf Gips KG/*



Centre board strips, approx. 100 mm wide and bond to the surface with lumps of Perlfix

Fig. 3.3: *Application method with board strips (uneven surface exceeding > 20 mm) /Knauf Gips KG/*

Fig. 3.4: *Application of a composite board /Knauf Gips KG/*

range of 20 to 100 mm, depending on the requirement. Depending on the building physical requirements, a "vapour retarder" may by inserted between the gypsum board and insulation to suit the application.

The anchoring of wall suspended loads on the composite board is implemented using cavity dowels observing the maximum permissible tensile and shear loads of the board type used. At higher loads, the anchor must be anchored in the basic wall that then acts as a cantilever arm in the area of the composite board width.

Glued insulations with composite boards are space-saving systems. However, stable and preferably even surfaces for the basic wall are required. If non-elastified polystyrene is used as an insulation material, there is a danger of impairing the sound insulation of the basic wall. In order to improve the performance ratio of "thermal insulation as a function of thickness", the trend is moving towards insulation materials with a low thermal conductivity (lower space losses).

Composite panels are now concentrating to a greater degree on grey polystyrene. By the addition of graphite, the thermal conductivity (λ = 0.031...0.033 W/(mK)) is reduced by about 20 % in comparison to white polystyrene (λ = 0.035...0.040 W/(mK) /3.2/.

3.2.2 Non-load bearing partitions and furrings with substructure

Non-load bearing internal partitions and furrings consist mainly of grids made of sheet metal profiles or wood, a double-sided (partitions) or single-sided (furrings) cladding of gypsum boards and cavity insulation, if required.

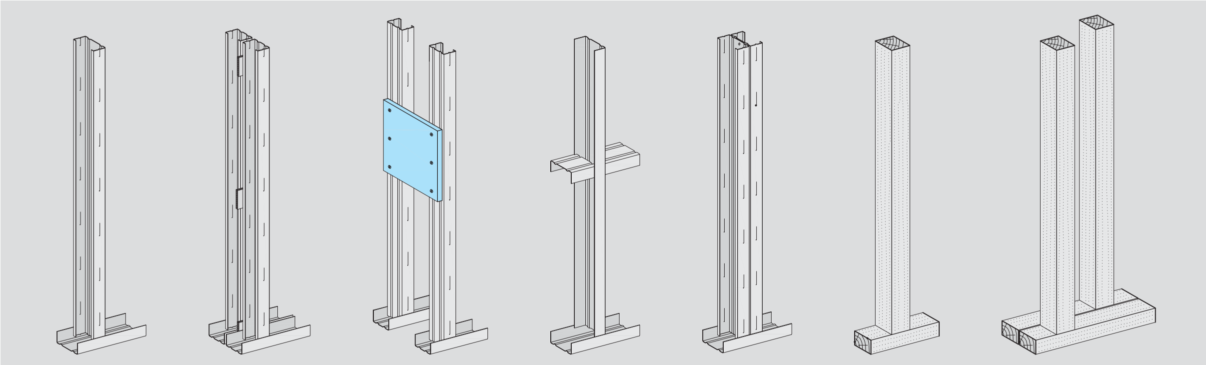


Fig. 3.5: Grid variants of metal stud and wooden frame partitions /Knauf Gips KG/

Fig. 3.6: Installation of an 11 m high stud partition /Knauf Gips KG/

The grid can be implemented as a single metal stud frame, double metal stud frame or crossbar type construction frame. The most common type is the single metal stud frame. The grid of metal stud partitions and furrings consists of thin-walled sheet metal profiles, which are used as connection or stud profiles. The gypsum boards are attached to the grid with drywall screws.

Metal stud partitions

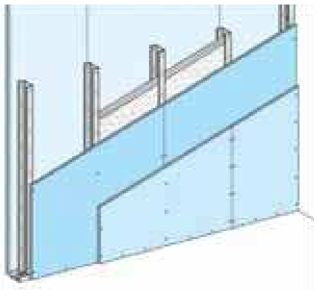
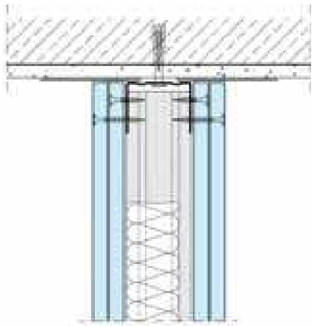
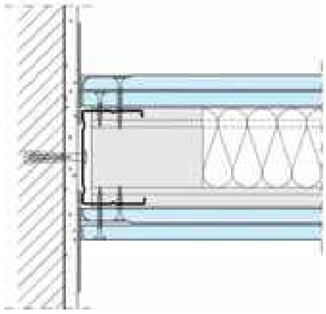
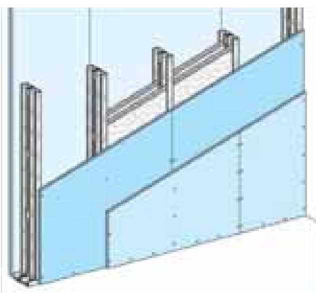
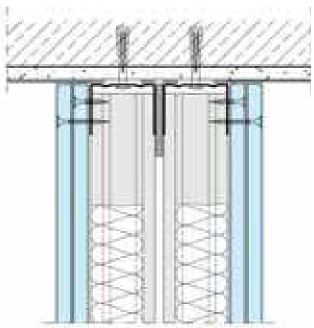
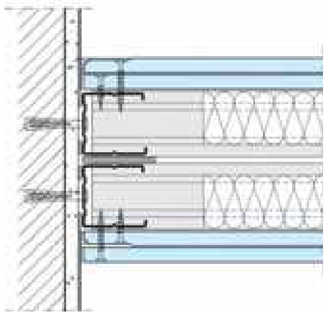
Based on the very good constructional and building physics properties, the non-load bearing partitions are primarily implemented as metal stud partitions with gypsum board cladding /3.3/.

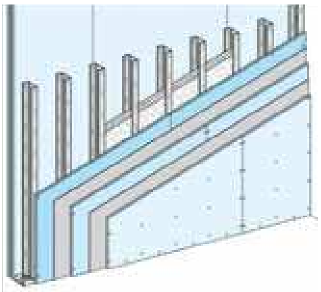
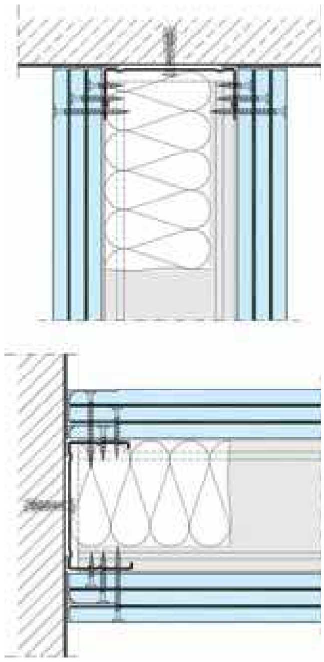
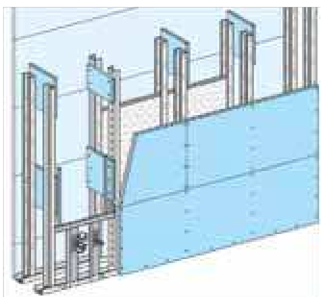
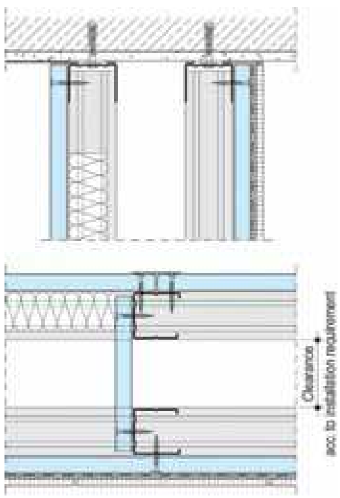
A stud frame is applied using pre-fabricated thin-walled sheet metal profiles (generally 0.6 mm thick) in U and C profile formats. The U profiles are anchored to the floor and ceiling. Between these, C profiles are generally inserted at a grid spacing of 600 or 625 mm (half width of the board cladding). See Fig. 3.5 for different ways of application. The gypsum boards are attached with a frictional bond to the stud frame. Depending on the requirements and construction, particularly those relating to fire protection and sound insulation, insulation material is inserted into the cavities between the cladding.

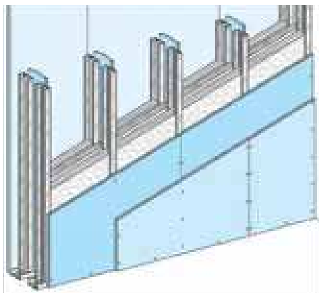
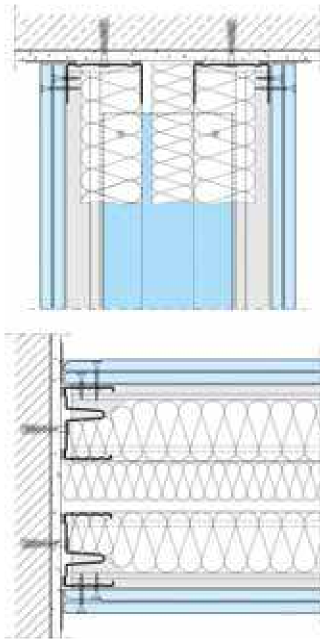
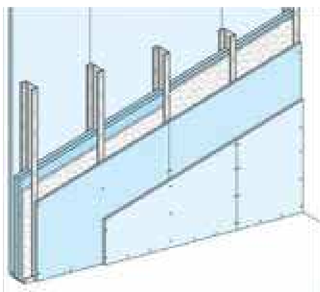
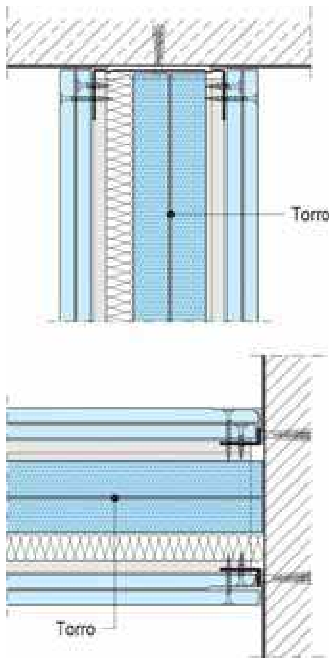
Using metal stud partitions, the constructional variability, even when faced with enhanced technical requirements such as wall height (up to 12 m, see Fig. 3.6), sound insulation (up to 81 dB), fire protection (up to 120 minutes, fire resistance), and burglar and break out security as well as technological requirements such as the installation of technical equipment, e.g. sanitary fittings, can be fulfilled with low area weights. High sound insulation values are possible in particular for the sound insulation area, such as the Knauf system when using special board types Diamant and Silentboard.

Fig. 3.6 and Tab. 3.1 show typical constructions.

Tab.3.1: Partition wall construction variants /Knauf Gips KG/

Construction		**Characteristics**
System sketch	**Sections**	
	 	• Single stud partition • C profile studs • Single layer to three layer gypsum board cladding • With and without insulation material layer • Total thickness 75 – 225 mm • Fire resistance 0 – 90 minutes • Sound reduction index $R_{w,R}$ = 45 – 67 dB • Wall height up to 9 m
	 	• Double stud partitions • C profile studs • Double gypsum board cladding • With and without insulation material layer • Total thickness 155 – 255 mm • Fire resistance 60 – 90 minutes • Sound reduction index $R_{w,R}$ = 63 – 71 dB • Wall height up to 5.50 m

Construction		Characteristics
System sketch	**Sections**	
		• Single stud partition • Security wall / Fire wall • C profile studs • Two or three layer gypsum board cladding • Single or double sheet metal inserts on both sides • With and without insulation material layer • Total thickness 101 – 177 mm • Fire resistance 60 – 120 minutes • Sound reduction index $R_{w,R}$ = 62 – 69 dB • Wall height up to 12 m
		• Double stud partition, braced • Installation wall • C profile studs • Double gypsum board cladding • With and without insulation material layer • Total thickness ≥ 220 mm • Fire resistance 30 and 90 minutes • Wall height up to 6.5 m

Construction		Characteristics
System sketch	**Sections**	
		• Double stud partition, braced • High special sound insulation wall • M profile studs (special section shape for high sound insulation) • Two and three layer gypsum board cladding • With insulation material layer • Total thickness 250 – 400 mm • Fire resistance 90 minutes • Sound reduction index $R_{w,R}$ = 73 – 81 dB • Wall height up to 10 m
		• Single stud partition • Bullet-resistant partition • C profile studs • Double gypsum board cladding • Torro (insertion of highly compact gypsum fibre boards in the partition cavity) • With and without insulation material layer • Total thickness 125 – 150 mm • Bullet resistance class FB 4 • Fire resistance up to 90 minutes • Sound reduction index $R_{w,R}$ = 47 – 53 dB • Partition height up to 5.5 m

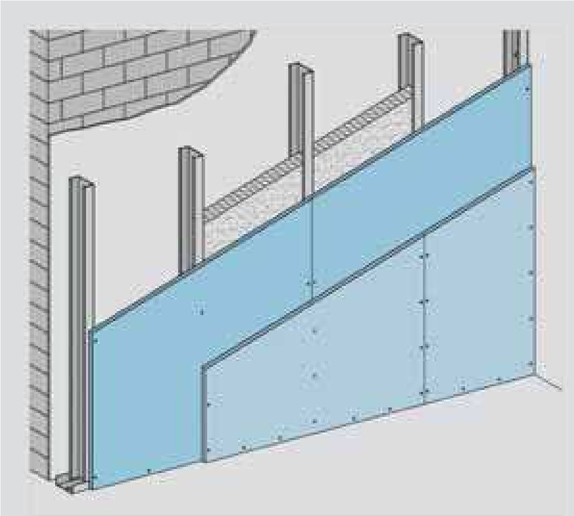


Fig. 3.7: Independent (detached) furring /Knauf Gips KG/

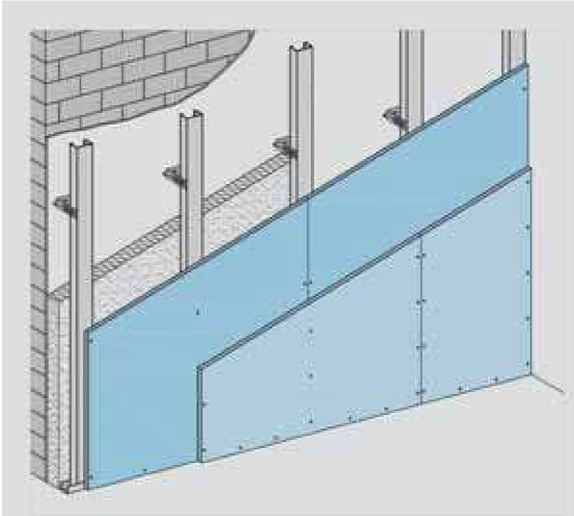


Fig. 3.8: Directly anchored furring /Knauf Gips KG/

All partition walls feature simple assembly by hand with industrially manufactured accessory materials. Geometric adaptation even on buildings with larger dimension tolerances is easily possible.

With the constructional selection and implementation of stud partitions, it must be noted that changes to the material and the construction design details can have a very significant effect on the sound insulation and fire protection properties.

Independent and directly anchored furrings with metal stud framework

Furring with a metal stud framework is a variable assembly system. These systems are not only suitable for the provision of a level surface, but also for improvement of the sound insulation (direct sound insulation, flanking sound insulation) as well as for the fire resistance of the basic wall, and are very well suited for thermal-based renovation. There are no demands made on the surface quality of the basic wall (state of the surface, evenness).

Independent furrings (Fig. 3.7) can be considered as equivalent to stud partitions with single sided gypsum board cladding. They consist of gypsum boards, a frame, insulation and, depending on the building physics requirement when used as an internal insulation system, it also incorporates a vapour retarder between the room side cladding and insulation material. The frame consists primarily of thin metal profiles. They are installed as a free-spanning arrangement in front of the basic wall and must therefore be self-supporting /3.1/.

The disadvantage is that the necessary "self-structural properties" particularly with larger wall heights require larger furring thicknesses.

Directly anchored furring with metal stud framework (Fig. 3.9) complies from a structural point of view with independent furring. In contrast, they are connected to the basic wall at points, so that "weaker" profiles can be used or very high versions can be implemented.

Normally, standard CD 60 ceiling profiles are used, whereby the minimum dimension of the cavity depth from the profile flange height is 27 mm.

The area of application is similar to independent furring. Important is the avoidance of negatively impacting "bridges" between the furring and the basic wall. This is why the bracing of the profiles to the basic wall is generally performed using decoupling spacers (universal brackets) that are arranged at spacings of 100 – 150 cm. Decoupling is undertaken using synthetic materials with elastic properties. Even in sound insulation applications, good results similar to those with independent furring are achieved.

Installation shaft walls

Vertical installations in multi-storey buildings are usually undertaken using installation shafts in most cases for reasons of cost. The supplies to the individual storeys

Fig. 3.9: Directly anchored furring with formation of the coupling point to the basic wall /Knauf Gips KG/

Fig. 3.10: Installation shaft wall design, Knauf system W628, Type A /Knauf Gips KG/

are via these installation shafts. The sealing of the shafts must be undertaken, so that building height and usage-dependent fire protection requirements as well as the sound insulation requirement between the storeys are correctly implemented.

The shafts are enclosed with installation shaft walls. Installation shaft walls are furrings that in addition to fulfilling other constructional and building physics tasks also feature classified fire protection properties (fire resistant, highly fire resistant, fire-proof). The construction must ensure that a spread of fire and smoke from the shaft to the storey is prevented (direction of the classified fire resistance i → o (in-out)).

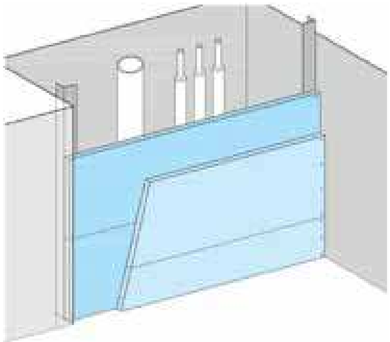
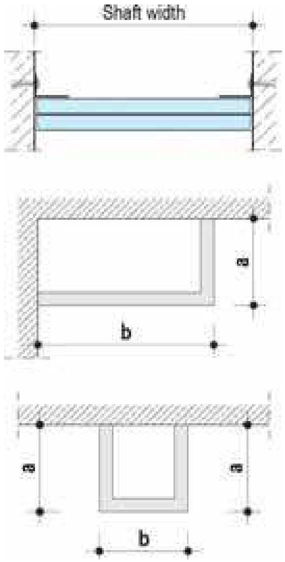
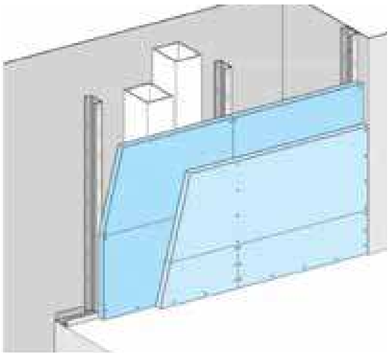
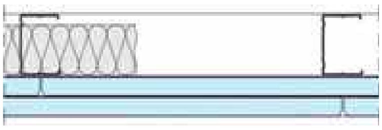
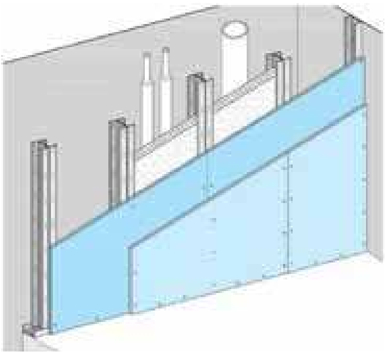
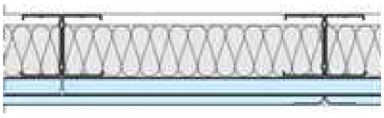
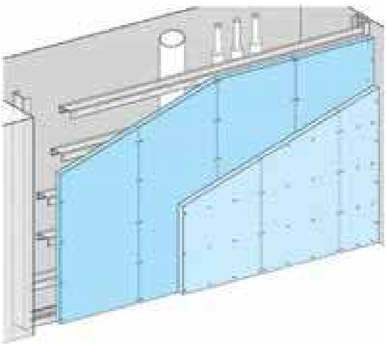
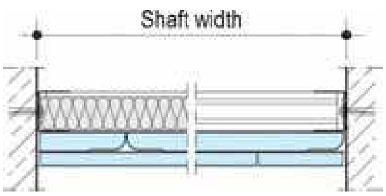
Depending on the geometry and design of the shaft, the construction variants in Tab. 3.2 are possible. Fig. 3.10 shows an installation example. The installation of approved access panels is permissible.

3.2.3 Ceiling linings and suspended ceilings

Drywall and lightweight constructions are excellently suited to complete raw ceilings (supporting structure) as well as for upgrading existing ceilings, as they significantly increase the serviceability of the ceiling with only a minor additional weight (approx. 12 to 30 kg/m^3 for common suspended ceilings).

The design of the lower side of the suspended ceiling, depending on the constructional possibilities as well as the building physics properties relating to fire

Tab. 3.2: Shaft wall constructions, Knauf system /Knauf Gips KG/

Construction		**Characteristics**
System sketch	**Section**	
		• Wall without metal stud frame • Connection profile L, C or U profiles as perimeter support • Double gypsum board cladding (2x25 mm) • Total thickness 100 mm • Fire resistance 90 minutes • Wall height up to 5000 mm • Shaft width ≤ 2000 mm
		• Single stud partition • C profile studs • Double gypsum board cladding • Total thickness 100 – 150 mm • Fire resistance 30 – 90 minutes • Wall height up to 5000 mm
		• Composite stud partition • C profile studs • Double gypsum board cladding • Total thickness 75 – 150 mm • Fire resistance 30 – 90 minutes • Wall height up to 5600 mm
		• Crossbar frame partition • C profile studs • Single or double gypsum board cladding • Total thickness 75 – 150 mm • Fire resistance 30 – 90 minutes • Wall height unlimited • Shaft width ≤ 5000 mm

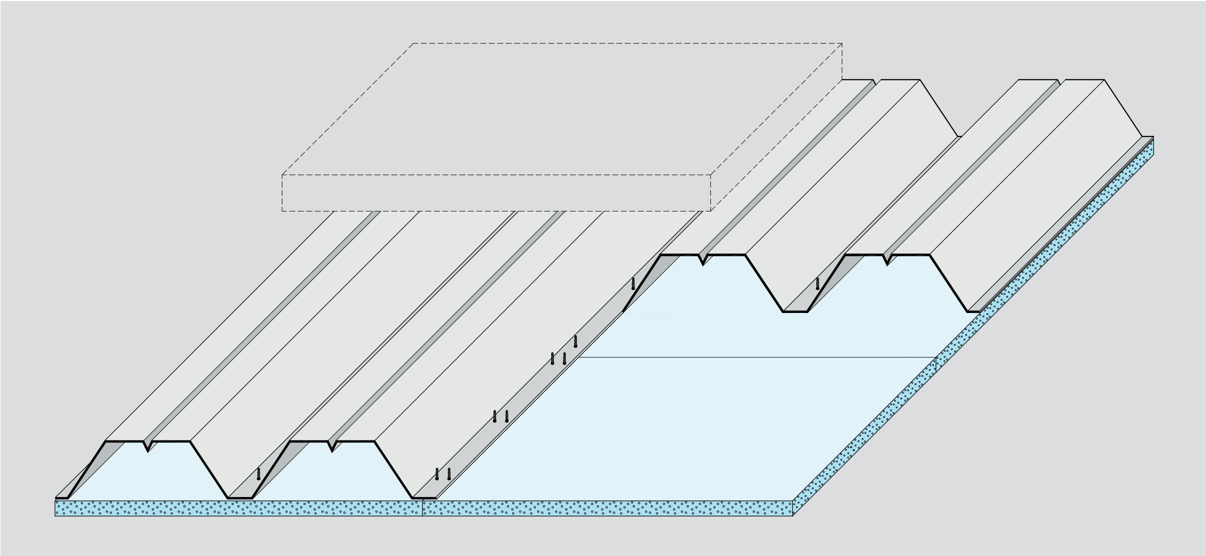


Fig. 3.11: Direct lining of trapezoid sheet metal /Knauf Gips KG/

protection, sound insulation and thermal insulation and the architectural preferences, can be implemented as a ceiling lining, as a suspended ceiling with its own frame construction or as a freely spanning ceiling without anchoring points on the ceiling.

Ceiling linings

Ceiling linings are applied directly under the building element to be encased. Here a direct attachment to the basic ceiling via directly anchored furring channels is possible.
They are preferred where a low height of construction is present and the loss of headroom should be avoided.
Fig. 3.11 is an application example.

Direct cladding of ceiling bottom with furring channels

Direct cladding can also be undertaken with a frame.
On the variants with a frame, it is anchored directly and without spacing on the anchoring substrate, and the cladding is screw fixed to the frame. This has the benefit of levelling any unevenness on the substrate and results in an improved sound decoupling of the cladding from the anchoring substrate.
The furring channels facilitate a more cost-effective installation (less installation time) and result in an impressive increase in the sound insulation particularly with the use of resilient channels /3.9, 3.10/.

Suspended ceilings

With sufficient height availability, the suspended ceiling is applied preferably using a metal profile framework. It is beneficial that the cavity between the insulation material (improvement of sound insulation by cavity dampening, thermal insulation) can also accommodate installations. Furthermore, the room volume to be heated can be reduced in rooms with high ceilings. The design of the suspended ceilings is implemented in Fig 3.12 using the construction principles shown with a simple suspended furring channel, with a double profile frame or flush profile frame.
The selection of the construction method is dependent on the corresponding constructional constraints on site such as the available suspension height and the possibilities for attachment to the basic ceiling. The minimum suspension heights are approx. 40 mm with universal brackets in conjunction with single channel grids or flush profile frame. The use of vernier (Nonius) hangers with the vernier hanger upper section and hanger connector facilitates the combination with a doubled profile frame suspension height that can be in the order of several metres. When 2 mm UA profiles are used as the carrying channel, the latter mentioned version for these profile span widths from 1000 – 2500 mm (spacing of the connection points on the basic ceiling) can be realized in dependence on the cladding thickness and the profile spacing /3.9/.
Fig. 3.13 shows an installation example with integrated dome (cupola).

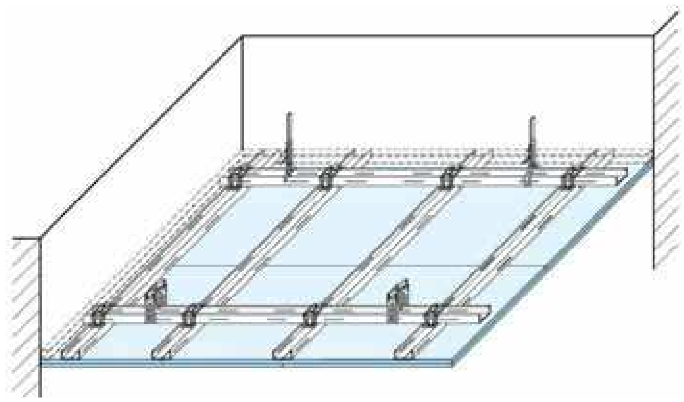
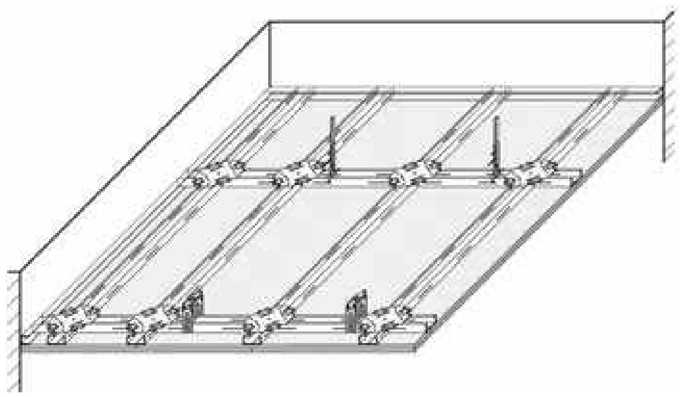
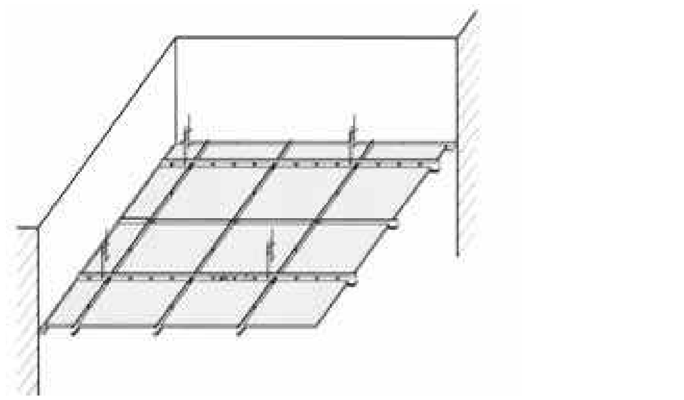
	*Alternative 1:* • *Suspended double level grid with C profile channels*
	*Alternative 2:* • *Suspended flush profile grid with C profile channels*
	*Alternative 3:* • *Dropped ceiling* *Suspended flush profile frame with special profiles* • *Dropped ceiling boards are applied to the stud frame*

Fig. 3.12: Constructive options for suspended ceilings /Knauf Gips KG/

Free spanning ceilings

For these ceilings, there is no suspension from the existing ceilings, but rather the self-supporting ceiling is freely spanned between two room walls and is anchored to them /3.9, 3.10, 3.12/. The benefits of the free spanning ceiling are in particular the following points.

- No additional loading of the existing ceiling (problem solver relating to load capacity of wood joist ceilings) by the subceiling
- Time-consuming search for anchoring points and testing of the anchoring points on the basic ceiling no longer necessary
- Cavity between basic ceiling and subceiling without hindrance from the hangers provides full availability for conduits (wiring, pipes) (installation levels in hall areas)
- Coupling free design to the basic ceiling facilitates maximum sound insulation and improvement of the fire protection
- Simpler, safer installation and more cost-effective installation when compared to other types of suspended ceiling installation

The free spanning ceiling consists of a grid made of UW profiles that are connected to the opposite wall and freely-spanning simple or composite profiles (CW 75, CW 100,

Fig. 3.13: Installation of a suspended ceiling with integrated cupola /Knauf Gips KG/

Fig. 3.14: Installation of a free-spanning suspended ceiling /Knauf Gips KG/

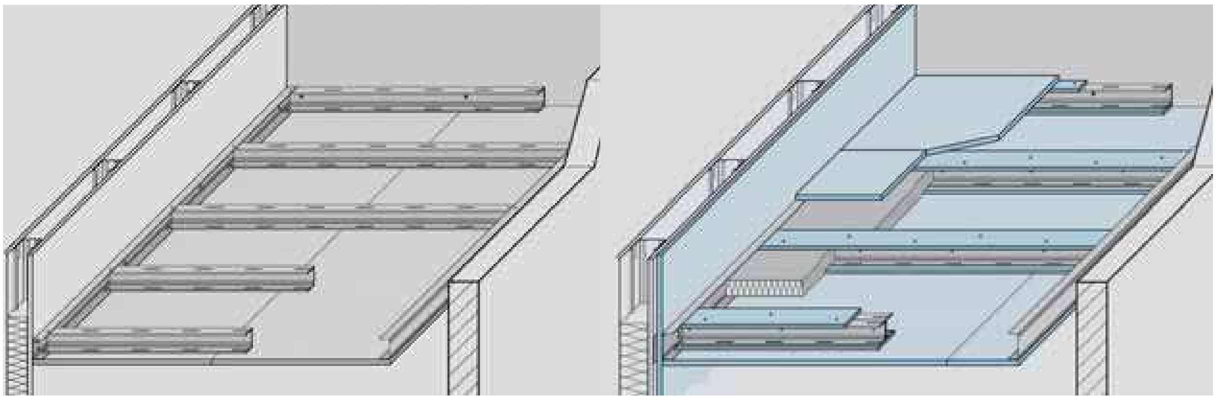


Fig. 3.15: Free-spanning subceiling with and without fire resistance /Knauf Gips KG/

CW 125), which are inserted into the connection profile (axial spacing max 600 or 625 mm) (Fig. 3.14 and 3.15). In combination with mineral wool layers, cooling strips made of gypsum boards on the profiles and a lower side cladding made of gypsum boards or Fireboard will achieve a fire resistance classification not just for a fire from below, but also for a fire from the plenum (installation level) /3.10, 3.13/.

3.2.4 Floor constructions

Floor constructions made of gypsum boards can be applied in 3 different variants: Pre-fab floor screed, hollow floors and raised access floors.
Whereas pre-fab floor screed is applied as dry flooring with or without insulation or as a heating floor screed, hollow and raised access floors provide an additional installation level.

Pre-fab floor screeds (dry floor units)

Pre-fab floor screed is a very interesting alternative to conventional wet screeds and is frequently referred to as dry floor units in practice. The weight of pre-fab floor screed is reduced by as much as 70 % compared to wet screeds with a comparable usage. Process related waiting times for setting and drying are not required. The achievable building and physical relevant characteristic values, related to the preferred application areas (residential buildings, schools, hospitals etc.), are fully compliant.
Pre-fab floor screed is created by the application of building units capable of manual assembly that are simply frictionally bonded to one another on site with adhesive (Fig. 3.16). The screed panels are generally pre-fabricated composite units made of adhesively bonded gypsum boards with staggered shiplap or monolithic gypsum fibre boards with a milled, robust shiplap. The latter are, for example, high-density gypsum fibre, such as Knauf Brio /3.14/. The elements are 18 or 23 mm thick (Tab. 3.3). They are applied at an offset, glue is applied in the notch and screwed or stapled together (Tab. 3.4) /3.15/.
For this system and dry screeds made of gypsum boards and in dependence on the application, particularly for live loads, different layer designs, even with higher insulation layer thickness and even with levelling layers acc. to Tab. 3.5 are possible. For levelling larger unevenness of the basic floor, use of bonded bulk levellers /3.16/ should be considered.

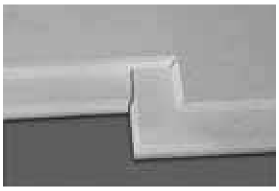
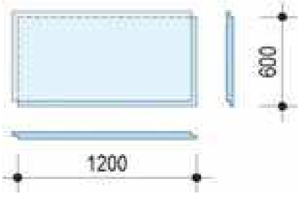
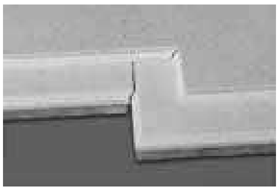
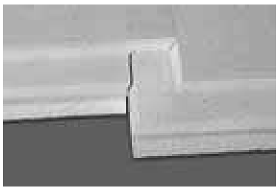
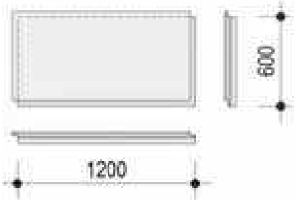
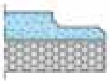
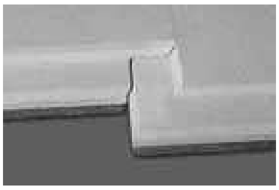
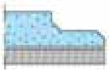
Hollow floor

Hollow floors consist of casings made of gypsum or gypsum fibre boards that are mounted directly on adjustable height supports. The hollow bodies (casings) accept the actual wearing layer, for example a flowing screed. The Knauf system GIFAfloor provides a gypsum fibre material that combines the casing and wearing layer in one, allowing the covering layer to be applied directly on the gypsum fibre boards (Fig. 3.17, 3.18).
Access to the cavity is only possible via access panels or ducts. The load applied to flooring is presented by the imposed load as a changeable load or live load. There are applicable standards to suit the usage of the flooring, e.g. Eurocode 1 (EN 1991) of standards, “Actions on structures” defining the area and point loads, for which the flooring construction is to be rated.

Tab. 3.3: Flooring structures made of dry screed for different live loads /Knauf Gips KG/

Usages and application areas	Imposed loads: Area load	Imposed loads: Single load	Substrate: Thickness in mm	Substrate	Possible design under the substrate / the underfloor heating Thickness in mm: 1 Mineral wool MW	2 Dry bulk leveller PA	3 Dry bulk leveller PA + floor board TUB 12.5	4 Wood fibre WF	5 EPS	6 EPO Light
Without underfloor heating system										
Rooms and corridors in residential buildings, bedrooms in hospitals, hotel rooms incl. the corresponding kitchens and bathrooms	2 kN/m^2	1 kN	18	Brio 18	10 to 20	20 to 100	20 to 100 + TUB	10 to 20	0 to 100	15 to 800
			23	Brio 23						
			25	TUB 2x12.5						
Corridors in office buildings, office areas, doctors practices, waiting rooms, lounges including the corridors, areas in sales rooms up to 50 m^2 in residential, areas in sales rooms up to 50 m^2 in residential, office and comparable buildings	2 kN/m^2	2 kN	18	Brio 18	-	20 to 30	20 to 100 + TUB	10 to 20	0 to 100	15 to 800
			23	Brio 23						
			25	TUB 2x12.5						
Office areas with higher loads	3 kN/m^2	2 kN	18	Brio 18	-	-	20 to 100 + TUB	10 to 20	0 to 100	15 to 800
			23	Brio 23						
Corridors in hospitals, hotels, old peoples homes, boarding schools etc.; kitchens and treatment rooms incl. surgery rooms without heavy equipment.	3 kN/m^2	3 kN	23	Brio 23	-	-	-	10 to 20	0 to 100	15 to 800
			30.5	Brio 18 + TUB12.5						
Corridors in hospitals, areas with tables, e.g. classrooms, cafes, restaurants, canteens, auditoria, reception rooms	4 kN/m^2	3 kN	35.5	Brio 23 + TUB12.5	-	-	-	10 to 20	0 to 100	15 to 800
			36	Brio 18 + Brio 18						
			37.5	TUB 2x12.5 + TUB 12.5						
Areas with fixed seating, e.g. in churches, theatres, cinemas, congress rooms, auditoria, meeting halls, waiting rooms	4 kN/m^2	4 kN	36	Brio 18 + Brio 18	-	-	-	10 to 20	0 to 100	15 to 800
			46	Brio 23 + Brio 23						
Freely walkable areas, e.g. museum and exhibition areas, entrance areas in public buildings and hotels, areas where large groups of people meet, e.g. in buildings such as concert halls, entrance areas; areas in retail stores and department stores, areas in factories and light-duty workshops	5 kN/m^2	4 kN	46	Brio 23 + Brio 23	-	-	-	10 to 20	0 to 100	15 to 800
With underfloor heating system Type B										
Rooms and corridors in residential buildings, bedrooms in hospitals, hotel rooms incl. the corresponding kitchens and bathrooms	2 kN/m^2	1 kN	18	Brio 18	-	-	-	max. 10	0 to 50	15 to 800
			23	Brio 23	-	-	20 to 50			
			25	TUB 2x12.5						
Corridors in office buildings, office areas, doctors practices, waiting rooms, lounges including the corridors, areas in sales rooms up to 50 m^2 in residential, areas in sales rooms up to 50 m^2 in residential, office and comparable buildings	2 kN/m^2	2 kN	23	Brio 23	-	-	20 to 50	max. 10	0 to 50	15 to 800
			25	TUB 2x12.5						
Office areas with higher loads	3 kN/m^2	2 kN	23	Brio 23	-	-	20 to 50	max. 10	0 to 50	15 to 800

Tab. 3.4: *Dry flooring elements, Knauf system Brio /Knauf Gips KG/*

Picture	Format	Edge design	Thickness
		Brio 18 	18 mm
		Brio 23 	23 mm
		Brio 18 WF 	18 mm + 10 mm wood fibre
		Brio 18 EPS 	18 mm + 20 mm EPS
		Brio 23 WF 	23 mm + 10 mm wood fibre

Application of adhesive *Laying* *Stapling*

Fig. 3.16: *Laying of pre-fab floor screed /Knauf Gips KG/*

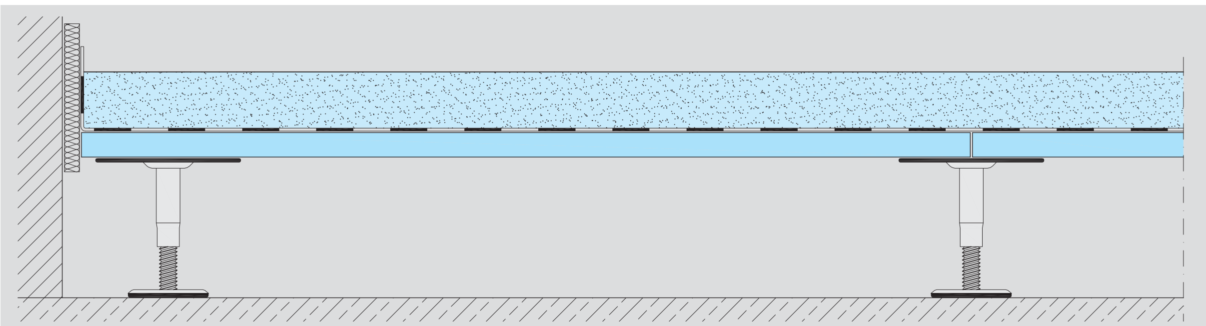


Fig. 3.17: *Hollow floor with hollow body casing made of gypsum boards and wearing layer made of flowing screed /Knauf Gips KG/*

Fig. 3.18: Hollow floor /Knauf Gips KG/

Tab. 3.5: Loads for hollow floor Knauf GIFAfloor /Knauf Integral KG/

Imposed loads		**Substrate (hollow floor element)**		**Support grid in mm**
Area load (kN/m^2)	**Single load (kN)**	**Thickness (mm)**	**Material 1)**	**The selection of suitable hollow floor supports is undertaken taking the height of the hollow floor in conjunction with the load capacity into consideration**
1	1	25		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	GIFAfloor FHB	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		600 x 600
5	6	28 + 13		425 x 425
6	7	32 + 13	GIFAfloor FHB + GIFAfloor LEP	425 x 425
6	7	32 + 18		600 x 600
6	10	32 + 18		425 x 425

For hollow floors, the selection of the support grid spacings, the load capacity of the individual supports as well as the load capacity of the gypsum or gypsum fibre boards in dependence on their thickness is the decisive parameter for the load capacity of the flooring construction. The testing and classification of the load capacity, for example, is acc. to EN 13213 (Hollow floors). Hollow floor, Knauf system GIFAfloor can accept loads in acc. to Tab. 3.4.

3.2.5 Room-in-room systems

In new construction as well as remodelling when converting the usage of large halls – the ceilings are high and the walls are far away – the interesting architectural alternative and oftentimes the very cost-effective solution is the usage of rooms with their own support structures within the building envelope. Structures of this nature are referred to as room-in-room systems. These systems facilitate a large degree of independence from the existing building fabric /3.18, 3.19/.

Typical applications are rooms for separate usage in halls and larger rooms – and it can well be possible that large attic spaces are involved – such as common and meeting rooms, offices, test and monitoring rooms in production plants, sanitary modules, residential areas in loft apartments, diagnostics rooms, teaching rooms and even rehearsal rooms for musicians.

An optimum room-in-room construction that does not just technologically cover the requirements, but also the technical and building physics requirements across a wide range is the Knauf system Cubo.

The construction principle just like the assembly and installation is very easy. The system consists of the usual semi-finished drywalling products. The supporting structure consists of telescopic system supports attached to the floor with dowels at appropriate spacings. To these, surrounding horizontal UA profiles in the support head area are attached to the respective connection elements. In this supporting system, free-spanning ceilings are used for room enclosure in the ceiling area, whereby the furring channels for the ceiling (generally CW or UA double profiles) are inserted into the UA perimeter runners of the support system. The wall levels of the support system are sealed with stud partitions made of CW/MW 100 studs. The partition walls are each integrated into the support system between the telescopic supports. Gypsum boards and/or Fireboard are used for cladding of the ceiling and wall areas. In addition to enclosing the room, the diaphragm action of ceilings and wall surfaces secures the bracing of the resulting room cell. The construction principle in shown in Fig. 3.19. /3.20/

The system features a large range of applications and variations (Tab. 3.6; Figs. 3.21, 3.22).

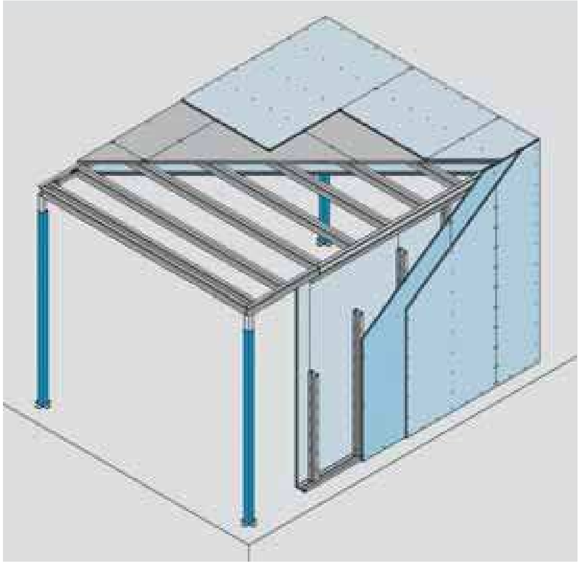


Fig. 3.19: Construction principle, Knauf system Cubo

For the variant Cubo Basis, independent rooms with a width up to max. 7500 mm and unlimited length are possible using this construction principle. The room height can vary as required from 2500 mm to 3700 mm. Even the partial connection of the room cell to existing flanking constructional components is possible. Openings up to a width of 2000 mm are permissible without any additional constructional measures. Depending on the construction design and dimensioning, a fire protection rating up to 90 minutes as well as sound insulation with a standardised level difference D_{nT} up to 55 dB can be achieved.

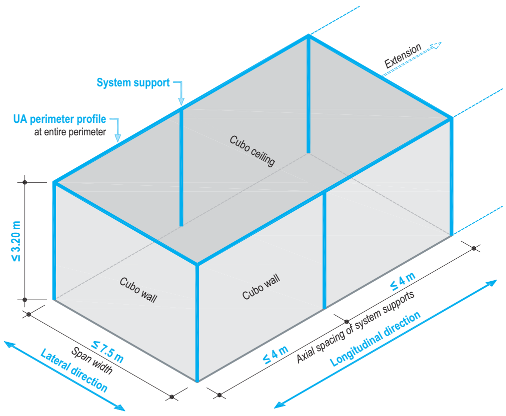
The system variant Cubo Empore is dimensioned so that this construction can bear additional structural and dynamic loads in comparison to the basis construction. The ceiling also features additional wooden composite boards for load distribution and is approved for the load cases "conditionally walkable", "static superimposed loads up to 0.5 kN/m^2" (Fig. 3.20) and "Live loads up to 2.0 kN/m^2". Accordingly, this system variant offers room concepts with usage options in a 2nd level for the installation in high rooms for the latter load case.

The 3rd system variant Cubo Escape Tunnel is conceived for use as a fire protection safety zone or as a zoning measure for escape routes (means of escape). The comprehensive "protective envelope" (wall and ceiling area) consists of two layers of Fireboard reinforced with an additional sheet metal layer. The system not only achieves the fire protection characteristics of a firewall, it

Tab. 3.6: Application variants of the system Cubo /Knauf Gips KG/

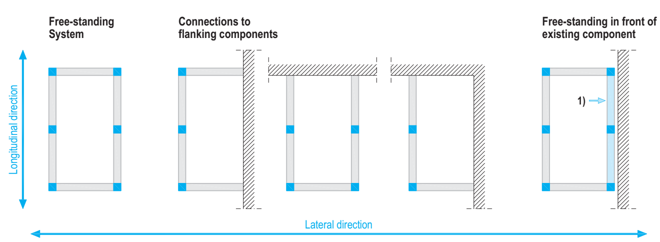
<table>
<tr><th>System / Construction data</th><th>Application variants</th></tr>
<tr><td>

Cubo Basis



Height ≤ 3200 mm
Width ≤ 7500 mm
Length ≤ 4000 mm x n

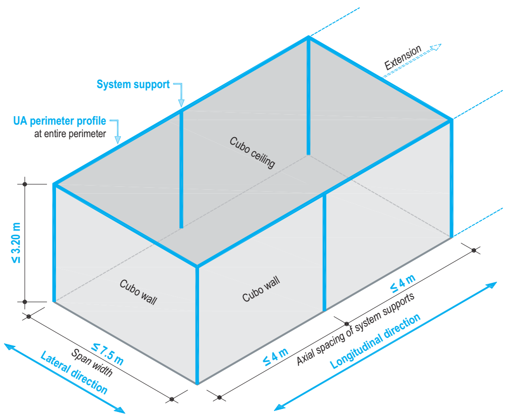
</td><td>



Design of the Cubo on Cubo in lateral direction and in the combination lateral and longitudinal direction is possible from a constructional perspective

</td></tr>
<tr><td>

Cubo Empore



Height ≤ 3200 mm
Width ≤ 5800 mm
Length ≤ 4000 mm x n

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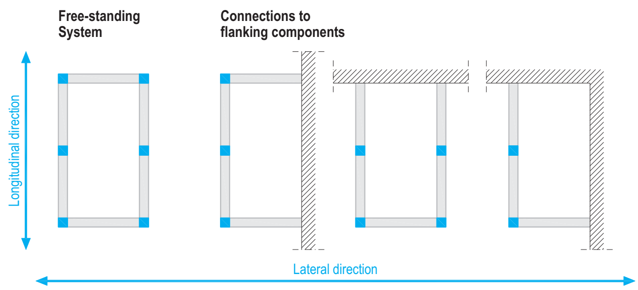

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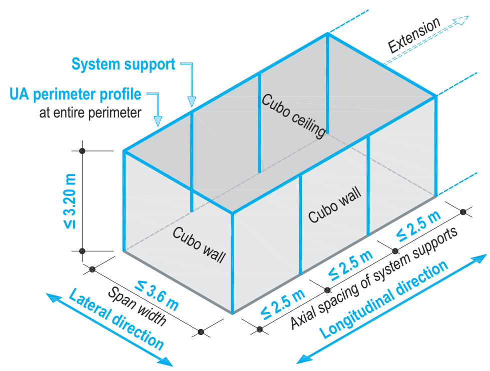
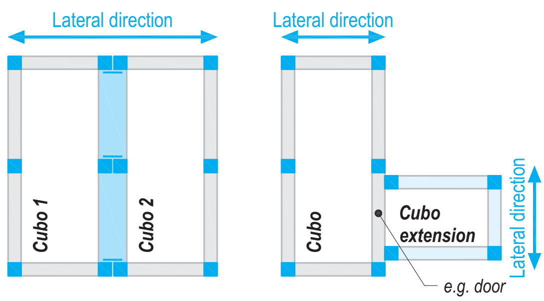
<tr><td>



Height ≤ 3200 mm
Width ≤ 3600 mm
Length ≤ 2500 mm x n

</td><td>



</td></tr>
</table>

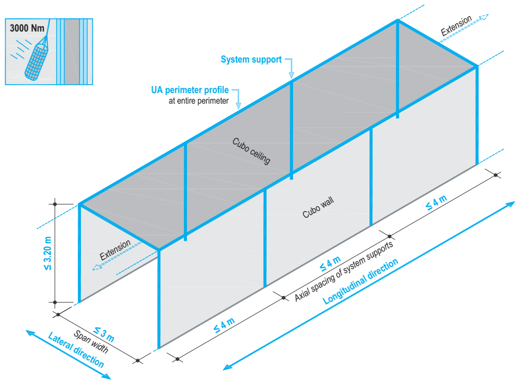
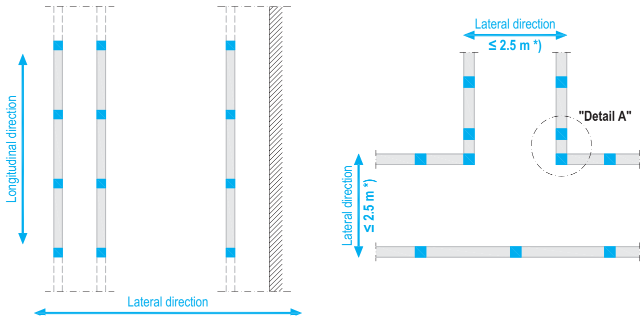
System / Construction data	Application variants
Cubo Escape Tunnel  Height ≤ 3700 mm Width ≤ 3000 mm Length ≤ 4000 mm x n	

Fig. 3.20: Visual loading test of system Cubo /Knauf Gips KG/

Fig. 3.21: Sanitary cell in the airport Berlin - Tegel /Knauf Gips KG/

Fig. 3.22: A 31.5 m long and 3.65 m wide Cubo construction in fire-resistant design in the attic for subsequent installation of the complete ventilation equipment in the old local court building in Offenbach (during installation) /Knauf Gips KG/

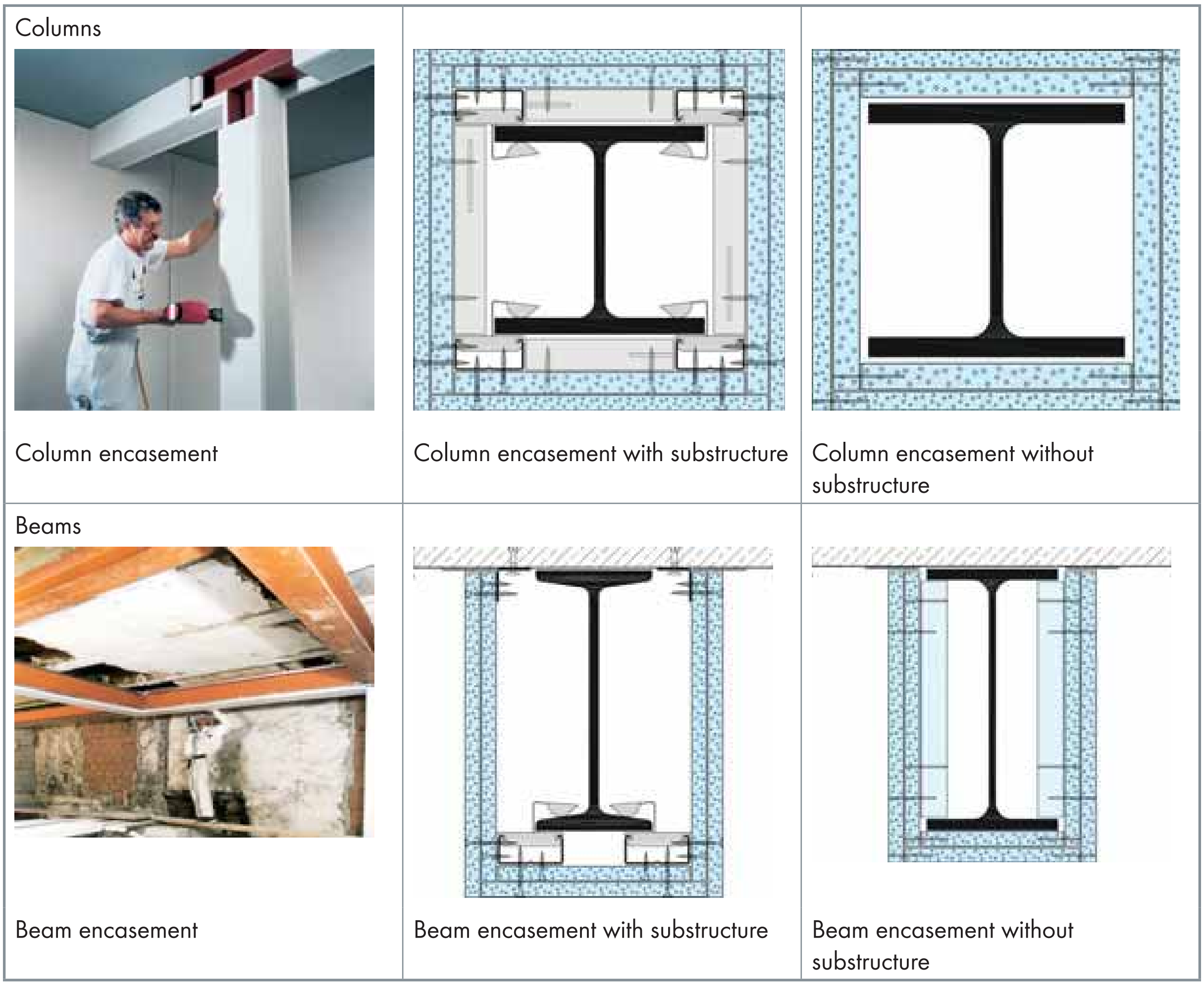


Fig. 3.23: Application examples of columns and beams in comparisons with and without a grid using Fireboard and double layer cladding /Knauf Gips KG/

also has the mechanical properties of a firewall with the classification 90 minutes and non-combustible as well as an impact stress resistance of 3000 Nm.

The advantageous characteristics of the room-in-room system can be described as follows:

- Modular system
- Manual assembly of industrially pre-fabricated semi-finished products
- Low weight
- Simple anchoring and connection elements
- Self-supporting frame
- Support of static additional loads (permanent loads, live loads)
- Simple and flexible integration of functional components such as doors, windows, sanitary fittings, etc.
- Detachable / convertible for use in manufacturing plants
- Variable sound insulation of the room cell for noise protection from the interior to exterior and vice versa with possible complete decoupling from the building
- Sound absorption in the room cell
- Variable fire resistance up to the firewall quality from the interior to the exterior (e.g. store rooms for fire hazard materials) and / or from the exterior to interior (e.g. "escape route")

3.2.6 Encasement for columns and beams

Columns and beams are mainly made of reinforced concrete as well as steel and wood, depending on the design. Steel and wood are critical in terms of their performance in a fire and must generally be protected by an encasement of non-combustible materials in order to guarantee the required fire resistance of these supporting structures.

Tab. 3.7: *Minimum thicknesses of Fireboard boards to provide a fire resistance for steel columns and beams /Knauf Gips KG/*

Knauf Fireboard cladding for steel columns K253										
Fire resistance class	**U/A ratio of the steel profile (m^{-1}) at board thickness (mm)**									
	15	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**
30 min.	≤ 210	≤ 300								
60 min.	≤ 46	≤ 100	≤ 230	≤ 300						
90 min.		≤ 40	≤ 140	≤ 170	≤ 260	≤ 300				
120 min.			≤ 38	≤ 68	≤ 110	≤ 180	≤ 280	≤ 300		
180 min.					≤ 35	≤ 50	≤ 76	≤ 105	≤ 150	≤ 210

Knauf Fireboard cladding for steel beams K252									
Fire resistance class	**U/A ratio of the steel profile (m^{-1}) at board thickness (mm)**								
	15	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**
30 min.	≤ 300								
60 min.	≤ 170	≤ 300							
90 min.	≤ 48	≤ 130	≤ 270	≤ 300					
120 min.		≤ 50	≤ 100	≤ 180	≤ 300				
180 min.				≤ 45	≤ 80	≤ 125	≤ 190	≤ 260	≤ 300

Steel beams and supports

The enhancement of the fire resistance is based on the fact that the encasement greatly delays the temperature increase of the steel profiles and the component can retain its structural stability for a longer period. Gypsum board and Fireboard are effective.

In the unprotected state, steel columns achieve the critical temperature of about 500°C due to the rapid heat up (at this temperature for common constructive steel, the yield strength of the existing steel stress is reduced, i.e. the construction fails) and therefore only have a fire resistance < 30 min.

The fire resistance to be achieved by the encasement is dependent on the following factors: Mass of the steel profile, cross-section of the steel profile A (cm^2)

- surface on which the heat acts, generally the circumference of the steel profile or the cladding surface U (cm)
- thickness of the cladding (material dependent)

The rating of the cladding is generally according to the ratio "circumference to cross-section area" of the steel profile (U/A ratio). According to the calculated U/A ratio and the required fire resistance, the material thickness for the required fire resistance relevant cladding can be selected acc. to Tab. 3.7 and 3.8, /3.10/.

Cladding made of gypsum boards is generally anchored using drywall screws to the metal profile framework encompassing the steel profile. With Fireboard, there is, in addition to this construction variant, the option of omitting the framework for front-sided and surface screw fixing of the boards under one another, and in any case, narrower cladding thickness when compared to gypsum boards is required providing a more economically beneficial narrow construction (Fig. 3.23).

Complete supporting structure made of steel

For fire related upgrading of complete frameworks, e.g. lattice or truss type bracing where the cladding of the individual construction elements (columns, crossbars, struts) is only conditionally possible or too complex, an engineered fire protection solution in accordance with the "shaft wall principle" is prudent. To suit the required level of protection, single or multi-layer cladding is arranged in front of the supporting structure. The attachment of the cladding boards (gypsum boards) is either directly on the support frame with or without stud frame or as

Tab. 3.8: Minimum thicknesses of gypsum boards to provide a fire resistance for steel columns and beams /Knauf Gips KG/

GKF cladding for steel columns		
Fire resistance class	**Board thickness GKF mm**	**U/A m^{-1}**
30 min.	12.5 [2)]	
60 min.	12.5 + 9.5 [1)]	
90 min.	3 x 15	≤ 300
120 min.	4 x 15	
180 min.	5 x 15	
1) The room side 9.5 mm thick panelling may also consist of wall boards (GKB) 2) Replacable with ≥ 18 mm thick wall boards (GKB)		

GKF cladding for steel beams		
Fire resistance class	**Board thickness GKF mm**	**U/A m^{-1}**
30 min.	12.5	
60 min.	12.5 + 9.5 [1)]	
90 min.	2 x 15	≤ 300
120 min.	2 x 15 + 9.5 [1)]	
1) The room side 9.5 mm thick panelling may also consist of wall boards (GKB)		

Fire protection with Fireboard during construction

After completion

Fig. 3.24: Fire protection cladding with Fireboard, Knauf system in the Frauenkirche Dresden /Knauf Gips KG/

an independent shell in accordance with the prevalent constructional constraints.

The cladding thicknesses are required to comply with a fire resistance classification of 30 to 180 minutes. /3.22/. With professionally applied cladding (observe the permissible cladding widths of the boards, max. 1000 mm at 30 mm board thickness), at the stated fire resistance even with cladding applied directly to the steel frame, the maximum surface temperatures on the steel surface will only reach about 200 °C, i.e. the construction retains its load-bearing capability in this time period.

Fire protection using this construction principle was implemented in the interior of the Frauenkirche in Dresden, Germany. Fig. 3.24 shows the cladding of the galleries with Fireboard.

3.2.7 Façade constructions

Lightweight constructions can be effectively and safely used in the area of the envelope of the building. In this case however, because the façade is exposed to moisture, the use of cement boards as exterior cladding and gypsum boards as interior cladding is practical.

The Knauf system "Knauf exterior walls with Aquapanel®" is such an exterior wall system on the market. In this system, cementitious panels are applied as exterior wall cladding /3.23/. The self-supporting frame consists of a single or double framework made of metal profiles (CW or UA profiles). Gypsum boards are used for internal cladding. Mineral wool is applied in the wall cavity as thermal insulation.

These exterior wall constructions can be applied as "integrated" walls or "curtain" walls.

With the integrated application versions, the exterior wall is placed on the load-bearing slab and the surrounding perimeter connections are attached directly to the supporting frame.

On the curtain wall versions, which can be applied as constructions with a double frame, one part of the frame is mounted on angle profiles, additionally mounted and attached to the ceiling slab of the main structure. For renovations, the implementation as a curtain façade or wall is possible /3.23/. Application variants are shown in Tab. 3.9.

Under the cladding of cement boards, a water-vapour transmission climatic membrane is applied as a water barrier to avoid ingress of water in the substructure and as a water channelling layer to drain away any water that has entered the structure.

For the metal profiles used, enhanced corrosion protection requirements compliant with the local climatic conditions and the requirements of the building authority must be observed.

The board joints are filled after attachment and then coated on the entire surface with an external render system.

In addition to structural stability, the exterior wall constructions must also comply with fire protection, thermal insulation, moisture protection and sound insulation criteria.

3.2.8 Complete buildings with structural lightweight steel construction

Structural metal frame wall panels

Structural metal frame wall panels are walls that feature a systematic bracing function for horizontal loads and also for vertical loads on the wall level. Here the gypsum board cladding reinforces the metal panels for horizontal loads; the vertical loads are supported solely by the frame.

Structural metal frame wall panels are used particularly with pre-fabricated houses. The construction with these panels is similar to a non-load bearing partition. However, please note that there are divergent constructive details such as special requirements for fasteners, clearances and connections. The cladding can also be considered on one wall side only for supporting the shear stresses. The boards are shear-resistant connected to the frame all the way round.

Lightweight steel construction

Lightweight steel construction is similar to structural wood frame panel construction. Buildings of lightweight steel design consist of a steel skeleton (see Fig. 3.25) to support the vertical loads and bracing walls and ceiling panel to transfer the horizontal forces. These wall and ceiling panels consist of thin sheet metal profiles and a

Tab. 3.9: Exterior wall constructions

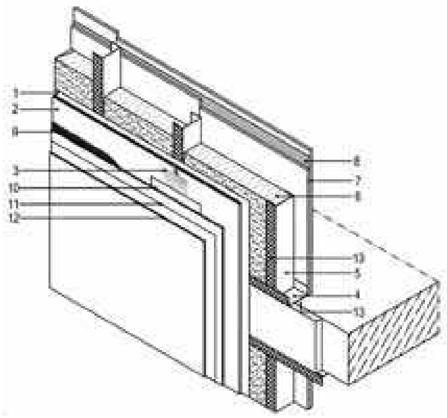
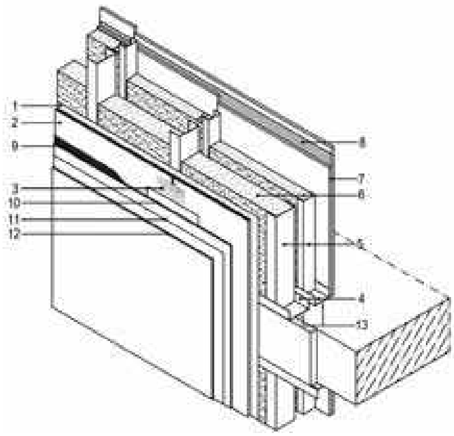
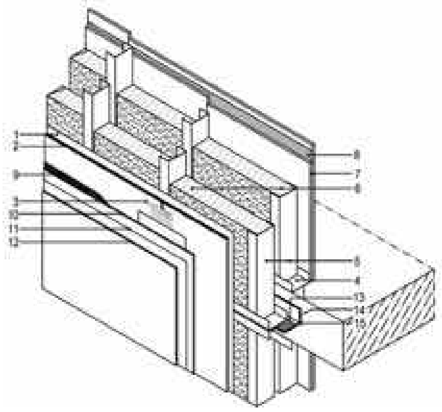
System	Design	Properties
Integrated wall Single stud partition		• Single stud partition • C profile studs • Total thickness 75 – 150 mm • Fire resistance 30 – 120 minutes • Sound reduction index $R_{w,R}$ = 43 – 48 dB
Integrated wall Double stud partition		• Dual stud partitions • C profile studs • Total thickness 200 – 250 mm • Fire resistance 90 minutes • Sound reduction index $R_{w,R}$ = 51 – 58 dB • Heat transfer coefficient • U = 0.20 – 0.28 W/ m^2K
Curtain wall Double stud partition		
Legend:	1 Climatic membrane 2 Cement board 3 Joint tape / joint filler 4 UW metal profile 5 CW or UA metal profile 6 Mineral wool 7 Gypsum board 8 Vapour barrier 9 Plaster system 13 Thermal decoupling 14 Thermal decoupling	

Fig. 3.25: Building with lightweight steel construction design /Knauf Gips KG/

bracing cladding of gypsum, gypsum fibre, cementitious or structural wood frame panel construction panels. The stability endangered thin metal profiles are protected from stability failures by the cladding. Lightweight steel construction offers a host of system-related benefits:

- Very low weight
- Excellent stability/self-weight ratio
- Dimensional tolerance
- Good building acoustic properties
- Facilitates fast assembly on-site
- High recycling and reprocessing potential of all materials used in the system
- Non-combustible dependent on the board material (building material class A), no increase of the fire load due to the construction
- Sophisticated jointing and connection elements
- Suitability for pre-fabrication
- Suitability for building extension (vertical extension) with limited load stability

Lightweight steel constructions are used for buildings with a limited number of storeys, generally up to 4 storeys as well as façade elements and room-in-room systems.

3.3 Materials for drywalling construction

3.3.1 Boards for cladding

The most important board types for drywalling are:

- Gypsum boards (previously designated as gypsum board)
- Gypsum fibre boards
- Wooden composite boards (chipboard, wooden fibre boards, plywood, OSB board, mineral-based chipboard)
- Cement boards

The greatest range of applications in drywalling is

Gypsum slurry is applied to the board liner

The board line is formed

The gypsum core sets on the setting belt

The boards are cut to size

Fig. 3.26: Manufacture of gypsum boards in highly efficient manufacturing plants /Knauf Gips KG/

certainly for the gypsum-based boards of the product groups gypsum boards and gypsum fibre boards. The annual global production exceeds, for example, the 3 billion m^2 threshold. The other versions therefore relate particularly to the application of this product group and the application in areas subject to moisture and in the exterior wall applications on cementitious boards.

Gypsum boards

Gypsum boards consist of a gypsum core, whose surfaces and long edges are enclosed by a board liner as a fixed composite and that can be manufactured very effectively in industrial conveyor systems (Fig. 3.26).

The main board properties are determined by the composite effect of gypsum core and board liner. The board liner has the function of a tensile reinforcement /3.25/.

The board types are classified in standards such as EN 520 in Europe.

Gypsum boards - range of standard boards

For different respective functions, there are various board types, e.g. in acc. with the EN 520 that differentiate by the external board liner and the additives in the gypsum core. The board dimensions generally are

- a width of 1200 or 1250 mm, and for very thick and heavy boards 600 or 625 mm,
- with thicknesses depending on the type of 9.5 mm, 12.5 mm, 15.0 mm, 18.0 mm, 20.0 mm and 25.0 mm and
- lengths as standard dimensions of 2000 to 3000 mm, whereby special lengths can be provided by the manufacturer for large projects.

Tab. 3.10 shows the different common board types arranged according to their primary properties

The exemplary assignment of the board types to EN 520 is indicated in Tab. 3.11, while the latter are explained in Tab. 3.12.

Tab. 3.10: Gypsum board types

Board type	**Field of application / properties**
Wallboard	Gypsum wallboards for attachment to a flat surface substrate, are designed for use as dry lining to walls, cladding of drywall partitions or in the manufacture of composite panels. Wallboards not less than 12.5 mm thick can be installed in situ as wall lining or furring, ceiling lining, or as cladding of partition systems
Gypsum wallboard impregnated	Impregnated gypsum wallboard for wallboard application areas, however, with a reduced water absorption rate; especially for use in areas of higher humidity (kitchens, residential bathrooms, etc.) as well as a substrate for tiles. These gypsum boards feature a specially impregnated gypsum core and generally a green coloured board liner.
Fire-Resistant Board	Fire resistant gypsum wallboard is designed for the same applications as gypsum wallboard, where a higher degree of fire resistance is required, as well as for cladding walls. The gypsum core generally contains short glass fibres.
Fire-Resistant Board impregnated	Impregnated fire-retardant gypsum wallboard is designed for the same applications as fire resistant wallboard, where a reduced water absorption is required (delayed water absorption). These gypsum boards feature a specially impregnated gypsum core and generally a green coloured board liner.
Plaster base boards	Is primarily intended as a plaster base

Tab. 3.11: Gypsum board types acc. to EN 520

Board type acc. to EN 520
Wallboard, Types A, D, R, I
Wallboard impregnated, Types H2, DH2, H2R, H2I
Fire-resistant board, Types DF, DFR, DFI, DFIR
Fire-resistant board impregnated, Types DFH2, DF, H2R, DFIH2, DFH2IR
Plaster base boards, Type P

Within these board groups, there are further special board types with application specific properties, known on the market with the manufacturer specific names (Tab. 3.13).

Production line manufactured gypsum boards are made with various edge types (Fig. 3.27) and dimensions.

The most important properties of gypsum boards can be summarized as follows:

- The density is approx. 700 to 800 kg/m^3, with hard gypsum board up to approx. 1000 kg/m^3 and for sound insulation and X-ray shielding boards up to approx. 1300 kg/m^3
- The combination of gypsum and board liner achieves a high flexural strength and structural stability
- The mechanical properties are dependent on the direction; the paper fibres, which are primarily aligned in the longitudinal direction (in the same direction as the rear side printing on the board), result in a greater stability with loading in this orientation
- With stresses in the directions of the board liner fibres (support the gypsum board transverse to the fibre orientation – referred to as transverse supporting) higher flexural strength and lower deformation
- Generally building material class A2 (non-combustible),

Tab. 3.12: Gypsum board types acc. to EN 520

Board type EN 520	**Field of application / properties**
A Gypsum board	Gypsum boards for cladding of non-load bearing drywalling constructions and structural wood frame wall panels, as dry lining for walls, with a face to which suitable gypsum plasters or decoration may be applied. Fire protection requirements are only fulfilled to a limited extent.
H Gypsum board with reduced water absorption rate	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", "Type H2" or "Type H3". The boards can be used in areas of high humidity.
E Gypsum sheathing board for exterior wall elements	Gypsum boards specially manufactured to be used as sheathing board 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 a reduced water absorption rate. They shall have a minimum water vapour permeability
F Gypsum board with improved core adhesion at high temperature	Same as Type A but these boards have mineral fibres and/or other additives in the gypsum core to improve core cohesion at high temperatures (in the event of a fire).
D Gypsum board with controlled density	Same as Type A, but these boards have a controlled density enabling improved performance in certain applications to be obtained
R Gypsum board with enhanced strength	Gypsum boards with a face, to which suitable gypsum plasters or decoration may be applied, where higher strength is required, have both increased longitudinal and transverse breaking loads.
I Gypsum board with enhanced surface hardness	Same as Type A, but are used for applications where higher surface hardness is required.
P Boards intended to receive gypsum plaster	This board type has a face intended to receive gypsum plaster

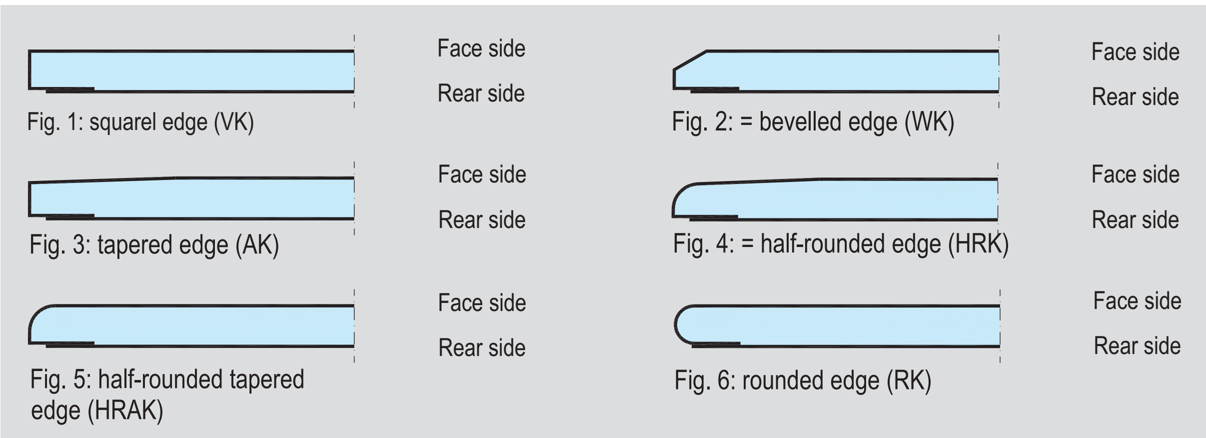


Fig. 3.27: Long edge joints of gypsum boards (Extract from EN 520)

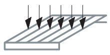
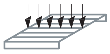
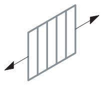
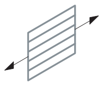
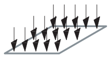
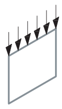
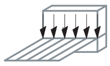
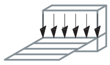
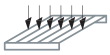
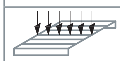
Tab. 3.13: *Special gypsum boards, Knauf*

Board name	Type acc. to EN 520	Special properties	Preferred application
Techniform Board 6.5 mm	D	• Gypsum board that can be shaped in the wet and dry state • When dry can be bent up to a radius of ≥ 1000 mm	• Design of curved walls and ceilings, rounded dormer features • Short flowing transitions with differing ceiling levels • Pre-fabrication of design units
Thermoboard, Thermoboard Plus 10 mm /3.26, 3.27/	DF	• Enhancement of the cooling and heating performance by up to 20 % in heating and cooling ceiling systems (system dependent) • Side lengths with pure cooling ceiling system up to 15 m possible	• Heating and cooling ceiling systems, particularly in office and commercial buildings as a smooth board or acoustic panel (perforated)
Diamant (Hard Gypsum Board) 10 mm; 12.5 mm; 15 mm; 18 mm /3.28, 3.29, 3.30/	DFH2I	• High mechanical performance strength (hardness, strength, high cantilever loads) • Higher density / area mass (approx. 12.5 kg/m^2 at t = 12.5 mm) with flexurally ductile structure	• Room-in-room system Cubo • Schools and sports facility construction • Hospital construction • Critical, highly frequented areas (e.g. corridors) • Areas of high humidity • Sophisticated sound insulation
Silentboard (sound shield board) 12.5 mm /3.31/	DF	• Highest density / area mass (approx. 17.5 kg/m^2) with flexurally ductile structure • Best sound insulation properties • Enhanced performance at low frequencies	• Constructional components with very high sound insulation • Slim sound insulating systems • Encapsulation of sound sources
Cleaneo (Cleaneo Acoustic) 12.5 mm /3.32, 3.33/		• Sound absorption • Reduces impurities in room air due to pollutants (e.g. emissions from cleaning and preserving agents) • Neutralizes unpleasant odours	• Reprocessing to acoustic boards • Cladding of ceiling systems (room acoustics, room design, improvement of air quality) in all buildings
Safeboard X-Ray Shielding Board 12.5 mm /3.34, 3.35/	DF	• X-Ray shielding without lead lining, lead equivalence of 12.5 mm board: 0.4 – 0.75 mm Pb dependent on the tube voltage	• X-Ray shielding in clinics, hospitals with furring, stud partitions and ceiling lining / suspended ceiling
Horizonboard 12.5 mm	DF	• Board with four-sided tapered edge	• Ceilings with high demands on the surface

no smoke production and no flaming debris / molten drips
- Good fire protection properties due to the crystallized water contained in the gypsum core (approx. 20 % of the gypsum core material)
- Building acoustics "flexurally ductile board"
- Fast absorption and release of water/water vapour (moisture regulating)
- Low expansion and shrinkage with climatic changes
- Simple processing by scoring/breaking or sawing without special tools

The technical – mechanical and building physical data acc. to EN 520 are summarized in Tab. 3.14.

Tab. 3.14: Average values of technical – mechanical and building physical properties of gypsum boards acc. to EN 520 /Knauf Gips KG/

Technical and mechanical data			
Stress	**Description / imposed load**		**Material value**
Flexural strength (thickness dependent) 1) σB		Transverse to paper fibres	≥ 3.0 – ≥ 7.9 N/mm²
		Parallel to the paper fibres	≥ 3.0 – ≥ 7.9 N/mm²
Tensile strength σB		Transverse to paper fibres	1.0 – 1.2 N/mm²
		Parallel to the paper fibres	1.8 – 2.5 N/mm²
Compressive strength σB		Perpendicular to the surface	5 – 10 N/mm²
		Parallel to the surface	5 – 10 N/mm²
Shear strength σB		Lateral to paper fibres	3.0 – 4.5 N/mm²
		Parallel to the paper fibres	2.5 – 4.0 N/mm²
Modulus of elasticity (bending tensile strength) 2) E		Transverse to paper fibres	≥ 2800 N/mm²
		Parallel to the paper fibres	≥ 2200 N/mm²
Surface hardness (Brinell)			10 – 18 N/mm²
Building physical data			
Equilibrium moisture at 20 °C, 65 % relative humidity			0.6 – 1.0 % by mass
Thermal conductivity acc. to EN 12524			0.25 W/mK
Thermal expansion coefficient at 50 – 60 % relative humidity		Dependent on the board thickness	0.013 – 0.020 mm/m
Water vapour diffusion resistance factor μ		At a density of 900 kg/m³ acc. to EN 12524	4 wet / 10 (dry)
Shrinkage and expansion		Change of the relative humidity by 40 % at 20 °C	approx. 0.2 mm/m
Capillary absorption height of water via paper liner free edge (cut edge) after 2 hours immersion in water		impregnated	70 – 80 mm
		impregnated	approx. 5 mm
Heat threshold			max. 50 °C
Combustibility		acc. to EN 520	A2-s1,d0

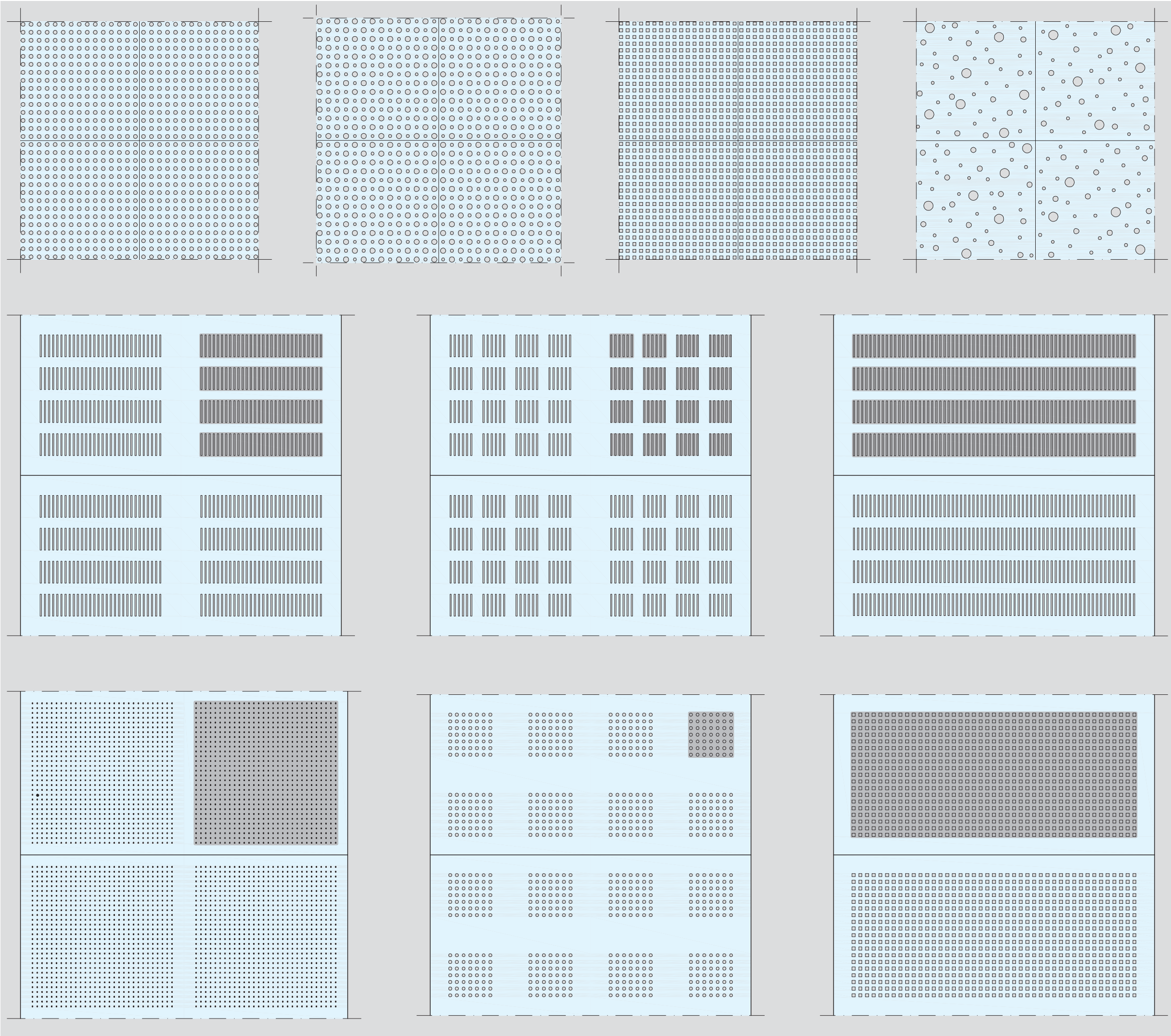


Fig. 3.28: Perforation design of gypsum boards /Knauf Gips KG/

Gypsum boards – range of reprocessed boards

By factory reprocessing of the gypsum boards produced on production lines, a comprehensive range of specific products can be manufactured.

- *Reprofiled boards*: Have cut edges and ends all around with differing formats.
- *Perforated boards, slotted boards*: For acoustic ceilings generally laminated on the rear with fleece and used as a design element (Fig. 3.28) /3.32/.
- *Laminated gypsum boards*: Gypsum boards can be laminated for special purposes with fixed layers, foil or plastic materials, e.g.:
- Foil made of plastic or aluminium as a vapour barrier
- Lead foil as X-Ray shielding
- Fleece for sound absorption
- Fleece for shielding against high-frequency electromagnetic waves ("electrosmog")
- *Mitred gypsum boards*: For manufacturing pre-fabricated components and high quality edge types (Fig. 3.29) /3.37/
- *Arched gypsum boards*: For special design in ceiling and wall areas (Fig. 3.30) /3.37/

Gypsum board insulation composite panels, e.g. acc. to EN 13950: With an insulating layer glued onto the rear side of the board made of mineral wool, EPS, XPS or PUR, particularly for thermal insulation associated with remodeling.

Gypsum fibre boards

Gypsum fibre boards consist of a mix of gypsum and cellulose fibres (recycled paper). The paper fibres act as reinforcement in these boards /3.38/. The level,

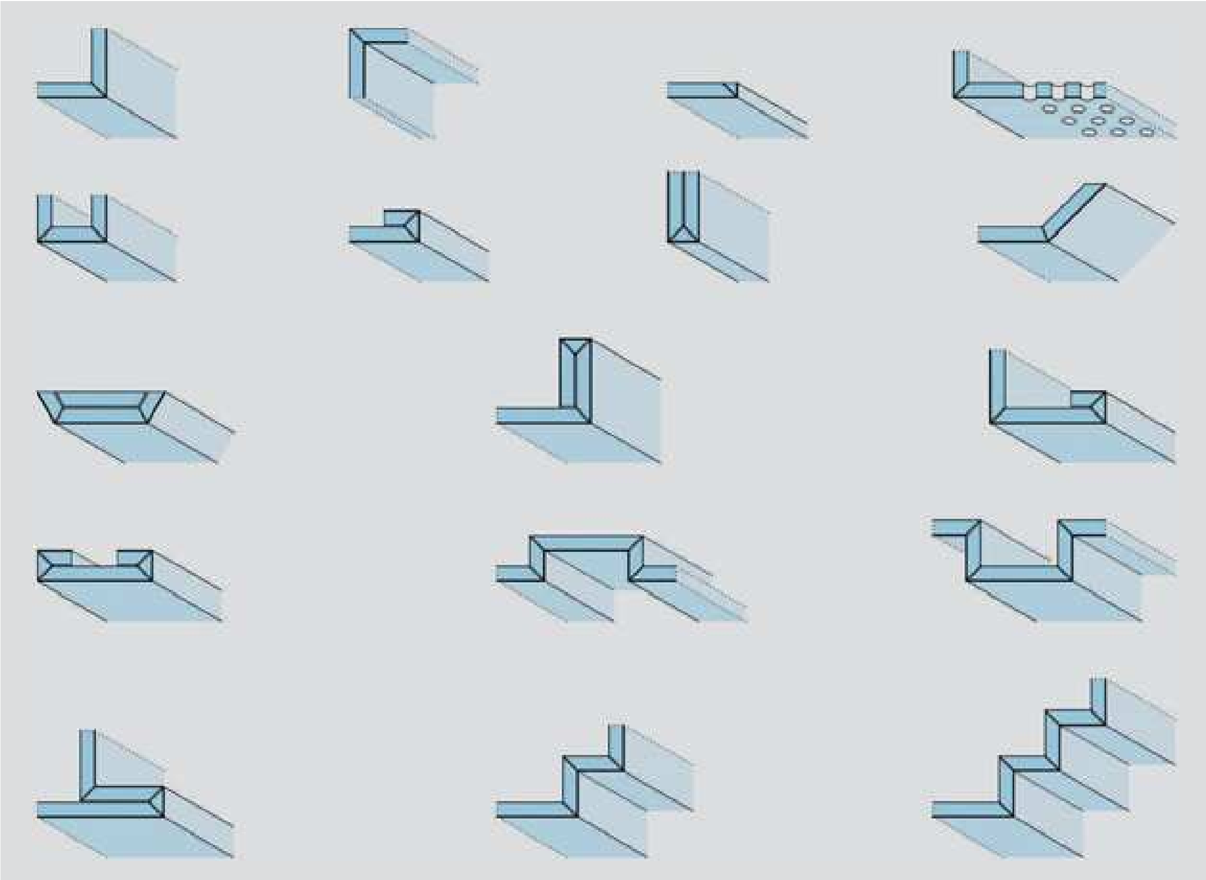


Fig. 3.29: *Mitred elements as design units (mitering technology) /Knauf Gips KG/*

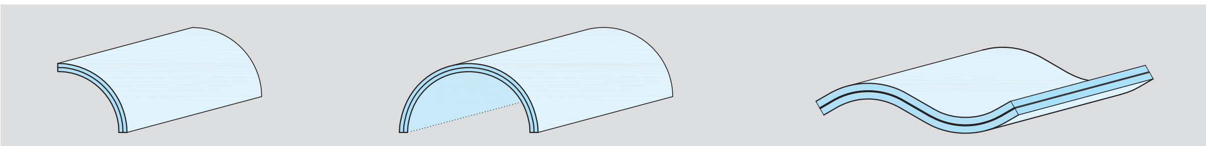


Fig. 3.30: *Pre-arched gypsum board elements as design units (moulding technology) /Knauf Gips KG/*

right-angled and four-sided sharp-edged boards are manufactured on highly-efficient conveyor systems according to different processes.

There are different board types for different intended purposes, e.g. acc. to EN 15283-2.

- Gypsum fibre boards GF
- Gypsum fibre boards with reduced water absorption rate (GF-H)
- Gypsum fibre boards with reduced surface water absorption (GF-W1; GF-W2)
- Gypsum fibre boards with enhanced density (GF-D)
- Gypsum fibre boards with enhanced surface hardness (GF-I)
- Gypsum fibre boards with enhanced strength (GF-R; GF-R2)

General characteristics of gypsum fibre boards:

- The board dimensions generally are
  - 1000 mm to 1260 mm, for very thick and heavy boards 600, 620 or 625 mm
  - Preferred thicknesses 10 mm, 12.5 mm, 15 mm, 18 mm, 20 mm and 30 mm
  - in lengths as standard dimensions of 1500 – 3100 mm, whereby the special lengths and widths can be provided by the manufacturer for large projects and special applications
- The density is generally approx. 1000 to 1250 kg/m^3, with high density board materials up to 1500 kg/m^3
- The mechanical properties are approximately directional dependent; acc. to EN 15283-2, the

following values (mean values) are determined for the flexural strength dependent on the board type

- ◦ Gypsum fibre boards, Type GF:
  5.5 N/mm^2 (board thickness < 18 mm)
  5.0 N/mm^2 (board thickness ≥ 18 mm)
- ◦ Gypsum fibre boards, TP GF-R1: 8.0 N/mm^2
- ◦ Gypsum fibre boards, TP GF-R2: 10.0 N/mm^2
- Other non-standardized strengths are approx. /3.25/:
- ◦ Compressive strength ≥ 7.5 N/mm^2
- ◦ Tensile strength ≥ 2.2 N/mm^2
- ◦ Shear with load perpendicular to board level ≥ 3.5 N/mm^2
- ◦ Surface hardness ≥ 25 N/mm^2 (with higher density up to 45 N/mm^2)
- ◦ Bending E-modulus ≥ 3800 N/mm^2
- Building material class A2 (non-combustible), no smoke (s1) production and no flaming debris / molten drips (d0)
- Good fire protection properties due to the crystallized water (approx. 15 % of the mass)
- Building acoustics "flexurally ductile board"
- Low level of expansion and shrinkage with climatic changes (0.25 – 0.6 mm/m with changes in air humidity of about 30 %), however less favourable compared to gypsum boards
- Simple to process with a saw and milling without special tools
- When the boards are sufficiently thick and the densities exceed 1200 kg/m^3, it is possible to produce special application-friendly edges

Gypsum fibre boards are particularly suited due to their good surface hardness and high wear resistance for pre-fab floor screeds extending up to hollow floor/raised access floors (boards with density 1500 kg/m^3).

Gypsum boards with mat reinforcement

These boards consist of a modified gypsum core, whose surfaces and long edges are covered by a glass mat layer. The boards are manufactured industrially on highly-efficient conveyor systems, e.g. according to EN 15283-1. An example of this board type is the board type Fireboard of Knauf, used as a special board particularly in fire protection applications /3.45/.

The board properties can be described as follows:

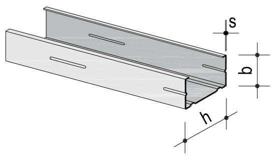
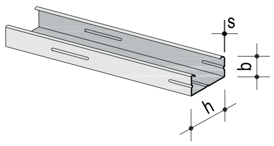
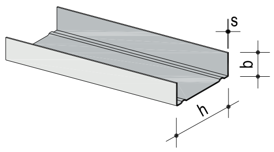
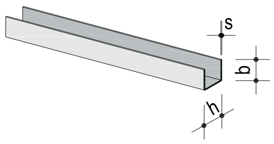
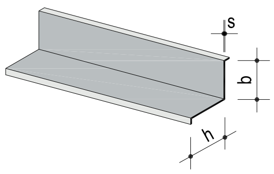
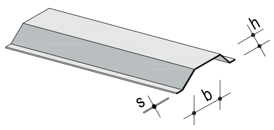
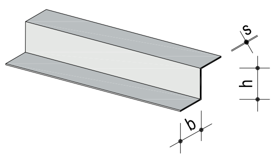
- The board dimensions are
- Board width 1200 mm or 1250 mm
- Preferred thicknesses 12.5 mm, 15 mm, 20 mm, 25 mm and 30 mm
- The standard length has dimension of 2000 mm, whereby the special lengths and widths can be provided by the manufacturer for large projects and special applications
- The density is approx. 800 kg/m^3
- The mechanical properties are direction dependent; the flexural tensile strength, for example, with a 20 mm thick board for boards with a density of approx. 820 kg/m^3
- ◦ At loading transverse to the direction of the fibres ≥ 4.9 N/mm^2
- ◦ Loading parallel to the direction of the fibres ≥ 1.8 N/mm^2
- Building material class A1 (non-combustible without combustible components)
- Good fire protection properties due to the crystallized water contained in the gypsum core (approx. 20 % of the gypsum core material)
- Minimal crack formation and thus a high stability even after long exposure to fire (dehydrated gypsum core, crystallized water has vaporized) on the component clad with Fireboard (continues to act as a heat shield even after the dehydration of the gypsum core)
- Low expansion and shrinkage with climatic changes
- Simple to process with scoring / breaking or a saw without special tools

Cement boards

Cement boards are water-resistant boards suitable for external application (e.g. façades) and for internal application, particularly for commercial wet areas (e.g. swimming pools, saunas, industrial kitchens, colliery shower rooms, laboratories). An example of this type of board is AQUAPANEL® Cement Board, Knauf system /3.39, 3.40, 3.47/, which as

- Cement Board Indoor has a board thickness of 12.5 mm for internal fitting, for cladding of ceilings or wall systems

Tab. 3.15: Examples for standard profiles made of sheet metal for wall and ceiling constructions

Profile type					**Example of use**	**Designation**
	Sections	**Height h (mm)**	**Width b (mm)**	**Thickness s (mm)**		
	C	100	50	0.6	Stud	C50/100/50
	C	60	27	0.6	Ceiling	C27/60/27
	U	75	40	0.6	Perimeter profile partition	U40/75/40
	U	27	28	0.6	Perimeter profile ceiling	U28/27/28
	L	50	50	0.5	Corner	L50/50
	W	50	50	0.5	Ceiling	W10/25/50/25/10
	Z	48	40	2.0	Bar	Z40/48/40

- Cement Board Outdoor has a board thickness of 12.5 mm for external fitting, for façade cladding or for ceilings or other exterior applications
- Cement Board Floor has a board thickness of 22 mm or as a composite element with mineral wool has a board thickness of 33 mm for internal application, or as a cementitious pre-fab floor screed

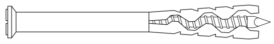
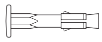
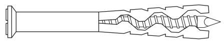
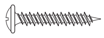
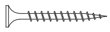
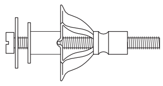
The cement board consists of a core made of Portland cement and aggregates as well as a glass gauze fabric on the front and rear. The longitudinal edges are half-rounded and reinforced.

The boards are manufactured industrially on highly-efficient conveyor systems.

The board properties for the example AQUAPANEL® Cement Board can be described as follows:

- The board dimensions are
  - Board width generally 900 mm
  - Preferred thickness 12.5 mm
  - In the length as standard preferred dimensions 1250 mm and 2500 mm
- Dry density is approx. 1050 kg/m^3 (Indoor) – 1150 kg/m^3 (Outdoor)

Tab. 3.16: Anchors for drywalling /Knauf Gips KG/

Substrate	**Anchoring material**	
Reinforced concrete	Nailable plug	
	Ceiling steel dowel	
Masonry	Nailable plug	
Metal stud partitions	Multi purpose screw (screwed into stud)	
	Drywall screw (screwed into stud)	
	Cavity dowel	

- Mechanical properties are practically direction-independent; the flexural strength at load is transverse to the board level at ≥ 7 N/mm^2
- Water vapour diffusion resistance μ = 50 (Indoor) – 66 (Outdoor)
- Low expansion and shrinkage with climatic changes Length change at 65 % to 85 % relative humidity: 0.25 mm/m (Indoor), 0.23 mm/m (Outdoor)
- Thickness change at 65 % to 85 % relative humidity: 0.1 % (Indoor), 0.2 % (Outdoor)
- Building material class A1 (non-combustible without combustible components)

3.3.2 Substructure

For stability reasons, components made of thin boards require a bracing substructure if they are not attached to the substrate by adhesion or mechanical anchoring. The substructure and cladding determine the structural properties (e.g. stability with static and dynamic loads, deformation) of the lightweight component and also influence the sound insulation and fire resistance.

Profiles

The metal profiles that are commonly used for drywalling are manufactured from corrosion protected thin sheet metal by cold-forming.

The profiles summarized in Tab. 3.15 are standard profiles, which occur in practice in the most diverse variants with respect to their application details and that can be combined with other profiles in a system. The objective is to achieve particularly good system properties with respect to sound insulation and fire resistance, structural performance etc. as well as to ensure the widest possible spectrum of system applications, adapted to the most varied geometric site application conditions in the building field. Thus, for example, the following special profiles are offered by Knauf for drywalling constructions:

- Hat-shaped channel 98 x 15 x 0.6 mm for slim ceiling design
- Resilient Channel 60 x 27 x 0.6 mm for ceiling lining and furring with enhanced sound insulation
- Flexible connection profile Sinus for curved walls acc. to Fig. 3.31 /3.46/
- Concave and convex arched CD 60 profiles for domes and cupolas
- UA profiles with 2 mm gauge for door build-in and for higher static performance strength

Anchoring, fasteners, connectors and hangers

Further important construction components for drywalling constructions are the fixing materials for anchoring drywalling constructions to flanking components (anchors), construction elements for suspended ceilings from the basic ceiling (hangers), connection elements for interconnection of the elements of the drywalling substructure to one another (connectors) and fasteners for mechanically fixing the cladding to the substructure.

As these materials connect the individual components of the drywall systems with each other as well as the entire construction component with the flanking components,

Fig. 3.31: Walls with radii from 125 mm with profile Sinus, Knauf system /Knauf Gips KG/

they are of great importance for ensuring the required properties. This applies particularly for ensuring the load-bearing capacity.

In order to avoid application faults, the system components provided by system providers should not be exchanged with other materials.

Anchors

The anchoring of drywall structures to flanking components is carried out using anchors suitable for the respective base substrate (Tab. 3.16).

For external walls, the anchoring is carried out exclusively by means of anchors made of steel, like for example, ceiling 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 approved for the respective application. 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.

A special approval for the application in seismic areas is strongly recommended.

Suspenders for suspended ceilings

For the simplest ceiling linings, the furring channels can be attached directly to the basic ceiling using dowels or screws to suit the existing substrate. Variable subceiling constructions with regard to the suspended height (usage of the plenum) can be implemented by suspensions.

A differentiation is made with adjustable brackets between universal brackets (also with sound insulation decoupling by rubber buffers) and variable height adjustable suspenders. An overview of the suspenders from Knauf can be found in Tab. 3.17 with specification of the minimum suspension heights of the system /3.9/. The load bearing capacity of the hanger must be determined, e.g. acc. to EN 13964.

Universal brackets / damping universal brackets and suspender systems with Nonius top are compression resistant suspension systems, which can also be used when the suspended ceiling is subject to pressure loads (e.g. wind loads and earthquake loads). Universal brackets / damping universal brackets are also suitable for wall linings.

Connectors

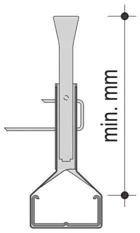
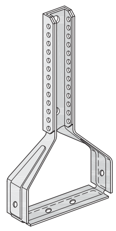
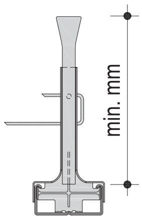
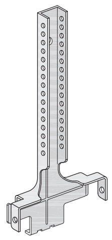
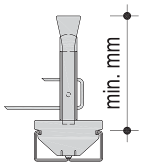
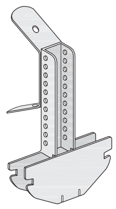
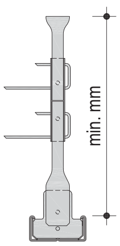
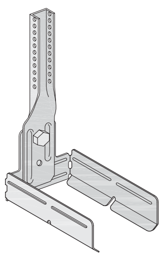
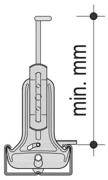
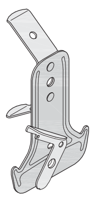
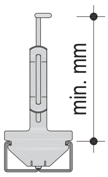
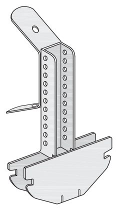
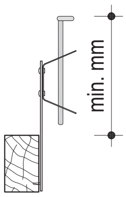
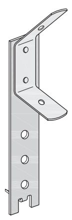
Profile connectors, in particular for suspended ceilings, are required on the one hand for the joint connection of profiles and on the other hand for the connection of profiles that run perpendicular to each other on a single level ceiling grid, for connection of both profile layers on a double level profile grid, or for the implementation of steps in the ceiling. Tab. 3.18 shows a summary of the most important connectors of the Knauf systems /3.9/.

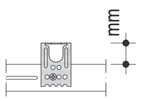
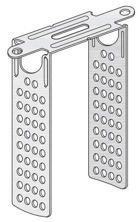
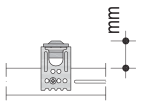
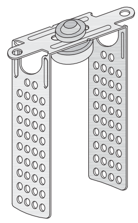
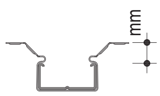
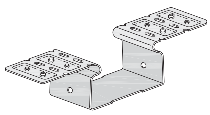
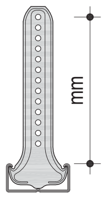
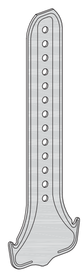
There are three methods of connecting metal profiles to each other: screws, rivets and crimping.

The screw connection is important, particularly for ceiling constructions in dynamic load scenarios (e.g. earthquakes). For direct screw fastening of the profiles (e.g. connection points for connection and furring profiles of free spanning ceilings) or the additional screw fastening of connectors or suspenders to the profile, depending on the metal gauge, metal screws with fillister heads and sharp or cutting point are used.

For constructional connections (not for load transfer), as an

Tab. 3.17: Hangers from subceilings /Knauf Gips KG/

Product sketches	**Detailed sketch**	**Description**	**Load capacity kN**	**Min. suspension height mm**
		Nonius stirrup	0.40	130
		Nonius hanger	0.40	130
		Combo hanger for CD 60x27	0.40	130
		Universal connector for CD 60x27	0.40	130
		Ankerfix with interlock for CD 60x27	0.25	110
		Combo hanger for CD 60x27	0.25	110
		Rapid wood hanger for wood	0.25	110

Product sketches	**Detailed sketch**	**Description**	**Load capacity kN**	**Min. suspension height mm**
		Universal bracket	0.40	5-180
		Damping universal bracket	0.40	5-180
		Clip fastener	0.15	7-27
		Anker hanger clip	0.25	up to 110

alternative to using screws, rivets can also be used, or the profiles can be crimped to one another. The disadvantage, however, is that this connection is very difficult to reverse and cannot be done without destroying it.

Board fasteners

Fasteners, e.g. acc. to EN 14566, are suitable for attachment of board materials to the metal substructure frame.

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 also 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.

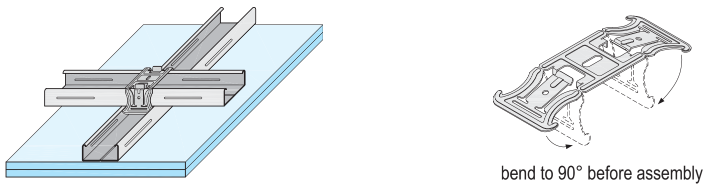
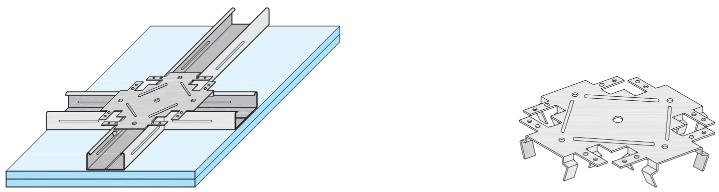
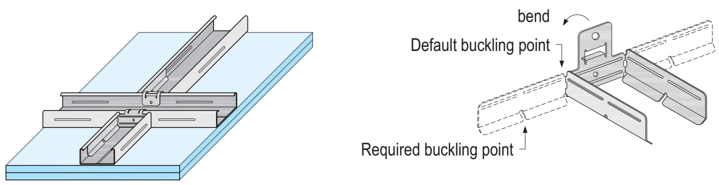
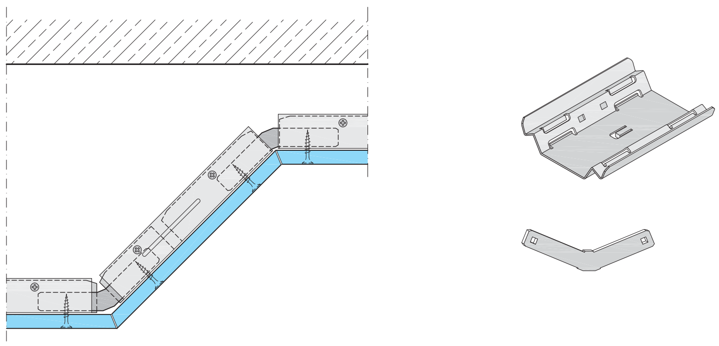
In the area of the ceiling linings, it is possible to fasten the cladding without a substructure directly on to level basic ceilings made of wooden beams or steel sheeting (e. g. trapezoid sheet ceilings). Drywall screws are also used to this purpose.

Tab. 3.19 shows the different types of drywalling screws and their applications.

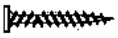
To ensure the load bearing capacity of the connection between the boards and the substructures, minimum penetration depths of the screws into the substructure must be complied with, and the predefined max. screw clearances to one another may not be exceeded. For non-load bearing walls, ceiling linings and subceilings, the required penetration depth of the screws in metal profiles is 10 mm.

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

Tab. 3.18: Profile connector for subceilings, Knauf system /Knauf Gips KG/

Detailed sketch	**Designation / purpose**
	Intersection connector Connection in the case of double level profile grids
	Ankerwinkel clip Connection in the case of double level profile grids
	Flush connector Connection in the case of single level profile grids
	Universal connector Connection in the case of single level profile grids
	Multi connector with multi adapter Connection of profiles at any degree setting
	Angle connector 90° Connection of profiles in the case of 90° angles

Tab. 3.19: Drywall screws for fixing the cladding /Knauf Gips KG/

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
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 of the board	Apertura gypsum boards on metal substructure up to 0.7 mm gauge
XTN 	Drywall screw with sharp point and self-tapping special thread for hard gypsum board	Hard gypsum boards (Diamant) on metal substuctures up to 0.7 mm gauge
Vidiwall screws 	Trumpet head with milling ribs, pin point	Gypsum fibre board on metal substructure up to 0.9 mm gauge
Screws for gypsum fibre pre-fab screed  	Trumpet head	Connection of gypsum fibre pre-fab screed units 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
AQUAPANEL® Maxi Screw SB (25/39) 	Screw head 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 pre-fabricated screed units in the area of the tier edge joints

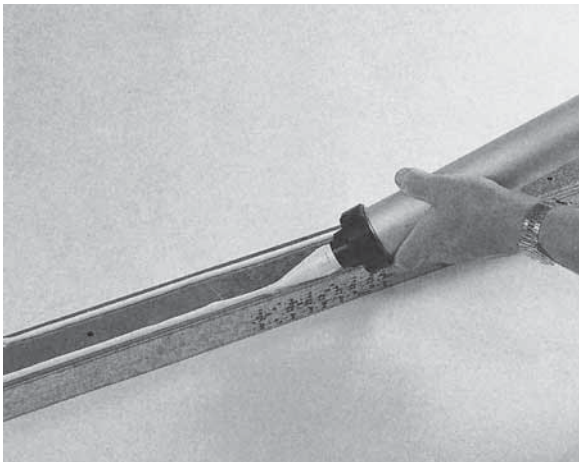


Fig. 3.32: Acoustical sealant application with application extrusion application on a connection profile

3.3.3 Insulation materials

Insulation materials in drywalling are used to improve the thermal insulation, as an absorber in noise and sound absorbing constructions, as impact sound insulation in flooring and to improve the fire resistance of building components in conjunction with board materials /3.41/. The demands made on the insulation material are very varied depending on the field of application. In thermal insulation, the thermal conductivity λ is the primary characteristic, where as in sound insulation the sound absorption capacity in dependence on the open pore structure of the insulation material is decisive, and in fire protection it is the combustibility of the insulation material. In the interior fittings of the building, the components are

Fig. 3.33: Machine application and smoothing of Readygips /Knauf Gips KG/

mainly provided considering the sound and fire protection demands. For this reason, in the vast majority of cases mineral wool insulation materials (glass wool or stone wool) are used. These insulation materials optimally meet both these requirements. They are open pore and feature excellent sound absorption properties and are generally non-combustible. Panel structure, panel material, panel mass, stability properties, etc. can vary greatly in the manufacturing process and can be readily adapted to the technological and technical requirements of the respective applications.

Cellular plastics (Polystyrene, Polyurethane, Phenol resin) are ideally suited for thermal insulation applications. They generally feature a predominantly closed cell cellular structure and feature good compressive strength. Cellular plastics made of Polystyrene, Polyurethane and Phenol resin are combustible. Elasticized polystyrene panels are suitable for footfall sound insulation.

Insulation materials are available in panel or roll format in thicknesses of 10 mm (acoustic insulation board) up to 240 mm (raftersqueeze insulation).

3.3.4 Others

Important components of drywalling systems are

- Sealing materials for sealing the connection joints of drywalling constructions to flanking components (acoustical sealant, sealing tape) /3.42/.
  In order to guarantee sealed joints (sound insulation) with uneven substrates, an acoustical sealant should be applied as a sealant (Fig. 3.32)
- Filler materials for sealing board joints and fasteners (e.g. screw heads). /3.43/
- Joint reinforcement tape for reinforcing joints /3.43/
  Higher safety for crack critical constructions is achieved with paper joint tape
- Finish filler materials to achieve a very level and smooth surface in preparation for covering with high-quality coverings /3.43, 3.44/ (Fig. 3.33)
- Adhesive compound for dry lining /3.43/
- Adhesive and basecoat as well as a mesh for façade layers as well as special materials that complement the drywall systems constructively and from the application side; sanitary accessories etc.

4 Seismic design of non-structural drywall systems

Tatiana Pali, Dominik Herfurth, Luigi Fiorino, Raffaele Landolfo

The international scientific and technical communities currently recognize the importance of proper seismic design of non-structural components. Previous earthquakes demonstrated the vulnerability of non-structural components to relatively low seismic intensity levels and showed that the damage or collapse of non-structural components might have severe consequences in terms of economic and social losses, limit the functionality of most affected buildings and pose a significant hazard to human life. Nevertheless, the issue of non-structural components has received less attention than the design of primary structural systems, thus leading to a lack of specific design provisions for these elements.

In this framework, lightweight drywall constructions using cold-formed steel members represent a valid alternative to traditional constructive systems for non-structural applications in seismic areas. In fact, these systems guarantee a good seismic behaviour with respect to damage limit states, mainly thanks to their lightness and low stiffness. For this reason, the current discussion explores in depth the issues and procedures related to the non-seismic and seismic design of non-structural drywall systems. In particular, the seismic behaviour of the non-structural drywall systems is investigated in terms of damage causes, typical damage and consequences. Furthermore, the seismic non-structural performance requirements are provided according to the performance-based design philosophy, whose main objective is to provide an adequate level of safety, but higher non-structural performance levels may be required for limiting the building damage or ensuring uninterrupted post-earthquake operations of the building. In this respect, damage mitigation measures, aimed at reducing the risks of lightweight steel partitions and suspended ceilings, are widely illustrated according to current standards. In addition, in an attempt to address the design shortcomings regarding the non-structural components, the ongoing research activities on drywall systems, especially for experimental purposes, are illustrated. Finally, a design example is illustrated in order to clarify the current seismic design approach.

4.1 The importance of non-structural components

The international scientific and technical communities currently recognize the importance of proper seismic design of non-structural components. The observation of their performance during past earthquakes, namely 1964 Alaska, 1971 San Fernando, 1989 Loma Pietra, 1994 Northridge and 1995 Kobe earthquakes /4.1/, has stimulated the growth of interest about this topic. These seismic events demonstrated the vulnerability of non-structural components to relatively low seismic intensity levels and showed that the damage or collapse of non-structural components could have major consequences in terms of economic and social losses, limit the functionality of most affected buildings and pose a significant hazard to human life (Fig. 4.1).

Nevertheless, the seismic performance of non-structural components and their effects on building behaviour

Fig. 4.1: Collapse of non-structural components in a residential building, 2009 Abruzzo Earthquake, L'Aquila, Italy

Fig. 4.2: Collapse of non-structural components in a storehouse, 2009 Abruzzo Earthquake, L'Aquila, Italy

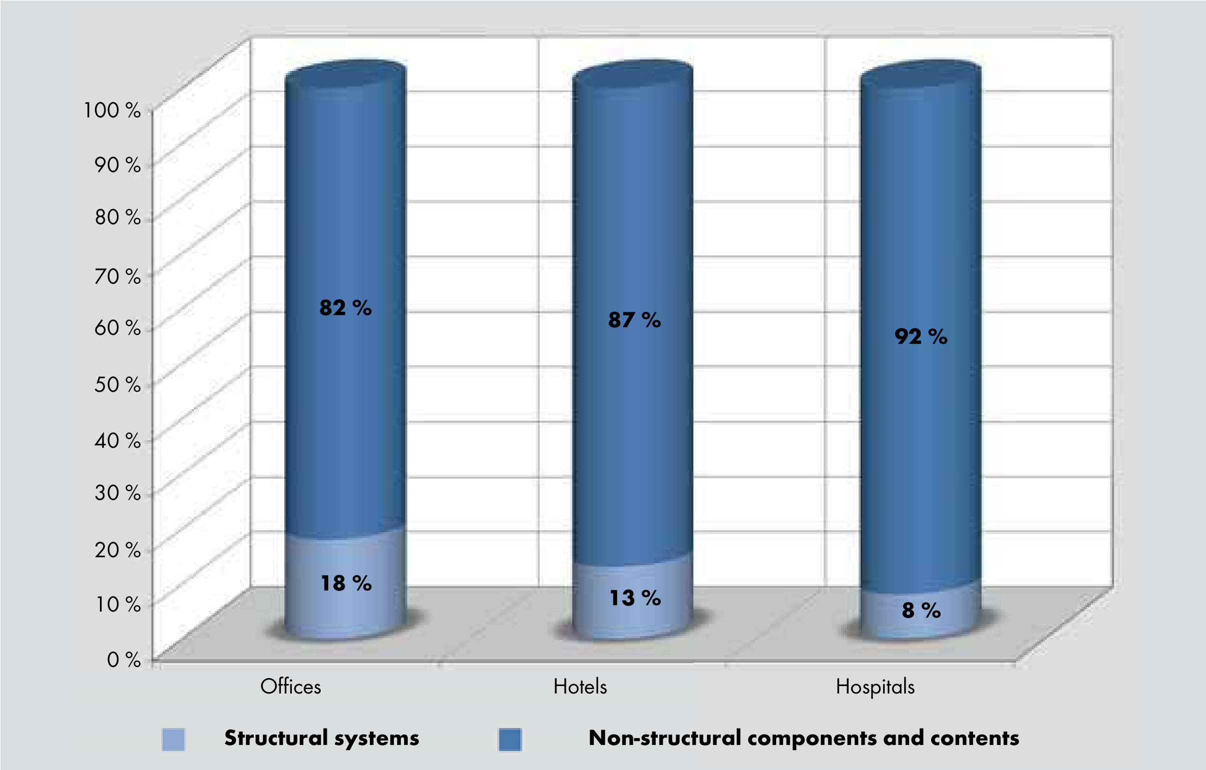


Fig. 4.3: Relative investments in commercial buildings /Whittaker and Soong (2003)/

are poorly understood, because specific guides and information about the relationship between non-structural damage consequences and structural response are very limited. Indeed, the issue of non-structural components and systems has received less attention than the design of primary structural systems, leading to a lack of specific design provisions for these components. However, there are several reasons that should encourage bridging these gaps.

Firstly, the primary objective of seismic design of non-structural components and systems is to reduce the risk of human life loss or injury to building occupants due to the damage or falling of non-structural components. For instance, the collapse of partitions and ceilings may cause hazards by the debris falling, impeding the safe exit from the facility and rescue operations during and after a seismic event (Fig. 4.2).

Furthermore, the non-structural components represent a high percentage of the total economic investment used in constuction of a building, and this aspect is particularly evident in complex civil constructions, in which the non-structural component cost is more relevant than the cost associated with other building types /4.1/. The economic incidence of the non-structural components on the total investment of commercial buildings, namely offices, hotels and hospitals, was investigated by previous studies. Fig. 4.3 shows, according to the study of Whittaker and Soong /4.2/, that the non-structural components, which are intended as architectural, mechanical and electrical components, building furnishings and contents, represent the largest percentage (from 82 to 92 %) of the original construction cost compared to the cost of structural systems (from 8 to 18 %). Moreover, the value of non-structural components, especially considering the equipment and contents, increases significantly in some building types (such as libraries, museums and high-tech laboratories), in which the non-structural property losses can be substantial and sometimes can even exceed the replacement cost of the building /4.3/.

Therefore, recent earthquakes highlighted that the most affected buildings, generally undamaged from the structural point of view, reported substantial non-structural damages and thus the temporary function loss /4.4/. In this way, the seismic design should go beyond the minimum code requirements for life safety, since the majority of building economic losses were due to both

Fig. 4.4: Damage to non-structural components in a public building, 2009 Abruzzo Earthquake, L'Aquila, Italy

direct and indirect repair costs of non-structural damages and both to the functionality interruption before resuming the ordinary activities. This critical issue is particularly relevant for essential facilities providing emergency and recovery services after a seismic event, such as fire and police stations, hospitals and emergency command centres (Fig. 4.4.).

Since collapse or damage of non-structural components could cause losses comparable to those of primary structural systems, the development of protection measures aimed to reduce the risks and to manage the vulnerabilities of the non-structural components and systems is becoming one of the most critical issues. For this purpose, from the outset of the design process it is necessary to involve good seismic design of both structural and non-structural components.

4.2 Definition and classification

Non-structural components are defined as those systems and elements housed in or attached to a building, which are not part of the main load-bearing structural system (Fig. 4.5) /4.3/. Although the non-structural components and systems are not required to participate in the building structural response and consequently not intended to resist the vertical and seismic loads, they are uniquely designed to support their own weight, which is transferred to the primary structural system of the building.

Nevertheless, some non-structural components and systems may interact with the structure and thus produce non-negligible effects on the building seismic response, such as in the case of rigid non-structural walls that could become part of the lateral load path (Fig. 4.6). Particularly, infill walls regularly distributed both in plan and elevation can significantly increase the lateral stiffness and strength of the structural system, while irregular infill walls can often negatively modify the seismic response by leading to undesired structural performance /4.5/. Therefore, a careful assessment of actual effects of non-structural components and systems on the building performance is essential to ensure proper design.

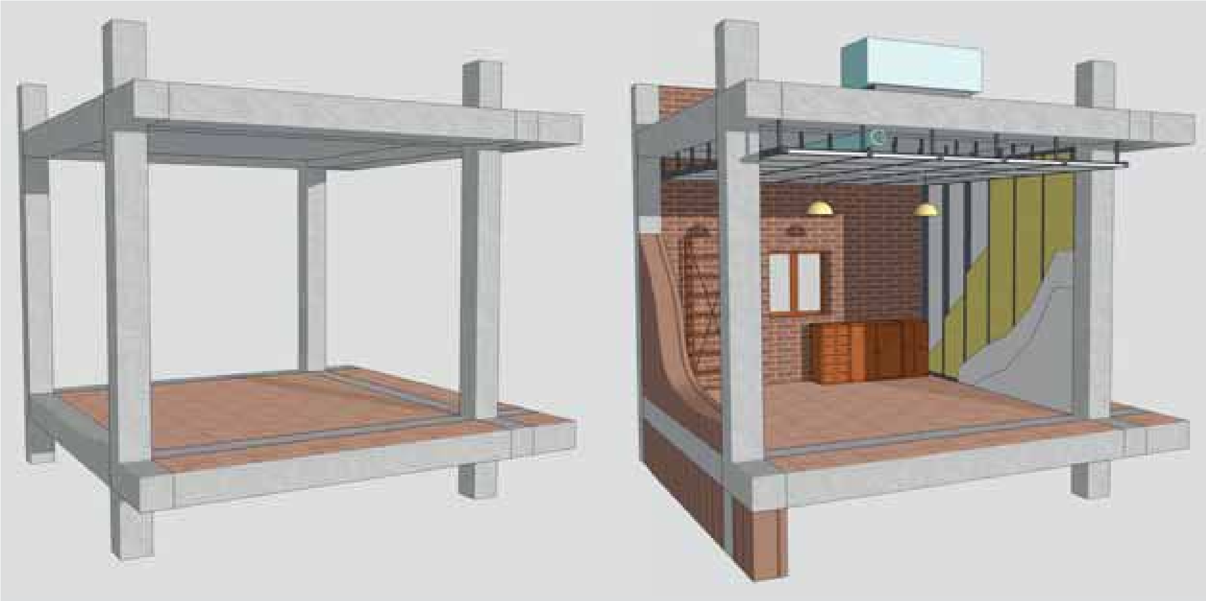


Fig. 4.5: Structural and non-structural components in a generic building

Fig. 4.6: Masonry infill walls in a reinforced concrete frame

The taxonomy of the non-structural components and systems is constantly evolving, since the growing demands of the construction sector and the innovative technological progress require increasingly complex systems. Nevertheless, in an attempt to reorder the study topic, several classifications are proposed in literature, and the most important are provided according to the typological functions of the non-structural components and systems in the building /4.3, 4.6/ or to their sensitivity to different response parameters of structure /4.7/.

A complete classification of the non-structural components and systems is provided in FEMA E-74 /4.6/, in which these elements are divided into three broad categories according to their typological functions (Fig. 4.7):

- Architectural components
- Mechanical and electrical components
- Building furnishings and contents

The first group includes interior partitions, suspended and attached ceilings, exterior curtain walls, pre-fabricated panels, cladding systems, cantilever systems (e.g. parapets and chimneys) and architectural ornamentations.

The mechanical and electrical components include boilers, pumps, piping systems, storage tanks, conduits and distribution systems, HVAC (i.e. heating, ventilation and air conditioning) equipment, elevators and escalators, transformers and lighting fixtures.

Some examples of building furnishings and contents are bookshelves, industrial storage racks, filing cabinets, industrial material furnishings, scaffolds in storehouses, and special equipment.

This classification includes permanent built-in parts, which may be considered as elements belonging to a building (e.g. the architectural components) and those required for the essential services (e.g. mechanical and electrical components). On the other hand, the building occupants in their ordinary use of space usually install building equipment and contents, which are considered as non-permanent items of the building. This latter category falls outside of the present discussion, and more indications are available in specific guides.

The seismic response of the non-structural components

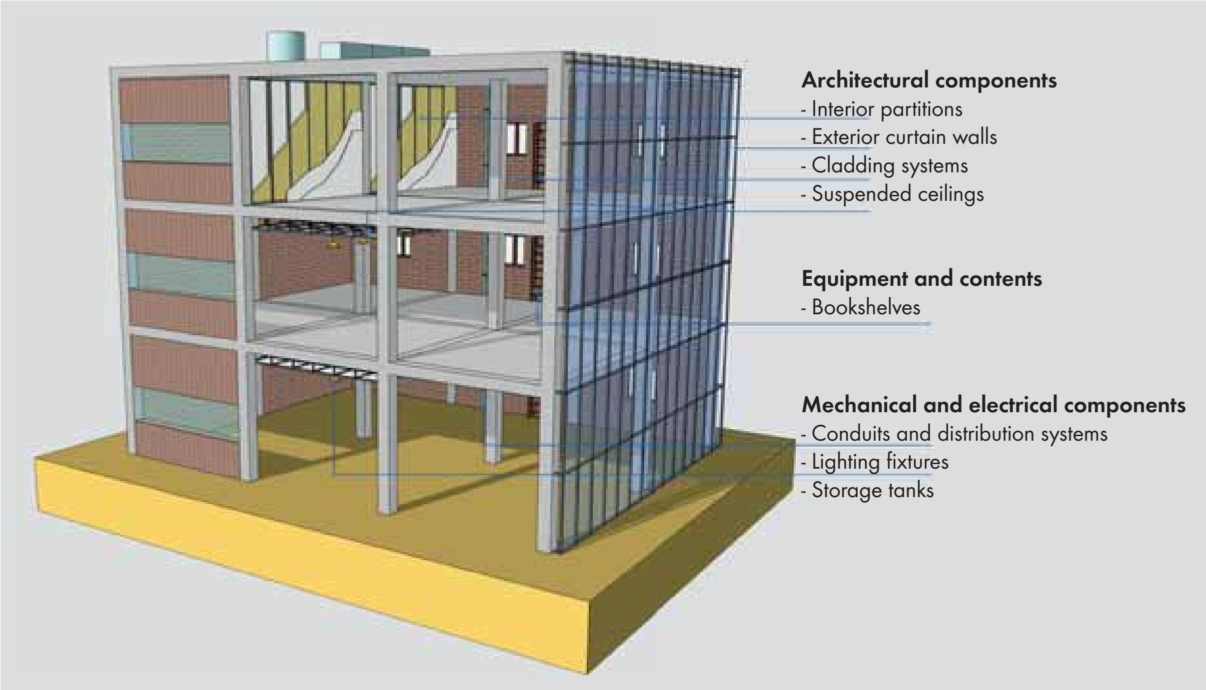


Fig. 4.7: Typological classification of the non-structural components and systems

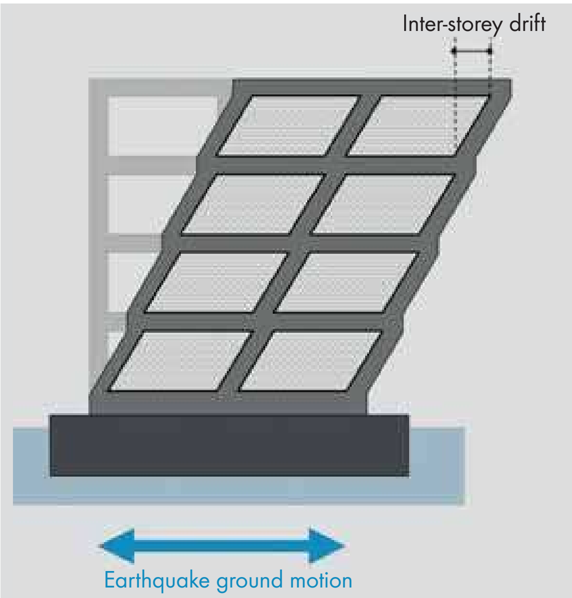


Fig. 4.8: Deformation-sensitive components: Definition of inter-storey drift

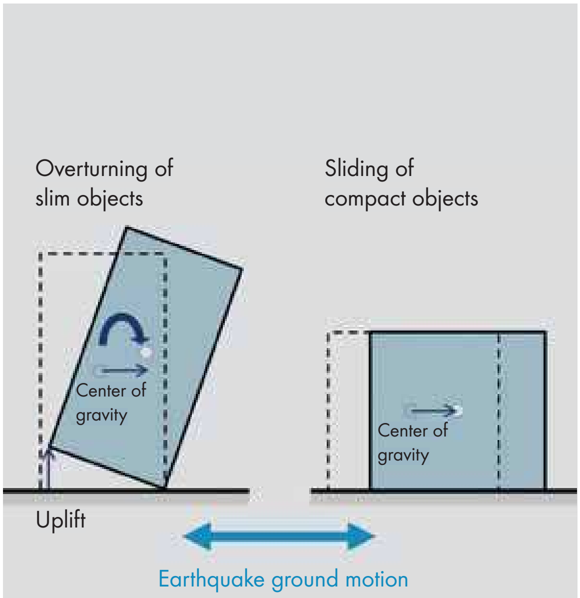


Fig. 4.9: Acceleration-sensitive components: Overturning of slim objects and sliding of compact objects

and systems is affected mainly by their sensitivity to several response parameters of the structure. According to ASCE/SEI 41-13 /4.7/, the non-structural components and systems are divided into three groups based on their seismic response:

- Deformation-sensitive components
- Acceleration-sensitive components
- Deformation- and acceleration-sensitive components

The deformation-sensitive components and systems are vulnerable and subject to damage due to deformation of the structure, which is imposed between the non-structural and structural elements through the attachment points undergoing differential movements. The deformation of structure is generally measured by inter-storey drift,

Tab. 4.1: *Response sensitivity of interior partition walls and ceilings /ASCE/SEI 41-13 (2013)/*

Architectural components	Acceleration-sensitive	Deformation-sensitive
Partitions		
Heavy	S	P
Light	S	P
Ceilings		
Directly applied to the building structure	P	
Suspended gypsum board ceilings	P	
Suspended lath and plaster ceilings	S	P
Suspended acoustic lay-in tile ceilings	S	P
P: Primary response; S: Secondary response		

Fig. 4.10: Interior heavy partition wall

Fig. 4.11: Interior full-height lightweight gypsum board partition wall /Knauf Insulation/

which is defined as the relative horizontal displacement between two adjacent floors (Fig. 4.8). Some examples of this category are curtain walls and interior cladding, which are rigidly connected to the structure, and piping systems usually running floor to floor.

The acceleration-sensitive components and systems are vulnerable and subject to damage due to inertial forces induced by the earthquake ground motion. Non-structural components having a large height or large mass may experience overturning or sliding (Fig. 4.9). Some examples are suspended elements, equipment anchored to the floor, parapets and appendages, chimneys and stairs.

The non-structural components and systems, which are sensitive to both deformation of the structure and inertial forces, are classified as deformation and acceleration-sensitive components. However, a primary mode of seismic response may be generally identified in these components, i.e., deformation sensibility or acceleration sensibility. For example, partition walls are defined as deformation- and acceleration-sensitive components, but they are primarily deformation-sensitive.

This chapter discusses the non-seismic and seismic design of the non-structural drywall systems by focusing the attention on the most common architectural non-structural components that are interior partition walls and ceilings. Tab. 4.1 identifies the response sensitivity of these elements /4.7/.

According to ASCE/SEI 41-13, the interior partition walls are defined as vertical non-load-bearing interior components that provide space division /4.7/. Depending on the component weight, the interior partitions may be classified as:

- Heavy partition walls

a) Integrated ceiling systems/Knauf Italia/ b) *Example of a typical assembly /Knauf Danoline/*

Fig. 4.12: Suspended acoustic lay-in tile ceilings

a) Example of a typical assembly /Knauf Italia/ b) Short-dropped furred gypsum board ceiling

Fig. 4.13: Ceilings directly applied to the building structure

- Light partition walls

Heavy partition walls may be usually made of reinforced or unreinforced masonry (Fig. 4.10), and they may be full-height or partial-height. Although they are defined as non-structural components, an engineering assessment is recommended in some cases, since they may significantly affect the overall seismic response of the building and participate in the lateral force resisting systems. Heavy partitions generally weight more than 0.50 kN/m^2 /4.8/. Light partition walls may usually be made of wood or lightweight framing covered with several cladding materials. The lightweight partition walls consist of a steel wall framing realized with stud members, having lipped channel sections (C-sections), usually spaced at 300, 312.5, 600 or 625 mm on the centre and connected at the ends to track members, having unlipped channel sections (U-sections). The wall framing is generally completed with lath and plaster finish or with several cladding materials, i.e. gypsum boards, wood or other materials. Even in this case, light partition walls may be full-height, which extend from floor-to-floor (Fig. 4.11), or partial-height that are stopped at the height of ceilings. Generally, the weight of light partitions is less than approximately 0.50 kN/m^2 /4.8/.

Ceilings are defined as horizontal and sloping assemblies attached to or suspended from the structure /4.8/. According to FEMA E-74, they are usually categorized in three wide-ranging categories depending on the type of attachment to the building structure:

- Suspended acoustic lay-in tile ceilings
- Ceilings directly applied to the building structure
- Suspended heavy ceilings

Suspended acoustic lay-in tile ceilings (Fig. 4.12a) are usually realized with supporting lightweight steel grids,

a) Hilton Squaire, Frankfurt am Main, Germany, JOI-Design GmbH Architect, 2011

b) Palazzo Mantegazza, Lugano, Switzerland, G. Camponovo and associates, 2008

Fig. 4.14: Suspended lightweight steel gypsum board ceilings

namely T-bar frames, that may be exposed or hidden spline systems. Generally, the suspended T-bar frames are completed with closure panels placed in the metal grid. These ceiling types are considered as integrated ceiling systems, since they are generally embedded with lighting fixtures and other mechanical elements, such as air diffuser and sprinkler heads (Fig. 4.12b).

Ceilings directly applied to the building structure (Fig. 4.13a) are generally built with surface materials, i.e. gypsum boards, wood or metal panels, laths and plaster, attached by means of adhesives or mechanical fasteners to concrete slabs or decking, structural beams, wood or metal joists placed for supporting floors. Furthermore, this last category includes ceilings with height less than 0.60 m from the building structure (Fig. 4.13b), also called short-dropped furred gypsum board ceilings, which are realized using gypsum boards attached directly to wood or metal furring strips or similar connected to the bearing members /4.7/.

The category of suspended heavy ceilings, as well as indicated by FEMA E-74, includes two ceiling types:

- Dropped furred gypsum board ceilings (i.e. suspended lightweight gypsum board ceilings)
- Suspended laths and plaster ceilings

The most common are the suspended gypsum board ceilings that generally have a height greater than 0.60 m from the building structure. They are usually realized with gypsum boards or other finish materials, i.e. metal or wood panels, attached to a lightweight steel two-way furring grid hanging from the structure by wires or other means (Fig. 4.14a, b). The other type of suspended heavy ceiling is the suspended laths and plaster ceiling with height greater than 0.60 m from the building structure.

Among these systems, lightweight drywall constructions using cold-formed steel members represent a valid alternative to traditional constructive systems for non-structural applications in seismic areas. In fact, these systems guarantee a good seismic behaviour, mainly thanks to their lightness and low stiffness. Furthermore, the use of these systems in different technical application fields has increased significantly over recent years, and consequently, they represent a large economic investment in the construction sector. For these reasons, the current discussion deals with the non-seismic and seismic deisgn issues and procedures for non-structural drywall systems, i.e. lightweight gypsum board partitions, suspended acoustic lay-in tile ceilings and suspended lightweight gypsum board ceilings.

4.3 General design issues and procedures

4.3.1 Assessment basis and implementation for partition walls

Non-load bearing internal partitions as drywalling constructions are characterized by their low self-weight, while at the same time featuring a slim design. On one side, their low load on the primary support structure represents an important contribution of the walls with regard to the structural design of buildings. In fact, in many cases, it is simply sufficient to register the weight of all lightweight partitions per floor slab using a fixed and comparatively small live load factor. Furthermore, the walls must be capable of withstanding static and impact-type effects and influences.

At first glance, this may appear to be absurd, as the attribute "non-load bearing" allows us to assume that the building element is in a permanent load-free condition. By definition, the counter term "load-bearing" can be understood to mean the acceptance and transfer of planned building loads, such as snow loads, live loads, etc. In real building usage situations, further unplanned loads such as those caused by persons and furnishings may occur. For this reason, non-load bearing walls must be designed to carry such additional and special load conditions, since they are intended as integrated components of a service unit of the building.

In this regard, for instance, European guidelines for the provision of applicability verification, e.g. the ETAG 003 guideline /4.9/, regulate the requirements and/or verification procedures for non-load bearing constructions. These incorporate a series of loads and stresses that occur in practical use for which the structural stability must be certified. Mainly these are loads induced by usage (e.g. cabinets and other cantilever loads) (Fig. 4.15), persons (e.g. impact, compression pressure and partition walls acting as barriers) or other special installation cases (e.g. wind loads with open façades).

In particular, metal stud partitions should be capable of withstanding these loads. Additionally, the constructional design of the wall, the design of the connections to transfer the supporting loads and the wall heights are other decisive parameters on their load limitations.

The application areas of drywall constructions continue to expand steadily, thanks to the diverse range of design possibilities and the good building physical properties. The growing demand of these systems has been made possible through continuous research projects, motivated by the increased requests of investors and contractors for larger wall heights (such as occurs for cinema complexes, museums, concert halls and other special buildings) and for more load capacities (e.g. with pressure surges in quenching gas systems, over-pressures and under-pressures in laboratories and other hygienic areas, swivelling arms for mounting medical and technical equipment).

For these reasons, the relevance of structural verification procedures for non-load bearing internal partitions is increasing.

Before the seismic design of these constructional components is analysed in greater depth, their fundamental structural requirements must be characterized.

Definition of partition wall loads

The basis for determining the loads and thus the minimum requirements are defined, for instance, by EN 1991-1.1. The code defines the respective categories for horizontal loads on balustrades and partition walls acting as barriers (Tab. 4.2). By assuming a horizontal line load in balustrade height at 1.2 m above the ground (or 0.9 m in dependence on the national regulations), the resistance of the wall to the compression pressure of a single person or even a whole group of people is simulated (Fig. 4.16a). At this load, there may not be a loss of the structural integrity whether it is with regard to a local or global failure state. The corresponding ultimate bending load-bearing capacity of the walls may not be exceeded by the imposed load value when observing a national safety level.

Furthermore, EN 1991 also governs the internal pressure acting on the walls as a load per unit area as well as

Fig. 4.15: Cabinet with asymmetric centre of load gravity to the surface of the drywall partition /Cambodunum Film GmbH/

the exterior wind loads that are to be considered (Fig. 4.16b). This can be the case with the façades open (open share of the façade per side of the building up to 30 % of the total surface). Special pressure factors are defined in the standard for determining the magnitude of the interior pressure. Depending on the wind zone and the building height, the interior pressure can be of very different magnitudes. Loads common in practice are in the range of 0.20 to 0.30 kN/m^2. In high rise buildings as well as in windy coastal areas and islands, this value may be significantly higher.

Unconsidered by EN 1991, the installation loads that are anchored to the partitions remain. However, they are a very important consideration when rating the partition, as these loads hanging on the wall, designated as cantilever loads, place a permanent load on the wall and thus have a significantly longer load duration than the above mentioned line load (special case) and wind load (brief duration). Corresponding requirements are defined in the ETAG 003. Hereby, an asymmetric centre of mass between the wall surface and centre of mass of the object hanging on the wall of 0.3 m is generally assumed (Fig. 4.17). The service loads as dependent on the intended purpose are between 0.4 kN/m (e.g., DIN 4103-1), 0.7 kN/m (e.g., DIN 18183-1) and 1.0 kN/m /4.9/. Higher loads such as 1.5 kN/m (e.g., DIN 18183-1) or 2.0 kN/m /4.9/ generally require additional constructional measures (sanistands, traverses, etc.).

Verification procedure for structural stability of partition walls

The structural stability on a non-load bearing partition can be verified by calculation as well as by technical testing. A fundamental prerequisite for the calculated proof is that the components used are each specified by a corresponding product or design standard. In case of metal stud partitions, the gypsum boards used can be rated with the defined characteristic strengths. For the lightweight steel profiles of the wall substructure, e.g., EN 1993-1-3 is a suitable basis. However, the lightweight steel partition systems generally only develop their actual performance capabilities by the mechanically effective bond between boards and profiles. A limitation to the individual design of the components without the application of the bonding effect, thus results in a significant underestimation of the stability. Moreover, as a consideration of the bonding effect produced between gypsum boards and steel profiles is not considered from a calculation point of view, the technical test proofs are a practice-orientated alternative. In component tests, the real performance of the constructions is determined. For this purpose, static loads are generally applied using a hydraulic cylinder and using suitable load distribution devices (e.g. beams for generating the linear loads and cantilevers for generating the cantilever loads) (Fig. 4.19). In cases of impact-type loading, the corresponding impact bodies (e.g. sack filled with glass marbles for simulating soft impacts and steel balls for hard impacts) are used to simulate an impact with a human body or with hard objects (Fig. 4.18). The load of the hydraulic cylinder and the drop height of the impact body are increased progressively in the test procedure until the structural limits of the wall are reached. The bending load-bearing capacity that applies is defined differently in the standards. For example, in the DIN 4103-1, failure is defined as "that condition in which an increase in the load is no longer possible or in which parts of the partition are destroyed to such an extent that the original structure of the wall ceases to exist. This also applies for cases where cladding becomes detached over a wide area from the rest of the wall structure." On the other hand, the ETAG 003 defines the bending load-bearing capacity through

Tab. 4.2: *Horizontal loads on parapets and partition walls according to Eurocode 1 /EN 1991-1-1 (1991)/*

Categories	**Line load q_k**	**Example**
Category A	0.2 to 1.0 kN/m	0.5 kN/m
Category B and C1	0.2 to 1.0 kN/m	0.5 kN/m or 1.0 kN/m
Category C2 – C4 and D	0.8 to 1.0 kN/m	1.0 kN/m
Cetegory C5	3.0 to 5.0 kN/m	3.0 kN/m
Category E	0.8 to 2.0 kN/m	2.0 kN/m

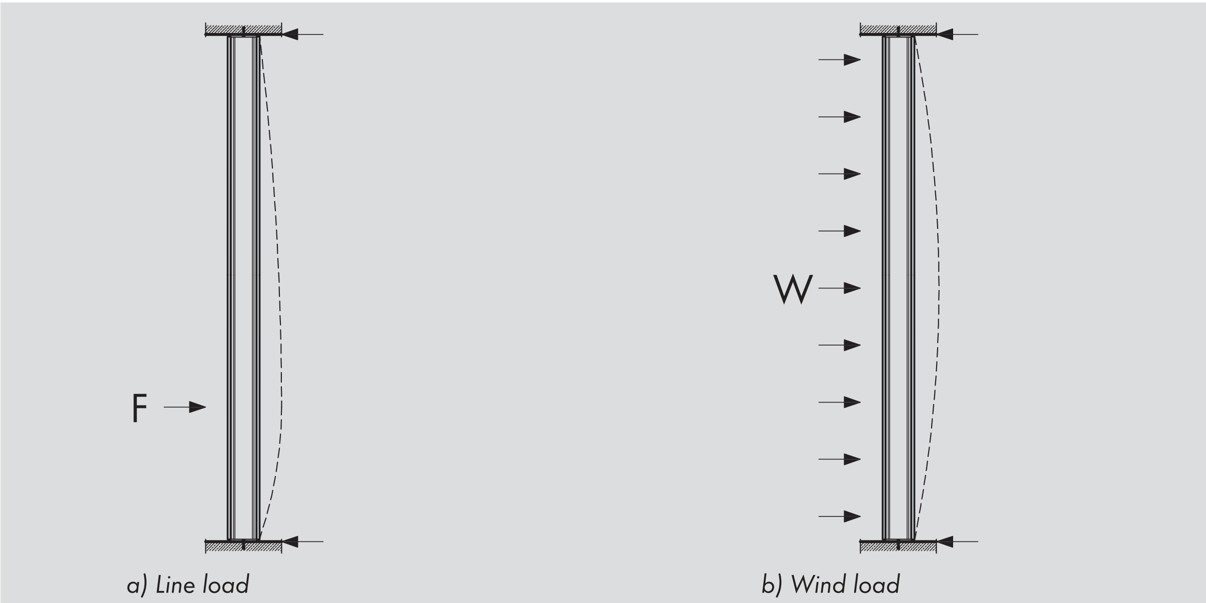


Fig. 4.16: *Definition of partition wall loads*

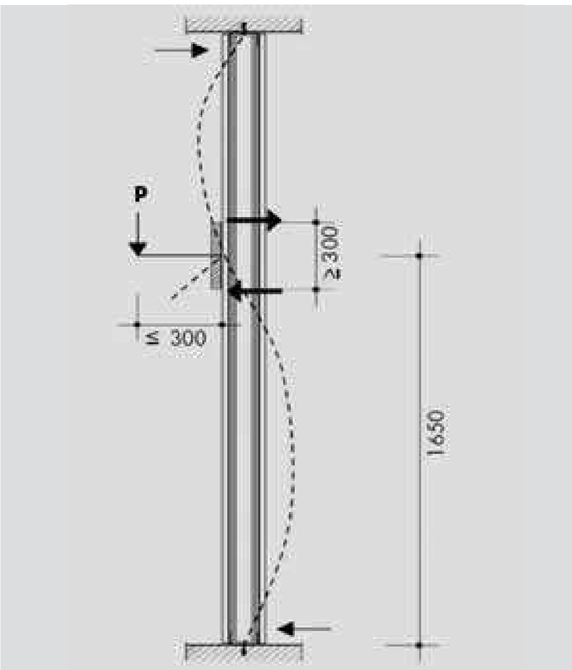


Fig. 4.17: *Installation loads anchored to partition walls designed as asymmetric cantilever loads*

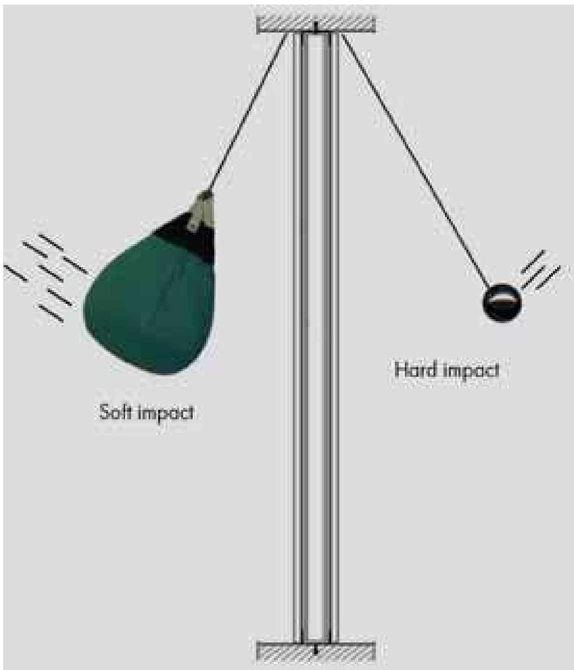


Fig. 4.18: *Hard and soft impact test for applying impact-type loading to partition walls*

Fig. 4.19: Line load test used for applying static loads to partition walls

Tab. 4.3: Deformation limits relative to the partition height (DIN 18183-1).

Requirement level	**Partition deformation limits**
Minimum requirement	$h/350 < f \leq h/200$
Average requirement	$h/500 < f \leq h/350$
Enhanced requirement	$f \leq h/500$
h: Partition height; f: Maximum deflection	

Tab. 4.4: Absolute deflection limits for partition walls (BS 5234-2).

Duty class	**Max. deflection**	**Max. residual deformation**
Light duty	25 mm	5 mm
Medium duty	20 mm	3 mm
Heavy duty	15 mm	2 mm
Severe duty	10 mm	1 mm

building damage criteria (e.g. for a large scale soft body impact test, they are “No penetration”, “No collapse” or “No other dangerous failure”).

Serviceability requirements for partition walls

In addition to the stability, the serviceability plays a decisive role for the application of the walls in a real installation and usage context. This means that the function or usage may not be negatively impacted when subjected to the service loads. For example, this situation can occur for large deformations during the imposed loads, remaining deformation after the effect of the imposed loads, cracks, negative consequences on the building physical properties (e.g. fire protection, sound insulation) and vibration responses.

With respect to component deformation, the deformation classes are generally defined on a national basis and compliance to them is generally required to ensure certain intended purposes. A partition wall without any particular demands may be categorised to a class with large permissible deformation limits, whereas a partition wall with a covering that is sensitive to deformation (e.g. larger

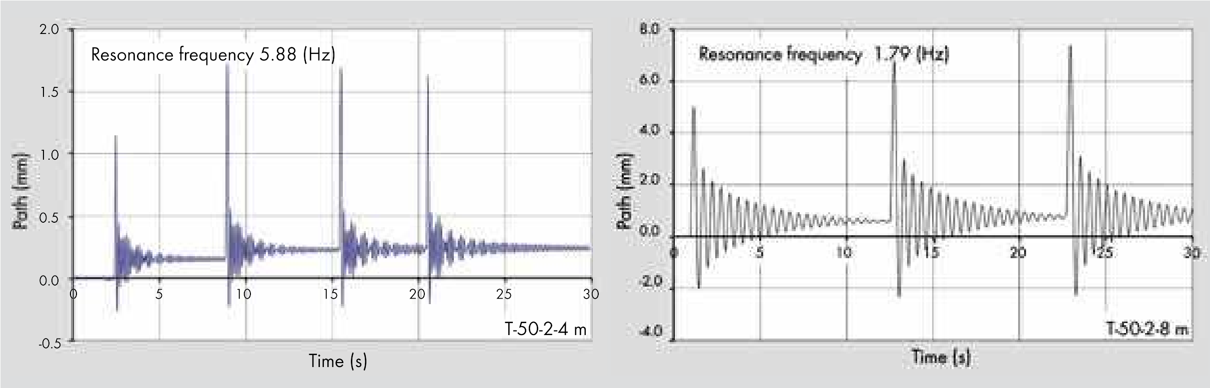


Fig. 4.20: Resonance frequency of partitions with 4 m and 8 m heights (vibrational response caused by impact) /Materialprüfanstalt für das Bauwesen, Braunschweig/

natural stone tiles) may be subject to a more stringent class with lower permissible deformation limits. Examples of deformation limits relative to the partition height can be found in the DIN 18183-1 (Tab. 4.3), while absolute deflection limits are defined in the BS 5234-2 (Tab. 4.4).

Furthermore, the resonance frequency of the component can provide an important contribution with regard to its intended purpose. Partitions with a resonance frequency that can be easily excited by persons (e.g. vibration, slamming doors, etc.) are considered to be "soft" and untrustworthy. In extreme cases, cracks can form on deformation-sensitive wall surfaces due to oscillation of the wall, because of excitation at its own resonance frequency. It is thus important to ensure that the partitions feature a sufficiently high resonance frequency. The following diagrams indicate the influence of doubling the partition height on the resonance frequency of 4 and 8 m height lightweight steel partitions with the double cladding layer and steel stud members, having C-shaped cross sections and web height equal to 50 mm. At a height of 4 m, the partition has a resonance frequency of about 6 Hz. An installation height of 8 m, with the same constructional design, reduces the resonance frequency to around 2 Hz, which can be energized easily by persons (Fig. 4.20).

4.3.2 Assessment basis and implementation for ceiling systems

Non-load bearing suspended drywall ceilings constructions are distinguished by their additive character. In contrast to the non-load bearing internal partitions, they do not solely provide room separating functions. They are simply a supplement to a storey ceiling or to a roof. They also differ in terms of their horizontal design and their non-direct accessibility for persons. They are thus subject to completely different loads than those for partitions and walls. Loads induced by persons such as the soft impact are thus irrelevant.

The fundamental construction principle of ceilings consist of the combination of several different basic components, employing a friction (partly interlocking) bond between a two-axis horizontal load distributing structure and a vertical load distributing structure. Many variants of designs, configurations and spacings are possible with these components. The ceiling systems created in this manner can greatly differ in terms of their structural characteristics, in terms of load-bearing and deformation performance, and they should be designed accordingly to suit their intended purpose.

Definition of ceiling loads

The loading of ceilings .consists primarily of their self-weight. Here, the lightweight steel grids, suspenders and connectors generally contribute a comparatively small portion of the overall load. This is generally determined by the type, thickness and number of cladding layers. For example, a sound insulating ceiling with two layers of heavy sound shield can quickly add up to a multiple of the weight of an acoustic ceiling with a single layer of perforated boards, that are relatively light. In addition to the self-weight of the ceiling components themselves, further vertically acting supplementary loads can be caused by insulation materials, lighting fixtures and similar

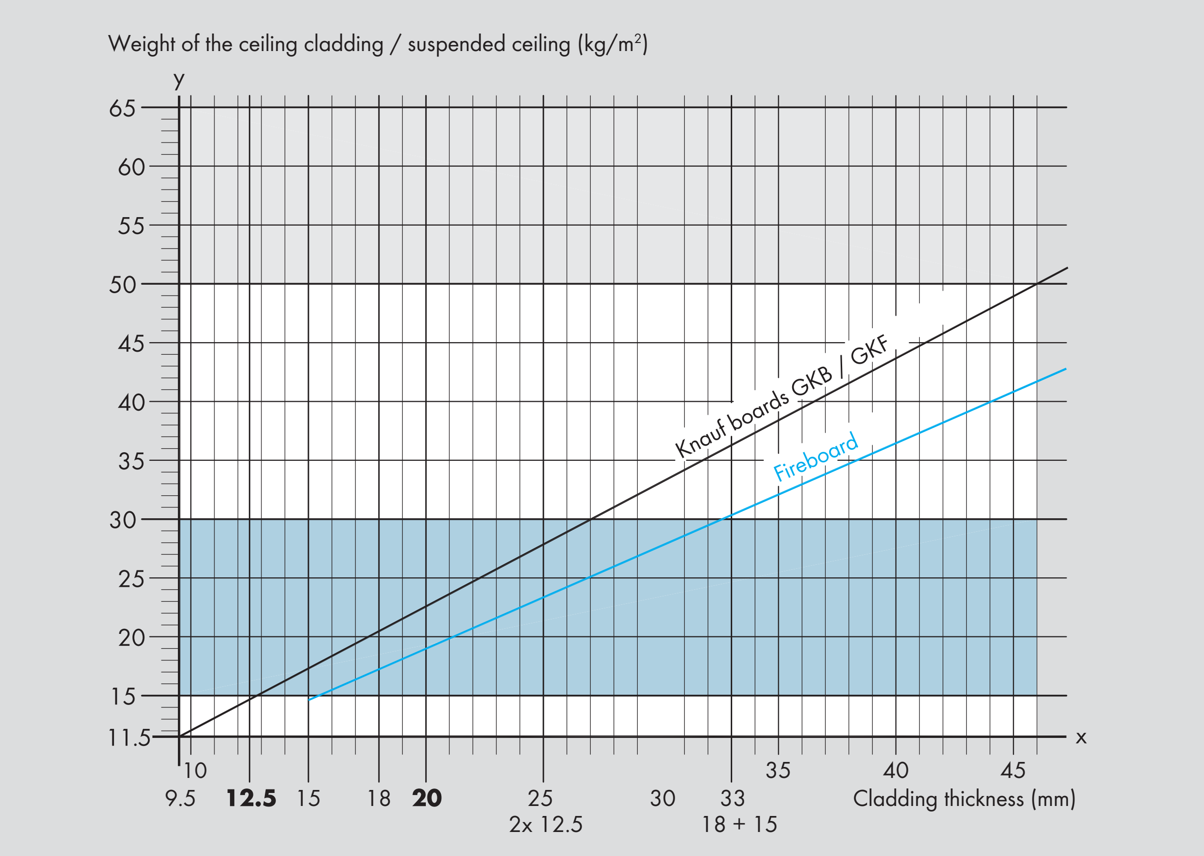


Fig. 4.21: Self-weight of suspended ceiling in dependence on the board type/thickness

components. All together, the sum of the self-weight and the additional loads form the basis for the evaluation of the quasi-permanent loads and thus the basis for the load case for the dimensional design of every ceiling.

As there is such a large range of boards, insulation materials and built-ins that are not fully encompassed in standards for loads, such as in the EN 1991, it is generally the case that the weight specifications from the manufacturer of these products are to be used for dimensioning (Fig. 4.21).

In addition to the quasi-permanent loads, in certain areas of application, changeable loads can also occur. Generally, these area loads are the result of over-pressure or under-pressure scenarios, as they can occur due to internal wind pressures (e.g. open window), ventilation systems, rooms subject to over-pressure (e.g. clean rooms) or quenching gas systems. A special area of application is the suspended ceilings in exterior areas not directly exposed to weather effects, such as those in access balconies, canopies and similar. They are exposed to the full wind load, as it effects the respective location on the building and the regular façade building components. Determination of this wind load is thus generally according to load defining standards such as EN 1991. In the other load cases mentioned beforehand, an exact examination and assessment of the installation situation is required. In this case, for example, the design expert for the ventilation or quenching gas system should provide relevant information on the expected pressures.

Verification procedure for the structural stability of ceiling systems

In the verification procedure for structural stability, the fundamentals applicable for the ceilings differ from those of the partitions, as generally a structural building component test is not performed for the ceilings. The proof is generally from the respective individual components of the suspended ceiling system, such as suspenders, connectors, carrying channels or laths, furring channels or laths, cladding (e.g. gypsum boards).

In case of channels or laths, the proof can be undertaken by calculation should this be covered via the corresponding product and design standard. On the other hand, thin metal profiles as well as suspenders and connectors, which are generally made of thin sheet steel, require structural stability classification via engineering testing. Two standards are available at European level for this purpose with EN 14195 and EN 13964, which govern the test procedure and the assessment of the results. The type of loading in the test is dependent on the intended application and the resulting load of the components. Conventional tests on ceiling components are tensile tests on suspenders and connectors (Fig. 4.22), compression strength tests on suspenders and bending test on profiles. Based on the average value of the maximum loads of a test series and the corresponding standard deviation (scatter of results), the 5 percentile value is defined as the characteristic strength of the respective components. The safety factor required for the definition of a permissible load, which is applied to the characteristic value, is applied differently according to the national applicable standards. For non-load bearing suspended ceilings, safety factors of 2.5 and 3 are generally used.

Different design variants of the ceiling components can lead to different permissible loads. Thus, for example, a suspender, whose suspension height is increased, may as a result withstand less of a compressive load before it buckles. Similar applies for bending of stressed channels and their span widths.

If all performance parameters for the individual components for a suspended ceiling are available, it can be dimensioned as a complete system. For this purpose, as is generally the case in structural design, the load flow in the construction is considered. In contrast to the proof with common supporting structures, this is not "from above to below", but rather "from below to above". The starting point is the lower-side lining and its self-weight and additional loads at the maximum span width between its support points, e.g. the furring channels, dimensioned to carry the load. The furring channels are also subject to a linear supporting force from the cladding, and they transfer this force via the connectors to the carrying channel that is connected to the basic ceiling via suspenders. The load flow ends with the transfer of the loads from the suspenders to the raw ceiling via dowels or via screw fasteners.

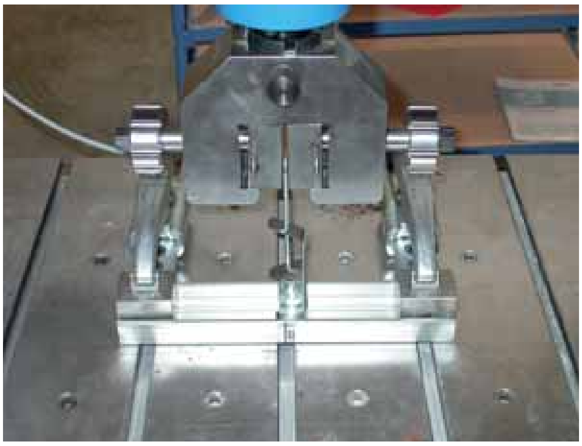


Fig. 4.22: Tensile test on a suspender in conjunction with a carrying channel section

The design consists ultimately in the comparison of a characteristic load of a ceiling component, determined by the load flow, with its permissible load, determined in a test. In the course of dimensioning the ceiling, in addition to consideration of different loads, different design variations of the ceiling system, particularly with regard to the profiles and suspender axial spacings are possible. The following table indicates which combinations of carrying channel and suspender spacings, are possible for a ceiling with a total weight of 0.15 or 0.50 kN/m² (Tab. 4.5).

Suspended ceiling systems receive their high level of application safety from their pronounced redundancy resulting from the high number of suspension points in conjunction with a high level of structural indetermination of the profiles (multi-span supports). Should a suspender fail locally, its load can be distributed via the profiles of several adjacent suspenders. A proportionate increase of the load of these adjacent suspenders (even exceeding the service load level) is possible without problems due to the large safety factor in comparison to the actual failure load.

Serviceability requirements for ceiling systems

For cladding layers and channels, in addition to the load-bearing capacity limits, the flexural stability must also be determined in the engineering tests. The expected deflection can be calculated from the span width, load and flexural stability. The EN 13964 provides three deflection classes (Tab. 4.6).

Tab. 4.5: *Axial spacing combinations of a ceiling with double profile frame*

Axial spacings of carrying channel	**Suspender spacings**		
	Load class kN/m²		
	up to 0.15	**up to 0.30**	**up to 0.50** [1)]
500	1200	950	800
600	1150	900	750
700	1100	850	700 [2)]
800	1050	800	700 [2)]
900	1000	800	-
1000	950	750	-
1100	900	750 [2)]	-
1200	900	-	-
1) Only use suspenders of load bearing capacity class 0.40 kN 2) Does not apply for carrying channel axial spacing of 800 mm			

Tab. 4.6: *Deflection classes for cladding layers and profiles acc. to EN 13964*

Class	**Deflection mm**
1	l/500 and not greater than 4.0
2	l/300
3	unlimited
l is the span width between suspension points	

It has to be selected acc. to the required level for serviceability, depending on the national regulations and on the application specific requirements of the selected ceiling system. The suspended ceiling with gypsum boards as a cladding layer and the l/500 (and ≤ 4 mm) criterion has become established in many countries. Not only does it ensure that a high level of surface evenness is provided (important, for example, with side light), it assures that in the event of greater deflection secondary effects such as formation of cracks in board joint areas do not occur.
Particular attention in this regard is to be directed to the deformation behaviour of gypsum boards. Where metal profiles exhibit practically an ideal elastic behaviour under a service load, gypsum boards may be subject to additional elastic deformation and even to time-dependant deformation effects (sagging). Thus, a test which increases the load continuously until a rupture occurs and is the sole condition for a deformation assessment of the boards is not sufficient. They must also be tested in a long-term test with permanent application of the service load and under defined climatic conditions. Overall, the total deformation combined of elastic and time-dependent deformation factors must satisfy the requirements in serviceability.

4.4 Seismic design issues and procedures

4.4.1 Seismic behaviour: Damage causes, typical damages and consequences

The non-structural components and systems have some physical characteristics, which define their seismic vulnerability, dynamic response and damage degree, identifying a unique seismic behaviour that is highly dissimilar from that of structures.
In fact, several factors contribute to their seismic behaviour that depends mainly on the characteristics of the earthquake ground motion, dynamic characteristics

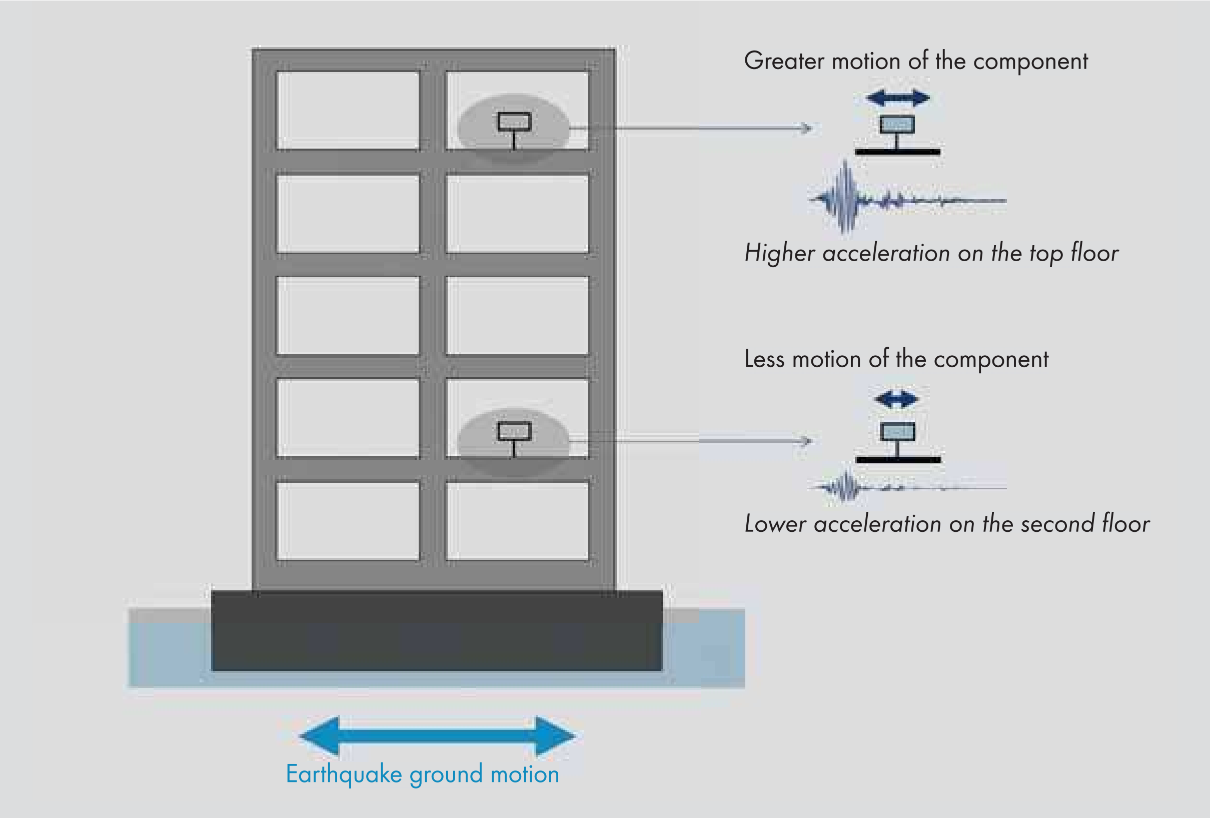


Fig. 4.23: Seismic response of non-structural components and systems based on their location in the building structure

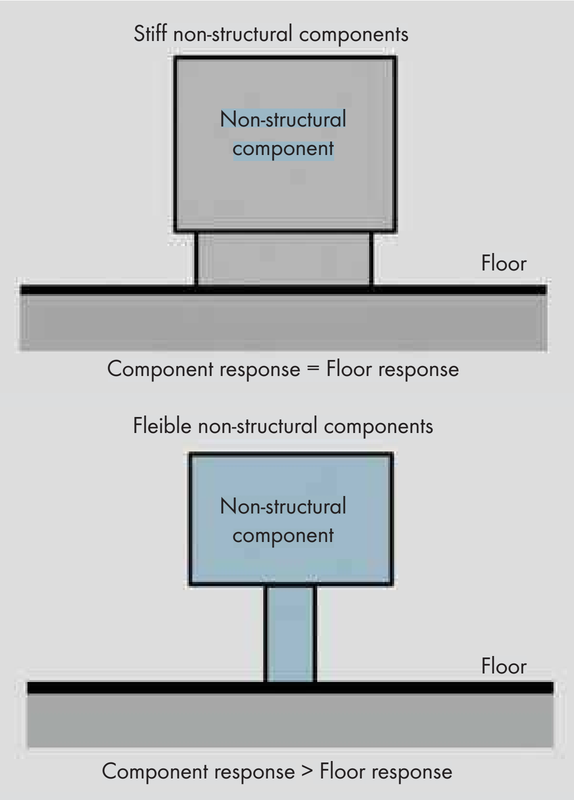


Fig. 4.24: Seismic response of non-structural components and systems based on the component's dynamic characteristics

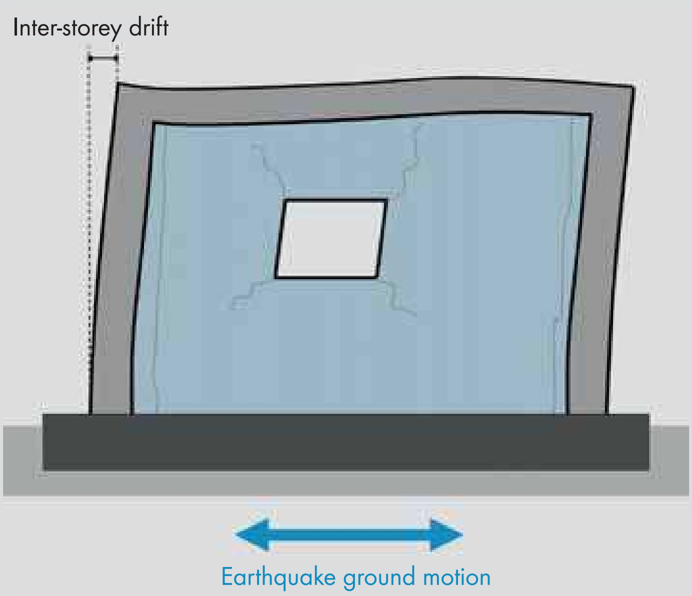
of the building structure, location of the non-structural components within the building structure and attachment type to the building structure, i.e. anchorage or bracing. In particular, taking into account that the accelerations and the consequent inertial forces induced by the earthquake ground motion are generally amplified along the building height from the foundation to the top, the non-structural components located on the upper floors are subjected to higher accelerations than those at the building base (Fig. 4.23).

Furthermore, other relevant factors defining the non-structural seismic response are the component weight, the interaction with other structural or non-structural components and the dynamic characteristics of the non-structural components /4.6/. Regarding this latter aspect, some non-structural components and systems are very flexible compared to relatively rigid structures and, in this case, they exhibit levels of seismic excitation much higher in comparison to those of the structure. On the other hand, very stiff non-structural components and systems show seismic excitations similar to the supporting structure (Fig. 4.24).

a) Overturning of shelvings in a cheese storehouse, 2012 Emilia Earthquake, Mantova, Italy /Marco Savoia/

b) Sliding of a compressor without seismic restraints, 1994 Northridge Earthquake, Los Angeles, USA /Wiss, Janney, Elstner Associates, Inc/

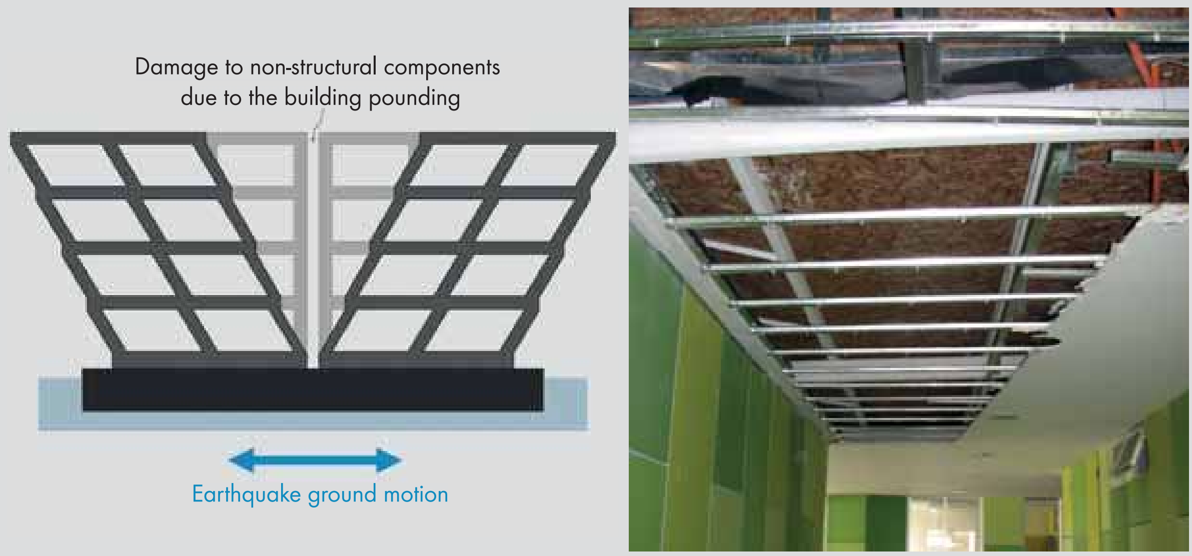
Fig. 4.25: Direct effects of the inertial forces on non-structural components and systems



a) An example of non-structural damage

b) Damage at joints between steel structure and a lightweight gypsum board partition, Knauf De Chile Ltda Offices, 2010 Chile Earthquake /Knauf Chile/

Fig. 4.26: Effects of the building deformation on non-structural components and systems



a) An example of non-structural damage

b) Damage to a suspended lightweight steel gypsum board ceiling due to the pounding at the interface between two service blocks, Terraustral Del Sol High School, 2010 Chile Earthquake /Knauf Chile/

Fig. 4.27: Effects of the building pounding on non-structural components and systems

A basic reference for the non-structural damage is FEMA E-74 that explains the sources of the seismic damage, the effects and the methods for reducing the potential risks. This document identifies four main causes of non-structural seismic damage due to the earthquake ground motion.

Firstly, the inertial forces, experienced during an earthquake by a building and non-structural components contained therein, may cause the overturning of slim objects (Fig. 4.25a) or the sliding of compact objects (Fig. 4.25b). As mentioned above, the components affected by this type of damage are defined as acceleration-sensitive components and systems (see section 4.2). A good performance of the acceleration-sensitive components and systems could be ensured if the anchorages or bracings between non-structural elements and supporting structure were detailed to prevent the movement under design loadings, without accidentally interfering with the behaviour of the structural system.

Furthermore, the building structure may cause damage to the interconnected non-structural components and systems during an earthquake. This damage type is due to the building deformations, and it depends on the attachment type in multiple points between structural and non-structural components and on the characteristics of the supporting stucture. For instance, a building structure

Fig. 4.28: Effects of the interaction between non-structural compnents and systems: Collapse of a suspended lightweight gypsum board ceilings due to the presence of a heavy HVAC ductwork in a commercial building, 2010 Chile Earthquake /Knauf Chile/

subjected to significant deformations (Fig. 4.26a) could cause relevant damages or breakages particularly in brittle materials, e.g. windows, partitions (Fig. 4.26b), claddings, glass or masonry infilled walls. As mentioned above, the non-structural components highly susceptible to the damage due to the building deformations are defined as deformation-sensitive components and systems (see section 4.2). The effects of the building deformation on the deformation-sensitive components and systems

a) Damage to a full-height lightweight steel partition due to the interaction with a reinforced concrete structure, Santa Maria Bianca Hospital, 2012 Emilia Earthquake, Mirandola, Italy /Angelo Masi/

b) Damage at the opening corners of a full-height lightweight steel partition, Santa Maria Bianca Hospital, 2012 Emilia Earthquake, Mirandola, Italy /Angelo Masi/

Fig. 4.29: In-plane damage to lightweight steel gypsum board partitions due to the building deformations

may be reduced by designing them to accommodate the expected lateral displacements without damage or by limiting the inter-storey drift of the structural system.

As highlighted in FEMA E-74, the pounding at the interface between adjacent structures or structurally independent portions of a building may be considered as another damage source to non-structural components and systems (Fig. 4.27a). The pounding generally occurs in correspondence with separation joints that are actual distances or gaps between two different structures. The separation joints, if properly designed, should decouple the vertical movements between the structures by avoiding transfer of impacts and allowing independent movements of each portion. In these cases, in order to ensure the functional continuity of the building, the non-structural components and systems (e.g. piping systems, conduits, partitions and ceilings) frequently cross the separation joints and extend into the other portion of the structure. For this reason, they should be designed to accommodate the expected horizontal movement at the separation joints to avoid this non-structural damage type in case of seismic events (Fig. 4.27b).

Finally, the interaction between adjacent non-structural components and systems is considered as another cause of non-structural damage (Fig. 4.28). An example is the case of mechanical and electrical components, such as lights, air diffusers, sprinkler pipes that interact and move differently with suspended ceilings causing considerable damages and service interruptions especially for essential facilities.

As discussed previously, the degree of damage of non-structural components and systems is related to the four causes listed above. The present discussion illustrates the earthquake effects and typical damages on lightweight

Fig. 4.30: Out-of-plane failure of an unbraced partial-height lightweight steel gypsum board partition due to the inertial forces, 1994 Northridge Earthquake, Los Angeles, USA /Wiss, Janney, Elstner Associates, Inc./

gypsum board partitions, suspended acoustic lay-in tile ceilings and suspended lightweight gypsum board ceilings. This section is accompanied by a photo documentation regarding the seismic performance of these systems during recent earthquakes, such as the 1994 Northridge Earthquake, 2009 Abruzzo Earthquake, 2010 Chile Earthquake and 2012 Emilia Earthquake.

ASCE/SEI 41-13 classifies lightweight steel gypsum board partitions as both deformation-and-acceleration-sensitive (see Tab. 4.1).

Primarily, the building deformations may cause in-plane damage to lightweight steel partitions. Full-height lightweight steel partitions, spanning floor-to-floor and attached at the top and bottom of the structure, may undergo shear cracking and significant distortions due to inter-storey drift of the structural system /4.10/. In these cases, typical in-plane damage are the breaking of surface materials, frame deformation and connection failing (Fig. 4.29a). A particular case is the in-plane damage of coupled wall elements, in which the weakest points of the system are the ends of wall panels subjected to different relative motions. In this case, the cracks appear at the opening corners (Fig. 4.29b).

Secondarily, the inertial forces induced by floor accelerations may cause out-of-plane damage to lightweight steel partitions rigidly attached to two adjacent floors. In these cases, typical out-of-plane damages involve the flexural cracking due to high accelerations, the failure of top connections leading to the partition overturning or the complete failure of connections and, thus, the collapse of the partition. Partial-height partitions inadequately braced to the structure above (Fig. 4.30) and partitions with high weight, heavy finishes or heavy items anchored to them (e.g. bookshelves, equipment or

Fig. 4.31: Damage to suspended acoustic lay-in tile ceilings, 2009 Abruzzo Earthquake, L'Aquila, Italy

a) Damage at the ceiling perimeter, L'Aquila University, 2009 Abruzzo Earthquake, L'Aquila, Italy /Angelo Masi/

b) Loss of the finish materials caused by the seismic interaction between ceiling and reinforced concrete structure, Terraustral Oeste College, 2010 Chile Earthquake /Knauf Chile/

Fig. 4.32: Damage to suspended lightweight steel gypsum board ceilings

other non-structural components) are more vulnerable to out-of-plane damage /4.6/.

These considerations show that the parameters controlling the seismic performance of lightweight steel gypsum board partitions are the weight and the height of partition systems and the attachment conditions with floor or roof structure or ceilings.

Suspended acoustic lay-in tile ceilings are classified as both deformation-and-acceleration-sensitive /4.7/ (see Tab. 4.1).

The building deformations may cause damage to these ceiling systems, since the differential movement with the structural system or other non-structural components, such as partition walls, may locally damage the ceiling.

However, the seismic performance of the suspended acoustic lay-in tile ceilings is mainly affected by the supporting system, i.e. the lightweight steel T-bar grids that are usually completed and stabilized by closure panels. The typical damage occurs initially at the ceiling perimeter, where it is highly vulnerable, and it is characterized by tile dislocation or falling and metal grid deformation (Fig. 4.31). This last type of damage is usually caused by the separation between channels and cross channels that are the main and secondary lightweight steel members for supporting the ceiling systems, and they are usually realized with different cross-section shapes depending on the design requirements of flexibility, safety and resistance. In this case, the ceiling system could become unstable and sway uncontrollably leading to its collapse /4.6/. This damage type may be avoided if the T-bar frame

Fig. 4.33: Failure of a large suspended lightweight steel gypsum board ceiling, 2012 Emilia Earthquake, Mantova, Italy /Marco Savoia/

is securely braced to the structure above. Suspended acoustic lay-in tile ceilings characterized by long spans and included in flexible structural systems or completed with heavy closure panels, which have weights greater than 0.10 kN/m^2, may represent an important risk for life safety, if they are not adequately designed /4.7/.

Furthermore, since the suspended acoustic lay-in tile ceilings are generally integrated with lighting fixtures, diffusers and mechanical ducts, they are also susceptible to the duct penetrations and to the differential movement with these items, whose interaction can cause damage to both the ceiling system and the other mechanical and electrical components.

Therefore, the seismic behaviour of suspended acoustic lay-in tile ceilings is strongly influenced by many variables, such as the mechanical properties of the lightweight steel supporting system, the characteristics of the ceiling finish material, the bracing and the attachment between the supporting system and structure as well as the interaction with other structural and non-structural components.

Suspended lightweight steel gypsum board ceilings are primarily considered to be acceleration-sensitive (Tab. 4.1).

Typical damage to these ceiling systems are the supporting grid deformation and the cracking of finish materials (Fig. 4.32a, b). This damage type could be prevented by properly installing the hanger wires supporting the ceiling system, by adequately anchoring the finish material to the lightweight steel furring grid and isolating the ceiling movement from that of the structural system or from other non-structural components (such as partitions, lights and diffusers). Regarding this latter aspect, ceiling systems could be damaged also by the presence of partial-height partitions attached to them, which could cause damage, with particular reference to the ceiling frame and the suspension and bracing systems.

Fig. 4.34: *Life safety risk: Damage to non-structural components, 2009 Abruzzo Earthquake, L'Aquila, Italy /Angelo Masi/*

Fig. 4.35: *Property loss risk: Damage to scaffolds in storehouses, 2012 Emilia Earthquake, Sant'Agostino, Italy /Marco Savoia/*

Fig. 4.36: Functionality loss risk: Damage to essential services in a biomedical company, 2012 Emilia Earthquake, Medolla, Italy /Marco Savoia/

Furthermore, suspended lightweight steel gypsum board ceilings are considered rigid in their plane, and when they are characterized by long spans, they may be highly vulnerable if they are not adequately designed (Fig. 4.33).

The typical damage to non-structural components and systems due to a seismic event involves three kinds of risks /4.6/:

- Life safety
- Property loss
- Functionality loss

These kinds of risks are the direct or indirect consequences that may result from damage to non-structural components and systems.

Life safety is defined as the risk associated with the loss of life or injury to the building occupants. For example, partition and ceiling failures may directly cause falling hazards and indirectly impede the rescue operations following an earthquake (Fig. 4.34).

Property loss is defined as the risk associated with the economic losses due to the damage to non-structural components and systems that represent the largest capital investment in most commercial buildings. For instance, in the recent Emilia-Romagna earthquake in Italy, important economic losses related to the damage to Italian commercial products were observed, e.g. Parmesan cheese and ceramics (Fig. 4.35).

The third kind of risk is associated with the loss of function of important buildings or lifeline structures, such as hospitals, police stations, manufacturing facilities, business and government offices that must suspend normal activities because of the non-structural damage. The interaction between non-structural components and systems may often cause failures that interrupt the use until the utilities are repaired. Furthermore, inactivity or reduced productivity of a facility are considered to be additional potential consequences of the building functionality loss (Fig. 4.36).

4.4.2 Performance objectives

Building performance is a combination of the performance of both structural and non-structural components /4.7/. Therefore, it is evident that the issue of the non-structural components does not play a secondary role in the ever-changing building codes and particularly in the more advanced ones, which are based on the performance-based design philosophy (see chapter 2 for more details). The main reason is related to the vulnerability and the higher seismic fragility of the non-structural components compared to supporting structures, therefore, they may be damaged by relatively low seismic intensity levels compared to those required for structural damage. This issue was highlighted in the 1971 San Fernando earthquake (Fig. 4.37), in which the load-bearing structures suffered slight damage, while the non-structural components were severely damaged resulting in significant economic losses and posing significant threat to life /4.1/. For instance, based on a survey carried out on 25 damaged commercial buildings during the 1971 San Fernando earthquake, the recorded property losses revealed that 3 % was attributable to structural damage and the remaining 97 % to non-structural damage (7 % for electrical and mechanical components, 34 % for exterior finishes and 56 % for interior finishes) /4.6/. For this reason, the 1971 San Fernando earthquake was the first disaster event that highlighted the awareness of paying more attention to the seismic design of the non-structural components and systems.

The main objective of the performance-based design applied to the non-structural components is to provide an adequate level of safety and protection to human

Fig. 4.37: Damage to non-structural components at the Olive View Medical Treatment Building, 1971 San Fernando Earthquake, Los Angeles, USA /NISEE-PEER/

life, by taking into account the actual seismic hazard and the importance of the non-structural components inside the building. In addition, higher levels of non-structural performance may be required to limit the building damage or ensuring uninterrupted post-earthquake operation of the building. These more complex requirements are assumed for historic facilities, essential facilities (hospitals, police and fire stations), manufacturing facilities and businesses, whose revenue losses may involve the functional interruption following an earthquake.

The seismic risk reduction of the non-structural components is implemented in different ways, depending on the building types, in which the non-structural components are installed, such as existing buildings, historic facilities, essential facilities or new buildings. The major advancements in the performance-based design concepts for non-structural components are included into the American building standards used for existing buildings, namely ASCE/SEI 41-13 "Seismic Evaluation and Retrofit of Existing Buildings" /4.9/, and for new constructions, namely ASCE/SEI 7-10 "Minimum Design Loads for Buildings and Other Structures". The present discussion is not intended to specifically deal with the design procedures described in the above stated codes, but rather the purpose is to illustrate the decision-making process aimed at choosing the performance objectives for existing and new buildings.

ASCE/SEI 41-13 describes the design procedure for existing buildings (Fig. 4.38) to be adopted for determining the building performance objectives", which are defined by combining the seismic hazard levels with the target building performance levels /4.12/. The building performance objectives are chosen in order to evaluate the risk type to be addressed, building performance likely after an earthquake and acceptability of structural and non-structural damage. In particular, this standard introduces three main sets for the building performance objectives:

- Basic performance objective
- Enhanced performance objective
- Limited performance objective

The buildings meeting the basic performance objective are expected to experience little damage from relatively frequent and moderate earthquakes, but significantly more structural and non-structural damage and potential economic losses from the most severe and infrequent earthquakes.

The enhanced performance objective is a more ambitious objective than that basic one, which permits targeting a seismic evaluation or to perform a seismic retrofit to a level greater than the basic performance objective. The purposes of this performance objective are to preserve the post-earthquake building operations, to ensure reduced damage and to increase the functionality. This more stringent requirement should be adopted in the case of essential facilities, and it involves a more careful design of a broader range of non-structural components. The enhanced performance objective may be obtained by using higher target building performance levels, higher seismic hazard levels, a higher risk category than the building would normally be assigned, or any combination thereof.

On the other hand, the limited performance objective is a less ambitious objective than the basic objective and thus is defined as the opposite of the enhanced performance objective. The aim of this performance objective is to address only serious non-structural falling hazards. In this case, the seismic evaluation or the seismic retrofit are performed to a level less than the basic performance objective, by using lower target building performance levels, lower seismic hazard levels or a lower risk category. According to ASCE/SEI 41-13, the building performance objectives are obtained by the target building performance

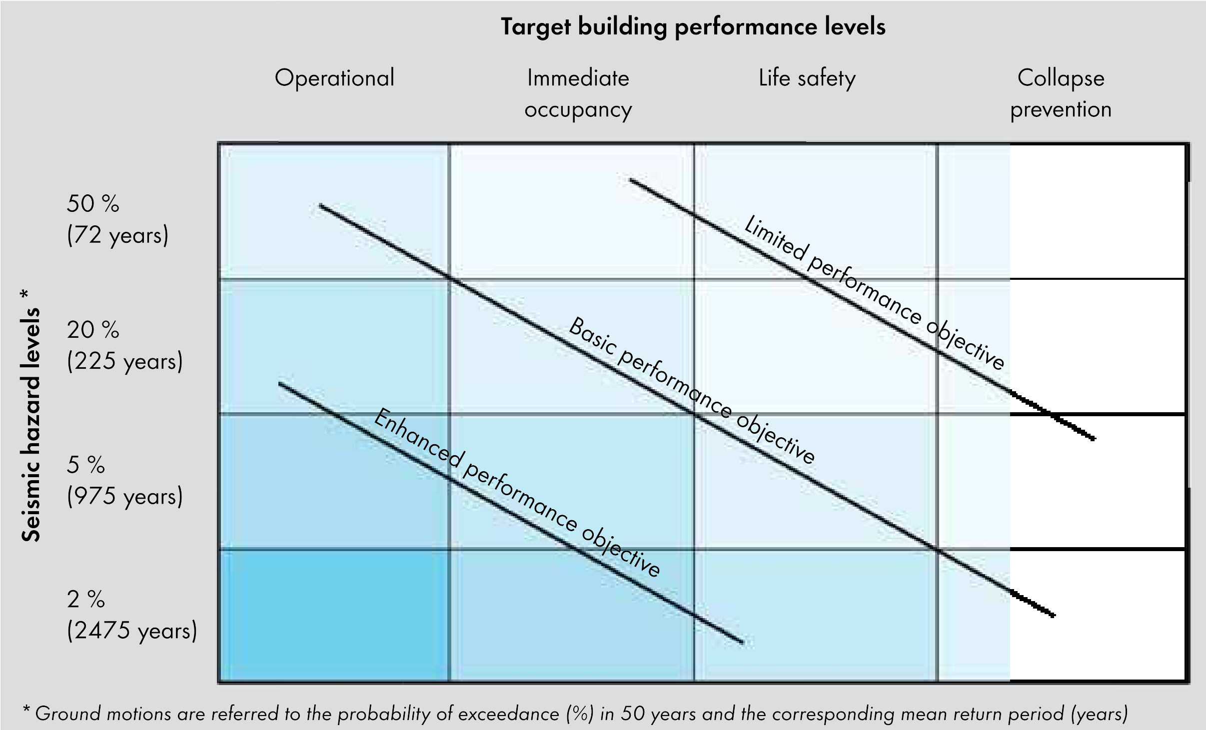


Fig. 4.38: Building performance objectives acc. to ASCE/SEI 41-13 (2013)

levels, which are defined as discrete damage states selected from the infinite damage states experienced by buildings during earthquakes. The damage states representative of the target building performance levels are selected on the basis of the damage consequences that are considered significant by the community, which depend on the choice to ensure the ordinary functions or the immediate occupancy of a building after earthquakes, or to avoid compromising life safety.

The design at different target building performance levels results in higher or lower seismic design forces and in specific requirements for more or fewer non-structural components /4.6/. The target building performance levels are designated according to ASCE/SEI 41-13 as combinations of target structural performance levels and target non-structural performance levels. Tab. 4.7 describes the estimated levels of structural and non-structural damage, which could be expected by rehabilitated buildings according to the different target building performance levels. These descriptions represent the damage condition for given seismic intensities that could affect a building.

The present discussion is focused particularly on the target non-structural performance levels, while the target structural performance levels are widely discussed in Chapter 2. ASCE/SEI 41-13 defines four discrete target non-structural performance levels for a building, which are summarized in Fig. 4.39:

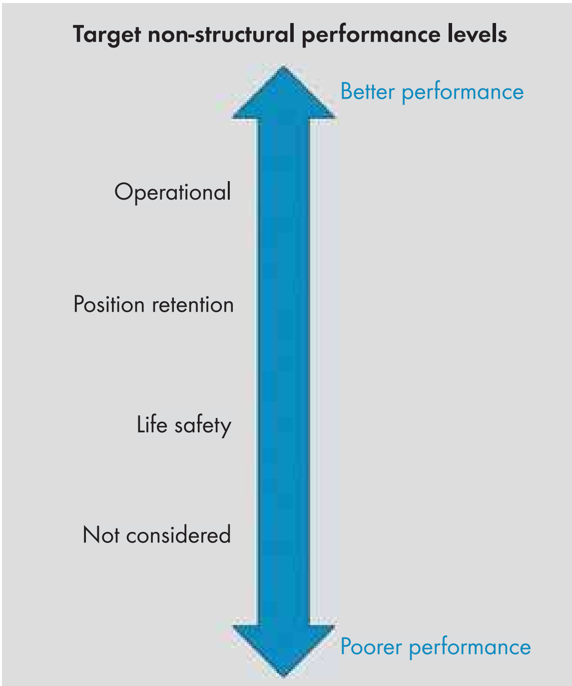


Fig. 4.39: Target non-structural performance levels

- Operational
- Position retention

Tab. 4.7: Target Building Performance Levels and damage control /ASCE 41-13 (2013)/

	Target Building Performance Levels			
	Collapse Prevention	**Life Safety**	**Immediate Occupancy**	**Operational**
Overall damage	Severe	Moderate	Light	Very light
Structural components	Little residual stiffness and strength to resist lateral loads, but gravity-load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Building is near collapse for aftershocks and should not continue to be occupied.	Some residual strength and stiffness left in all storeys. Gravity-load-bearing elements function. No out-of-plane failure of walls. Some permanent drift. Damage to partitions. Continued occupancy might not be likely before repair. Building might not be economical to repair.	No permanent drift. Structure substantially retains original strength and stiffness. Continued occupancy likely.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of façades, partitions and ceilings as well as structural elements. All systems important to normal operation are functional. Continued occupancy and use highly likely.
Non-structural components	Extensive damage. Infills and unbraced parapets failed or failure imminent.	Falling hazards, such as parapets, mitigated, but many architectural, mechanical and electrical systems are damaged.	Equipment and contents are generally secure, but might not operate due to mechanical failure or lack of utilities. Some cracking of façades, partitions and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable. Less damage and low life safety risk.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.

- Life safety
- Not considered

The operational performance level is defined as the post-earthquake damage state in which the non-structural components are able to resume their pre-earthquake function.

The position retention performance level is defined as the post-earthquake damage state in which the non-structural components are damaged and may not function, but they are secured in place following the earthquake.

The life safety performance level is defined as the post-earthquake damage state in which the non-structural components are damaged and dislodged from their position, but the consequences of the damage do not pose a risk to life safety. The objective of this performance level is the elimination of falling hazards associated with the non-structural components, though this condition may imply that the non-structural components are not functional or repairable after strong earthquakes.

The not considered performance level is adopted when the building rehabilitation is not addressed to reduce the non-structural components' risk. In fact, the non-structural rehabilitation may compromise the ordinary building activities and, in some cases, in order to avoid functional interruptions, it may be preferable not to deal with the reduction in non-structural vulnerability.

Tab. 4.8 describes the estimated levels of non-structural damage for the architectural components according to

Tab. 4.8: Target non-structural performance levels and damage control – Architectural components /ASCE/SEI 41-13 (2013)/

Component group	**Target non-structural performance levels**		
	Life safety	**Position retention**	**Operational**
Partitions (plaster and gypsum)	Distributed damage; some severe cracking, crushing and racking in some areas.	Cracking at openings. Minor cracking and racking throughout.	Minor cracking.
Ceilings	Extensive damage. Plaster ceilings cracked and spalled, but did not drop as a unit. Tiles in grid ceilings dislodged and falling grids distorted and pulled apart. Potential impact on immediate egress. Potential damage to adjacent partitions and suspended equipment.	Limited damage. Plaster ceilings cracked and spalled, but did not drop as a unit. Suspended ceiling grids largely undamaged, though individual tiles falling.	Generally negligible damage with no impact on reoccupancy or functionality.

ASCE/SEI 41-13.

ASCE/SEI 7-10 describes the design procedure for new buildings and, by including the performance-based design criteria, it specifies quite comprehensive requirements for the non-structural components to be adopted only in some facility types. Mentioning the terminology used in ASCE/SEI 41-13 and previously illustrated, ASCE/SEI 7-10 defines the target building performance levels as basic and enhanced, while the limited performance objective is not permitted for new constructions (see Chapter 2 for more details). The basic performance objective is adopted by following the requirements specified by the code for standard occupancies, while the enhanced performance objective is pursued by using the requirements defined by the code for essential facilities /4.6/. However, this standard does not preclude the possibility of adopting the enhanced performance objective also for non-essential facilities, by developing and implementing specific seismic design criteria defined with the clients' needs. For this purpose, the designers may rely on the proper structural analysis, in order to check the compliance between design and performance objectives specified in the planning phase.

The performance-based design concepts illustrated above highlight an important aspect that is the need for discussion and agreement between the designers and clients regarding the choice of performance objectives. In this way, the definition of demanded seismic performance level of the non-structural and structural components represents the starting point of the planning process and thus is an integral part of seismic design.

Furthermore, since the seismic design of non-structural components is a balance between the potential losses and the costs of damage mitigation, an understanding of the costs and benefits is an important issue of performance-based design applied to the non-structural components. A preventive economic analysis, which is aimed at assessing the direct costs of seismic damage and the indirect losses due to the interruption of building activities, should guide the selection of the performance objectives /4.8/.

4.4.3 Damage mitigation

The performance-based design concepts for non-structural components and systems involve the suitable design of seismic anchorages and bracings. With specific reference to lightweight steel gypsum board partitions and suspended lightweight steel gypsum board ceilings, FEMA E-74 provides many techniques to reduce their potential risk, by indicating mitigation measures of the seismic damage for these systems. These protective measures should be identified based on the importance of the non-structural components, the consequences of their failure and the context in which they are installed. Furthermore, the selection of the damage mitigation measures must

Tab. 4.9: The non-structural seismic mitigation methods for architectural components /FEMA E-74 (2011)/

Architectural component	Seismic mitigation methods
Partitions	
Light	ER
Ceilings	
Suspended gypsum board ceilings	PR
Suspended acoustic lay-in tile ceilings	PR
PR: Prescriptive details; ER: Engineering Required details	

be consistent and in agreement with the objectives and non-structural seismic performance levels defined in the planning process. This aspect involves an appropriate choice of non-structural mitigation design methods, since a component that must meet the operational or position retention performance levels requires more attention and complexity than another that must satisfy the life safety level.

The design solutions for non-structural components and systems provided in some documents, such as FEMA E-74 and FEMA 454 /4.6, 4.20/, are schematic and common solutions given with the intent to provide correct design indications. The non-structural seismic mitigation design methods are generally classified into three broad categories:

- Non-engineered (NE) details
- Prescriptive (PR) details
- Engineering required (ER) details

Non-engineered details are defined as generic seismic protection methods that do not require the engineering design. The non-engineering protection measures should be applied only for lightweight components installed in non-critical facilities.

Prescriptive details are defined as standard seismic restraint methods developed in specific installation guidelines, which allow the direct mounting of the restraint details without the need for engineering design. These protection measures should be used only for suspended acoustic lay-in tile ceilings with weights up to 0.19 kN/m^2 in non-critical facilities.

Engineering required details are defined as designed seismic bracing, anchorage and restraint methods that require the engineering design. These protection measures should be adopted in essential facilities.

Tab. 4.9 defines the non-structural seismic mitigation methods for lightweight partitions and suspended ceilings. Regarding the seismic mitigation measures, ASCE/SEI 7-10 provides some general design requirements for the anchorage and bracing details of architectural components, e.g. partitions and suspended ceilings. In particular, the code sets out a general principle, according to which a continuous load path with sufficient strength and stiffness between the components and the structural system should be provided. This condition is ensured by satisfying the requirements about the component attachment type listed in this standard.

Seismic mitigation details for lightweight steel gypsum board partitions

According to ASCE/SEI 7-10, partitions that are connected to the ceiling and partitions higher than 1.80 m should always be laterally braced to the supporting structure, independently of ceiling bracing. Nevertheless, this code does not provide specific seismic mitigation details, which instead are widely discussed in FEMA E-74. For lightweight gypsum board partitions, it is necessary to distinguish the cases of full-height partitions and of partial-height partitions. The damage type and degree of these systems are controlled by the details at the top connection of partition walls.

In order to isolate them with building deformations, full-height partitions should be protected by providing an in-plane slip joint at the top connection, while they should be fixed at the base. In this way, since the slip joint is able to accommodate the in-plane movement, the partition walls are free to slide at the top and they are also restrained out-of-plane. In particular, the seismic detail requires that the studs and full-height gypsum

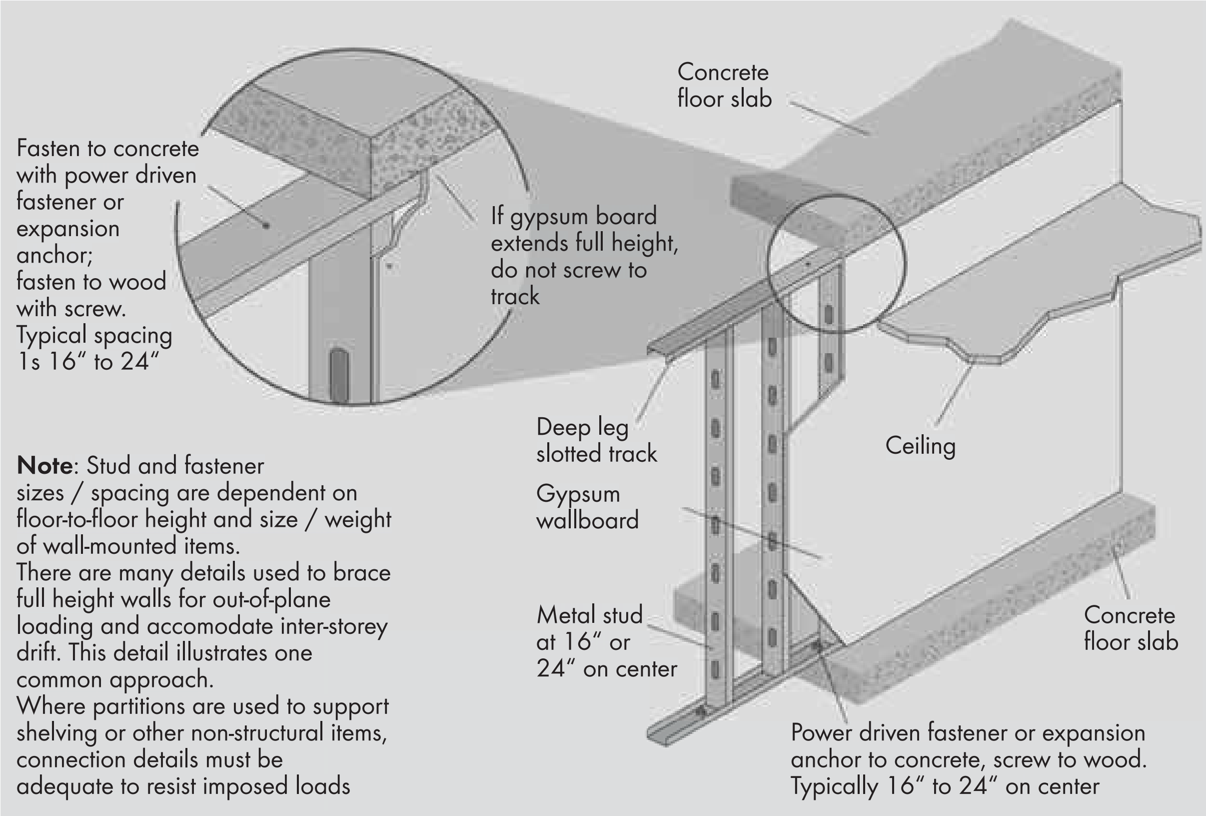


Fig. 4.40: Seismic bracing for full-height lightweight gypsum board partitions /FEMA E-74 (2011)/

board panels are not screwed to the top track (Fig. 4.40). Additional requirements for fireproofing, waterproofing and soundproofing may be required. When full-height partitions are located in low-height buildings, they should be fixed to the top structural elements by providing additional lateral resistance systems to the building.

Partial-height partitions, which are more susceptible to out-of-plane damage, should be laterally braced to the structural elements above, but not to the suspended ceiling systems thus avoiding interaction between them. According to FEMA E-74, the stud braces, which are usually connected to the upper structure and to the partition top with appropriate angle profiles, are required at intervals between 1.20 m and 2.40 m in the longitudinal direction of the partition wall. The brace spacing is defined for limiting the horizontal deflection at the partition top that should be compatible with the ceiling deflection. If the distance between the partial-height partition and the overhead structure exceeds 1.80 m, braces could be realized with boxed studs or back-to-back studs (Fig. 4.41).

If the partition walls are used to laterally support other non-structural components, thicker studs and appropriate top attachments for full-height partitions and thicker braces or closer spacing between them for partial-height partitions should be adopted in order to resist additional loading.

The intersection details between interconnected perpendicular walls should be correctly designed, since restrained partition walls in one direction could restrain the partition slip in the other direction.

Seismic mitigation details for suspended acoustic lay-in tile ceilings

The damage mitigation measures for suspended acoustic lay-in tile ceilings and suspended lightweight steel gypsum board ceilings require prescriptive details about the seismic bracings and the perimeter conditions, in order to limit the ceiling movement.

Regarding the suspended acoustic lay-in tile ceilings, ASCE/SEI 7-10 requires that the suspended acoustic ceiling located in low seismicity areas should not necessarily be restrained, but they should accommodate only horizontal movement of the supporting structure.

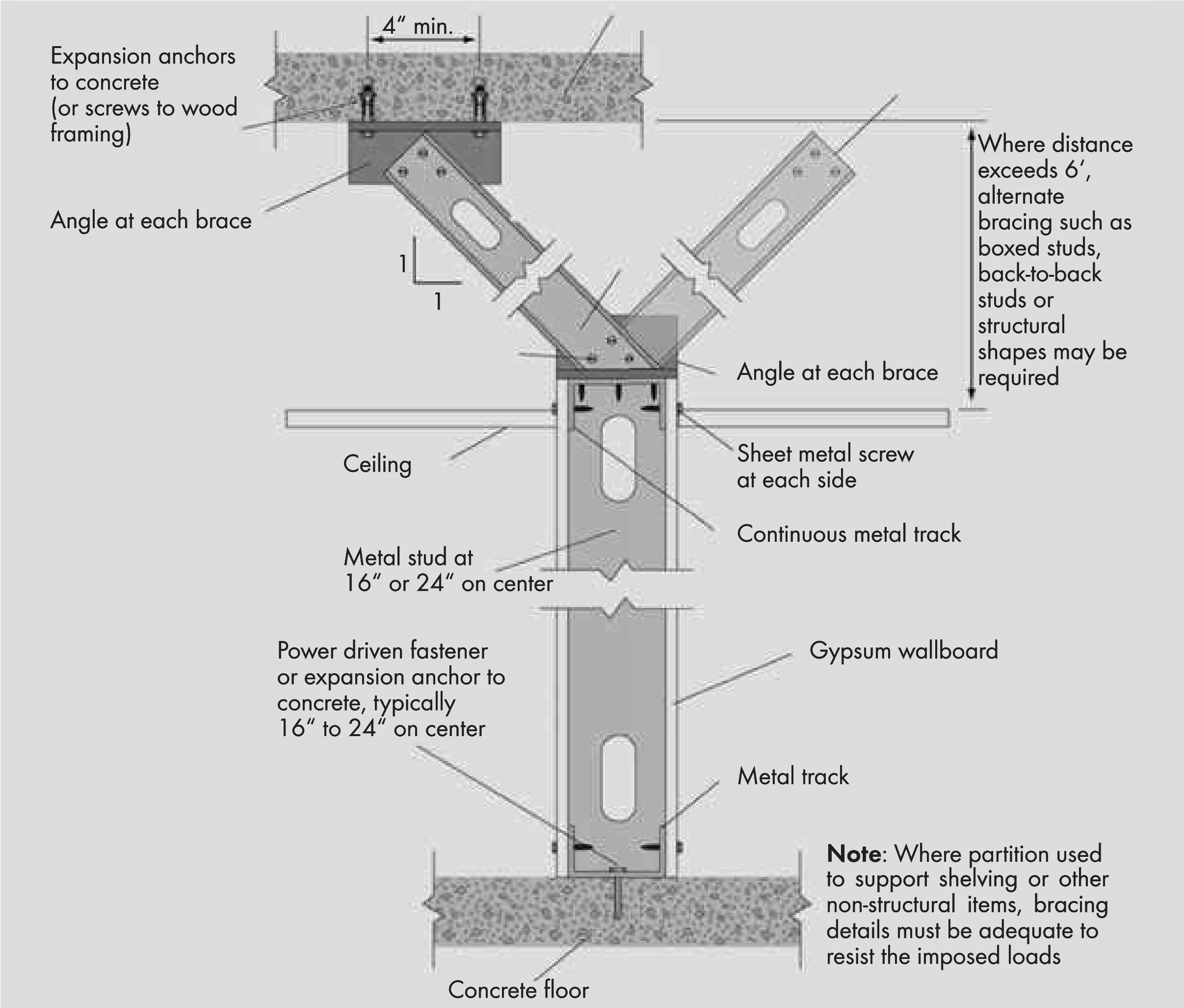


Fig. 4.41: Seismic bracing for partial-height lightweight gypsum board partitions /FEMA E-74 (2011)/

On the other hand, in high seismicity areas and for essential facilities, this standard requires restrained suspended acoustic ceilings with appropriate rigid or non-rigid lateral bracings and specific perimeter details. These seismic design requirements are adopted for suspended acoustic ceilings larger than 92 m^2 and with a total weight greater than 0.19 kN/m^2. The standard includes special mitigation measures for suspended acoustic ceiling systems larger than 232 m^2, requiring seismic separation joints or full-height partitions that separate the total surface in restrained ceiling portions, each of which having a ratio between the long and short dimensions less than or equal to 4.

Non-rigid seismic bracings, which laterally secure the ceiling systems to the overhead structure at regular intervals, should be usually realized with a vertical compression steel strut attached to main runners and four-way diagonal wire braces (Fig. 4.42a) /4.6/. Sometimes, cold-formed steel profiles could be used as rigid bracings by replacing the compression strut and the diagonal wire bracings (Fig. 4.42b). Regarding the compression strut size, a 2.50 mm diameter wire may be used for distances between ceiling and upper structure up to 1.80 m or a cold-formed steel stud profile for distances up to 3.00 m (Fig. 4.43). Fig. 4.44 shows the diagonal wire brace attachment to a concrete floor or roof and to a steel deck with concrete fill. Moreover, FEMA E-74 provides prescriptive details based on Californian standards for schools and essential buildings. According to this code, the lateral bracings should be placed at a spacing between 2.40 m and 3.60 m in each direction for essential buildings, and not more than 3.60 m in each direction for school buildings (Fig. 4.45). The ceiling suspension system should be completed with vertical

a) *Example of non-rigid bracing assemblies /FEMA E-74 (2011)/*

b) *Example of rigid bracing assemblies /FEMA E-74 (2011)/*

Fig. 4.42: Seismic bracings for suspended acoustic lay-in tile ceilings

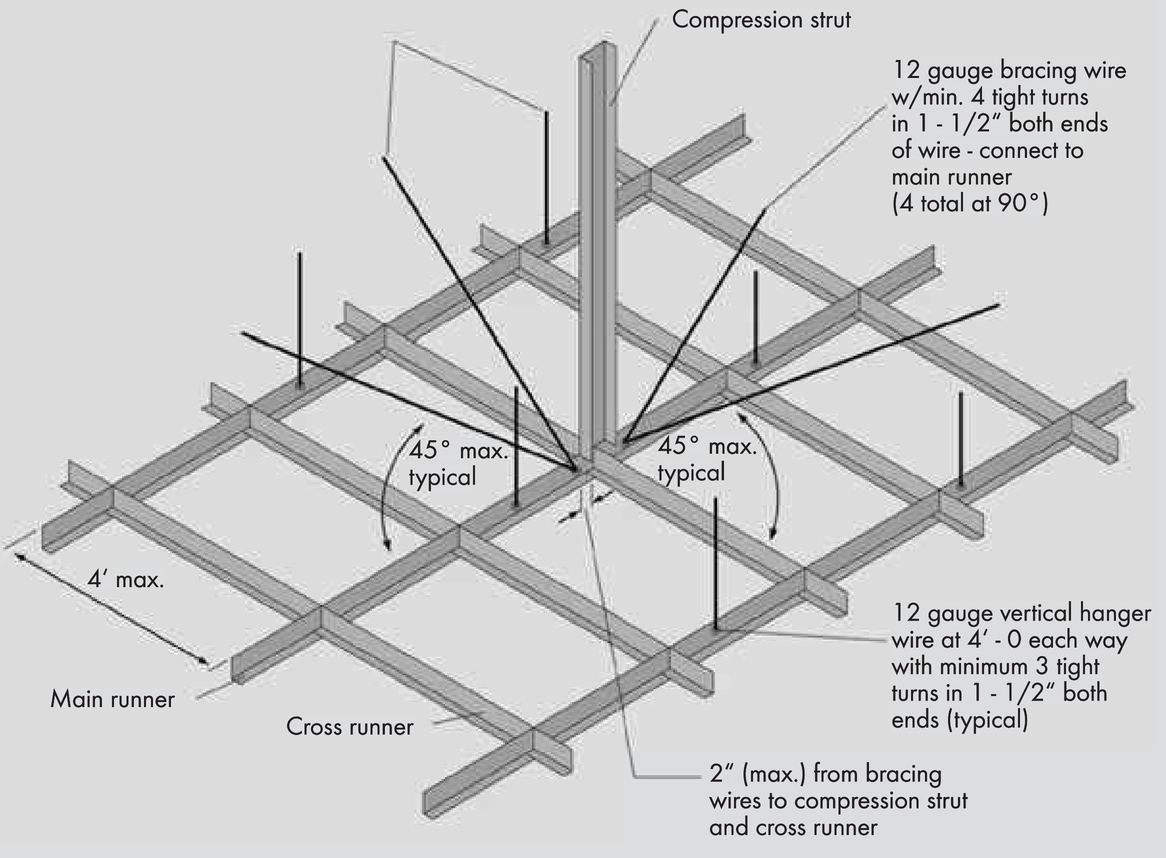


Fig. 4.43: Seismic bracing for suspended acoustic lay-in tile ceilings / FEMA E-74 (2011)/

hanger wires placed along the main runners at intervals of at least 1.20 m and securely attached to the structure above.

Furthermore, for reducing the seismic damage at the ceiling perimeter, the ceiling grid should be rigidly attached along two adjacent walls and separated from the two opposite walls with a clearance of at least 2 cm. At the ceiling perimeter, angle profiles provide the vertical support to the ceiling system, and they should not be connected to the ceiling grid at the floating edges. This mitigation detail allows free sliding of the ceiling systems and avoids buckling and pulling of the ceiling grid, by

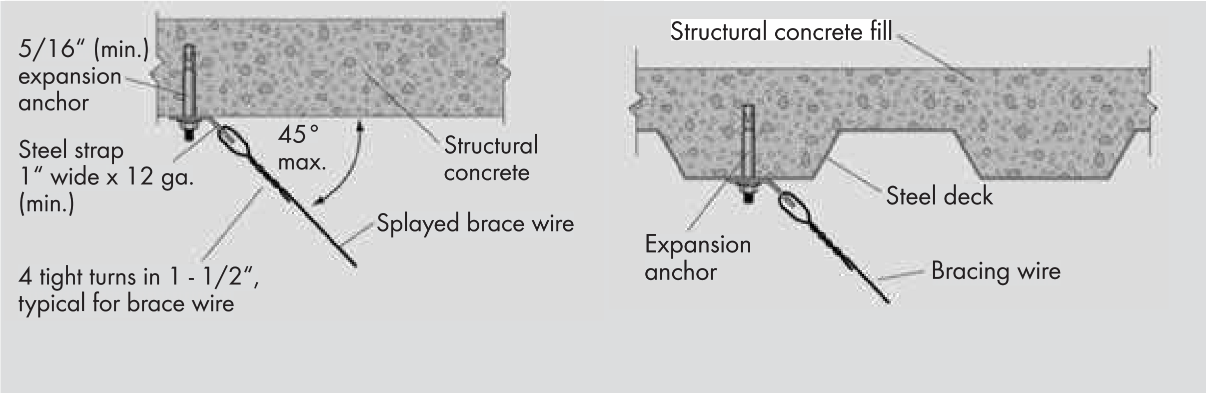


Fig. 4.44: Overhead attachment details of the diagonal wire braces

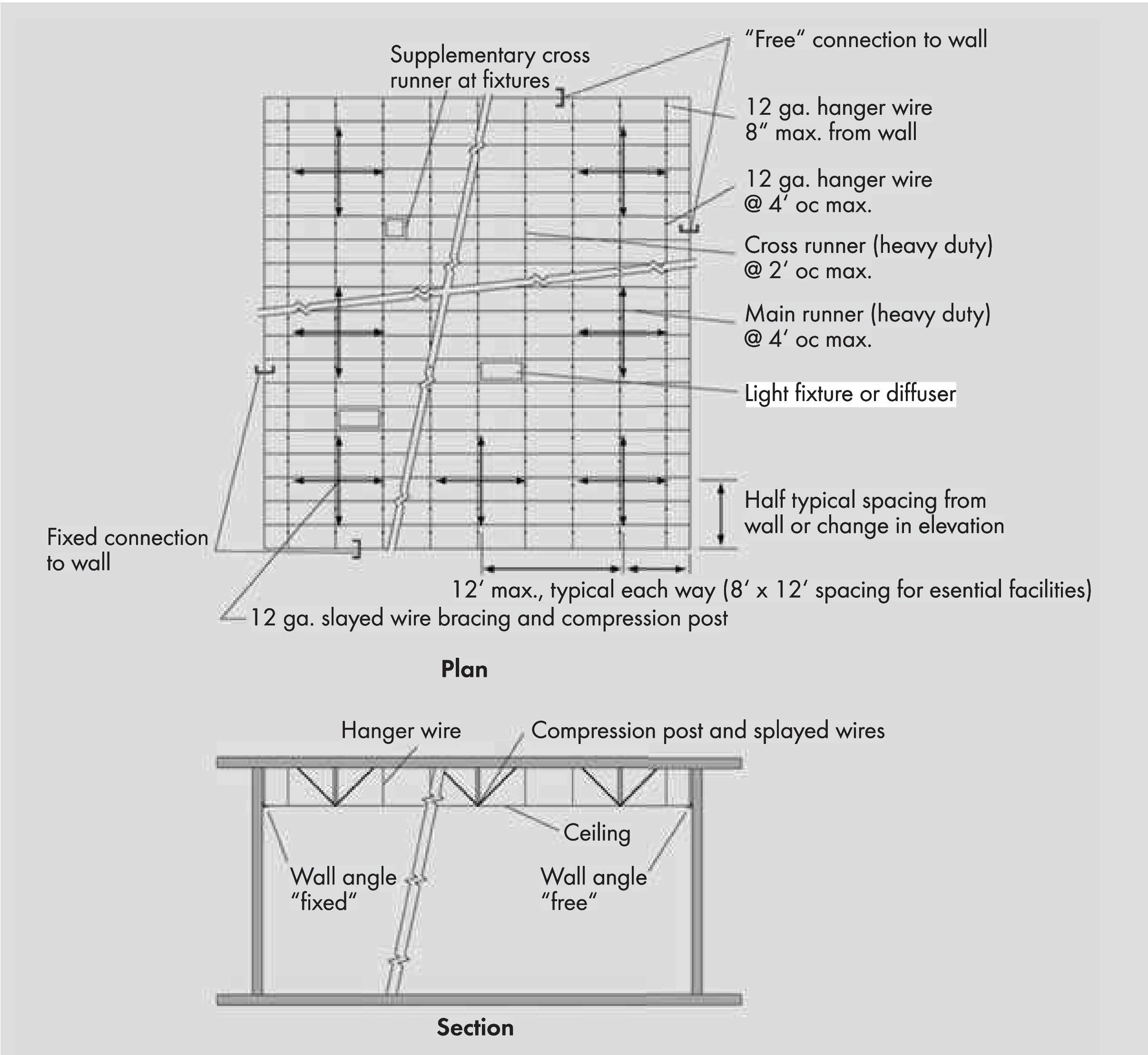


Fig. 4.45: General layout of the seismic bracings for suspended acoustic lay-in tile ceilings /FEMA E-74 (2011)/

allowing free wall deformation during an earthquake. Supplemental framing and hanger wires may be required for light fixtures, diffusers and other mechanical and electrical components supported by the ceiling grid. However, if the weight of supported items exceeds the loading capacity of the ceiling grid, independent wires attached directly to the structure are preferred for reducing the potential risk of falling for these heavy items.

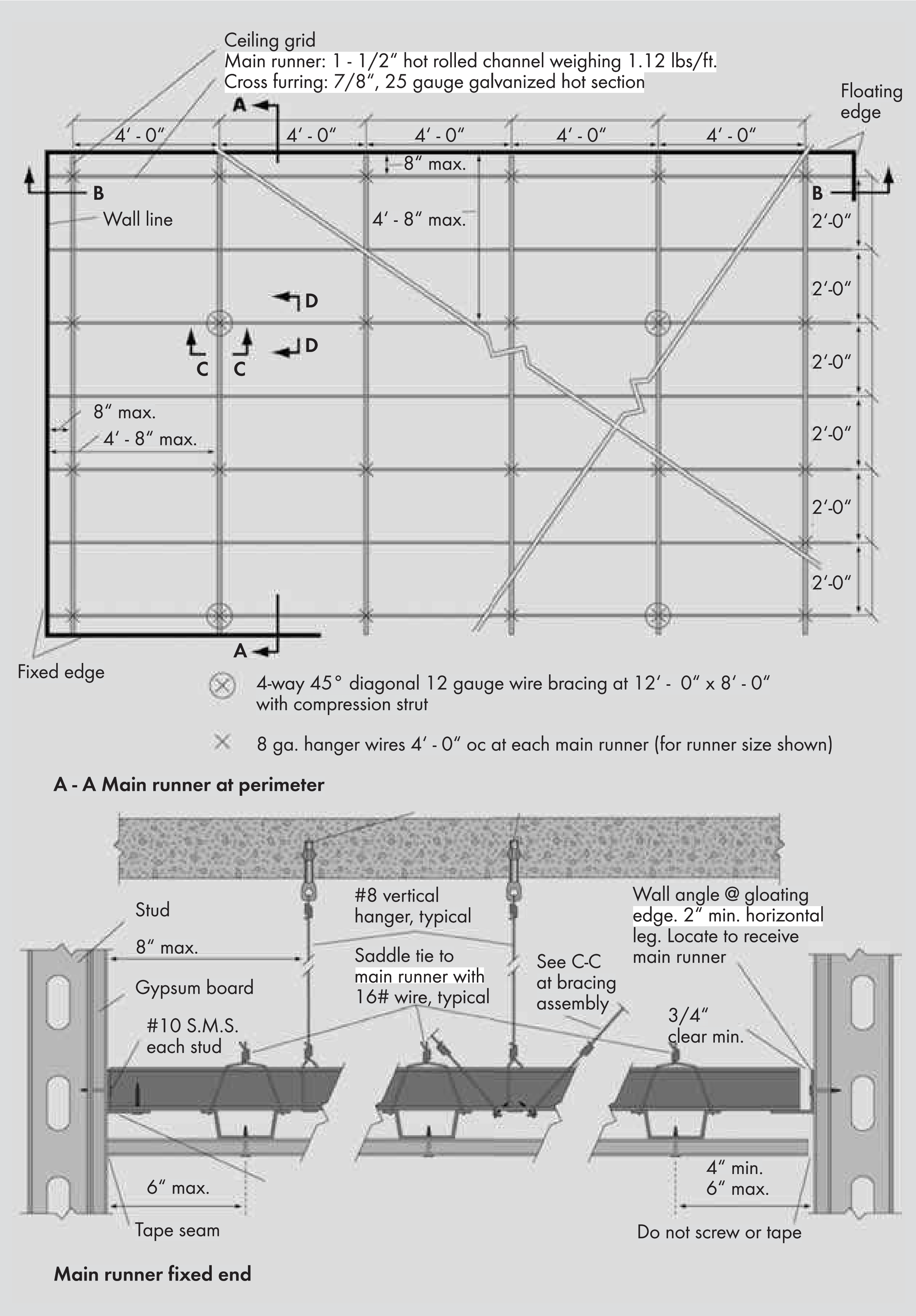


Fig. 4.46: General layout, arrangement of typical seismic bracing assemblies and perimeter details for suspended lightweight gypsum board ceiling /FEMA E-74 (2011)/

Tab. 4.10: Summary of non-structural seismic mitigation details

Architectural component	**Seismic mitigation details**
Lightweight gypsum board partitions	• Partitions connected to suspended ceilings and partitions higher than 1.80 m should be laterally braced to the above structural elements, independently of ceiling bracing. • Full-height partitions should be seismically protected by providing an in-plane slip joint at the top connection, obtained without screwing the studs and gypsum board panels to the top track. • Partial-height partitions should be laterally braced to the above structural elements with stud braces placed at intervals between 1.20 m and 2.40 m in the partition's longitudinal direction. If the distance between the partial-height partition and the overhead structure exceeds 1.80 m, braces could be realized with boxed studs or back-to-back studs.
Suspended acoustic lay-in tile ceilings	• Suspended acoustic ceilings larger than 92 m^2 and with a total weight greater than 0.19 kN/m^2 should be laterally braced to the above structural elements. Suspended acoustic ceiling systems with a total surface larger than 232 m^2 should be divided in restrained ceiling portions by means of seismic separation joints or full-height partitions. • In high seismicity areas and for essential facilities, suspended acoustic ceilings should be appropriately restrained with non-rigid or rigid lateral bracings. • Non-rigid seismic bracings should be realized with a vertical compression steel strut attached to main runners and four-way diagonal wire braces. As regard the compression strut size, a 2.50 mm diameter wire may be used for distance between the ceiling and above structure up to 1.80 m. • For distance between the ceiling and above structure up to 3.00 m, rigid seismic bracings should be realized by replacing the compression strut and the diagonal wire bracings with cold-formed steel profiles. • The lateral bracings should be placed at a spacing between 2.40 m and 3.60 m in each direction for essential buildings, and not more than 3.60 m in each direction for school buildings. • The ceiling suspension system should be completed with vertical hanger wires placed along the main runners at intervals of at least 1.20 m and securely attached to the structure above. • The ceiling grid should be rigidly attached along two adjacent walls and separated from the two opposite walls with a clearance at least of 2 cm.
Suspended lightweight gypsum board ceilings	• Suspended gypsum board ceilings, with areas less than or equal to 13.40 m^2 or made of gypsum board panels in a single plane, do not have to meet the seismic design requirements if they are surrounded by and connected to walls or soffits laterally braced to the structure above. • The seismic mitigation details for suspended lightweight steel gypsum board ceilings are similar to those illustrated above for suspended acoustic ceilings.

Seismic mitigation details for suspended lightweight steel gypsum board ceilings

Moreover, ASCE/SEI 7-10 also provides important design requirements for suspended lightweight gypsum board ceilings realized at multiple levels and for other suspended heavy ceilings that are completed with plaster, wood or metal panels. The seismic design requirements listed in this standard are not required for suspended ceilings with areas less than or equal to 13.40 m^2 or made of gypsum board panels in a single plane that are surrounded by and connected to walls or soffits laterally braced to the structure above.

Regarding the seismic bracings and the ceiling perimeter details for suspended lightweight steel gypsum board ceilings, the seismic mitigation methods are similar to those illustrated above for suspended acoustic ceilings. Fig. 4.46 shows the general layout, the arrangement of typical seismic bracing assemblies, and the perimeter details for suspended lightweight steel gypsum board ceiling.

In the case of suspended lightweight steel gypsum board ceilings, light fixtures, diffusers and other mechanical and electrical components should be independent of the structural system and supported directly by main runners or by supplemental framing connected to main runners. Tab. 4.10 summarizes the seismic mitigation details illustrated for lightweight gypsum board partitions, suspended acoustic lay-in tile ceilings and suspended lightweight steel gypsum board ceilings.

4.5 Codification

4.5.1 General

The seismic analysis of non-structural components and systems by means of rational methods has been broadened over the past 40 years. The issue arose following the 1964 Alaska and 1971 San Fernando earthquakes that led to the inclusion of the seismic analysis procedure for non-structural components in some international building codes, such as the 1967 Uniform Building Code /4.14/. Subsequently, the non-structural seismic design provisions were extended to a wide variety of non-structural components and systems, even if they were focused only on the safety of critical equipment in essential facilities.

Only in the last three decades, several guidelines and standards have developed more accurate seismic design provisions and evaluation procedures for non-structural components, in order to ensure proper performance during earthquakes. The approach of the building codes regarding the non-structural design followed three different paths. A first code category is involved in providing prescriptive requirements for common products, such as suspended ceilings, by means of seismic protection details and specifications. A second code category is based on the assumption that the non-structural components should be designed for lateral seismic forces that are proportional to the element weight. In this regard, the equivalent lateral force method is used for acceleration-sensitive components, so that the anchorages and bracing systems should be able to withstand the earthquake accelerations. The third code category requires that the deformation-sensitive components should be designed to accommodate the design inter-storey drifts of the primary structure.

In this section, the seismic design provisions for non-structural components provided by different international standards are presented and commented. Documents involved in the current study are the European code, namely EN 1998 (Eurocode 8), and the American codes for new buildings ASCE/SEI 7-10 /4.11/ and for existing buildings ASCE/SEI 41-13 /4.7/.

4.5.2 European code

EN 1998-1 defines the design seismic requirements for non-structural components and systems. In particular, the EN 1998-1 specifies the procedures to evaluate the seismic demand on acceleration-sensitive components by means of the equivalent static design force method in section 4.3.5, while it provides the design criteria to define the relative displacement demand on deformation-sensitive components by imposing inter-storey drift limits on the main structural system in section 4.4.3.

Definition of the design seismic forces

According to EN 1998, the non-structural components of normal importance, which may cause risk to human life or affect the main structures or services of critical facilities, must be verified to resist the horizontal equivalent static design force, F_a , applied at the component's centre of mass in the most unfavourable direction and defined as follows:

$$F_a = (S_a \cdot W_a \cdot \gamma_a)/q_a \qquad (4.1)$$

where S_a is the seismic coefficient applicable to the non-structural component (i.e. the design acceleration normalized with respect to the acceleration of gravity); W_a is the component weight; γ_a is the importance factor of the component that ranges between 1.5 (for important or hazardous components for the life safety) and 1.0 (for all other components); and q_a is the behaviour factor for the component that varies between 1.0 and 2.0 for different component typologies. In particular, the upper limit value of the behaviour factor for partitions and anchorage elements of suspended ceilings is set equal to 2.0 by the code.

Specifically, the seismic coefficient may be obtained by:

$$S_a = \alpha \cdot S \cdot \left[\frac{3 \cdot (1 + z/H)}{1 + (1 - T_a/T_1)^2} - 0.5\right] \geq \alpha \cdot S \qquad (4.2)$$

where

- a is the ratio between the peak ground acceleration a_g and the acceleration of gravity g
- S is the soil factor, set equal to 1.0 for rock sites
- H is the total building height
- z is the height of the component's centre of gravity measured from above the foundation level
- T_a is the fundamental vibration period of the non-structural component
- T_1 is the fundamental vibration period of the building in the relevant direction of excitation.

The Eq. (4.2) takes into account the earthquake ground motion, the soil factor, the structural amplification and the flexibility or stiffness of the non-structural component. As regards the dynamic amplification, it asserts that a rigid non-structural component (i.e. with a fundamental period $T_a \approx 0.0$ s) attached to the building roof (z = H) experiences 2.5 times the acceleration of a similar element located at the ground floor (z = 0.0 m). Obviously, a flexible non-structural component is subjected to larger accelerations than a rigid element /4.15/.

Furthermore, EN 1998 states that the influence of non-structural components and their fasteners on the structural behaviour should be taken into account. However, the code does not provide other detailed specifications for considering this influence in the design procedure, except for the case of masonry infill walls in reinforced concrete frames.

In addition, the European code asserts that the seismic analysis of particularly dangerous or important non-structural components should be based on a realistic model of the building structure using appropriate response spectra derived from the response of the supporting structure. It is obvious that the simplified procedures described above are not permitted in these cases, but nevertheless the code does not provide additional specific guidelines to perform such analysis /4.16/.

Definition of the seismic relative displacement demands

The damage limitation requirement for non-structural components, which is specified for seismic design events corresponding to the serviceability limit states, i.e. for frequent low-intensity earthquakes, should be satisfied by limiting the design inter-storey drifts of the main structure to the code-specific values that are defined for different component typologies.

Specifically, EN 1998 requires that the inter-storey drift ratio, defined as the ratio between the design inter-storey drift corrected with a reduction factor ($d_r \cdot v$) and the storey height h, should be limited to:

- 0.5 % for buildings having non-structural components made of brittle materials and attached to the structure
- 0.75 % for buildings having ductile non-structural components
- 1.0 % for buildings having ductile non-structural components fixed in a way so as not to interfere with structural deformations

In particular, the design of inter-storey drift, d_r, is evaluated as the difference of the average lateral displacements at the storey top and bottom, which are obtained by a linear analysis of the structural system based on the design response spectrum (i.e. for a rare seismic event with 475-year return period). The reduction factor, v, takes into account the lower return period of the seismic action associated with the damage limit state, and it ranges between 0.4 and 0.5, depending on the importance class of the building.

However, EN 1998 is not very clear on some aspects. Firstly, the standard classifies the non-structural components as brittle, ductile or isolated elements, but it does not specify clearly the procedure to categorize them and the required minimum ductility capacity for defining a non-structural component as "ductile". Furthermore, the standard does not justify the prescribed drift limits to 1.0 % for non-structural components that should be designed so as not to interfere with structural deformations. These observations reveal an important omission in the current European code requirements.

4.5.3 American code for new buildings

The 1997 Uniform Building Code /4.17/ provided important changes in the design procedure of non-structural components after the initial provisions of the

1992 Tri-Services Manual /4.18/ and 1994 Uniform Building Code /4.19/. In fact, the 1997 UBC introduced the concepts on the amplification of earthquake shaking as a function of building height and the dynamic amplification of force levels experienced by flexible components compared to rigid components /4.8/.

Nowadays, the seismic design requirements for non-structural components in the United States are established in the ASCE/SEI 7-10 /4.11/ that in Chapter 13 defines the "minimum design criteria for non-structural components permanently attached to structures and for their supports and attachments". In particular, ASCE/SEI 7-10 specifies the general design requirements for non-structural components in section 13.2, the procedure to evaluate the design seismic force demand and the seismic relative displacement demand on non-structural components in section 13.3, the requirements for component attachments in section 13.4 and the requirements for architectural components in section 13.5. The listed requirements for non-structural components should be satisfied with specific-designs submitted for approval to the authority having jurisdiction or with components' seismic qualification certificates produced by the manufacturer. In particular, the seismic qualification shall be provided by means of acceptable methods for determining the component seismic capacity, i.e. analysis, testing or experience data obtained through recognized procedures. Furniture, temporary or movable equipment, and some architectural, mechanical and electrical components, depending on their seismic design category, are exempt from the code requirements (see ASCE/SEI 7-10 for more details).

The 1997 UBC and ASCE/SEI 7-10 standards are quite similar in form used for defining the design provisions of the non-structural components, although significant differences must be highlighted. In fact, for expressing the ground shaking intensity, the 1997 UBC adopted seismic coefficients depending on the seismic zone and soil type, while the FEMA 303 /4.20/, later the International Building Code /4.21/ and currently the ASCE/SEI 7-10 use the peak ground accelerations, which are mapped for long and short period structures by the codes.

Definition of the design seismic forces

According to ASCE/SEI 7-10, the horizontal design seismic force F_{ph} to be applied at the component's centre of mass and distributed relative to the component's mass distribution, is defined as follows:

$$F_{ph} = \frac{0.4 \cdot a_p \cdot S_{DS} \cdot W_p}{\left(\frac{R_p}{I_p}\right)} \left[1 + 2 \cdot \frac{z}{h}\right] \tag{4.3}$$

where

- a_p is the component amplification factor that considers the expected dynamic amplification of the peak floor acceleration for flexible components
- S_{DS} is the design spectral response acceleration (i.e. for design earthquake with return period of 475 years), that is calculated for short-period (T = 0.2 s) and 5 % damping structures
- W_p is the component weight
- R_p is the component response modification factor depending on the component typologies (whose definition is similar to the behaviour factor in the European codes) and it ranges between 1.5 and 3.5 for architectural components
- I_p is the component importance factor, which depends on the building occupancy
- z and h are the height of the component's attachment point to the structure and the average height of building roof with respect to the base, respectively.

In Eq. (4.3), the design spectral response acceleration is equal to $S_{DS} = 2/3 \cdot S_{MS}$, in which S_{MS} is the spectral response acceleration at short periods adjusted for the site class effects. Fig. 4.47 shows the design response spectrum according to ASCE/SEI 7-10.

In Eq. (4.3), the 0.4 S_{DS} value represents the design peak ground horizontal acceleration including the site effects, while the product $0.4 \cdot S_{DS} \left[1 + 2 \cdot \frac{z}{h}\right]$ represents the design peak floor horizontal acceleration at the component's attachment point. Since the code considers that the design peak ground horizontal acceleration varies linearly along the building height, a non-structural component attached to the building roof (z = h) experiences an acceleration equal to three times the acceleration at ground level. This last consideration emerged in a study carried out by FEMA 303 about the records of 405 instrumented buildings located in

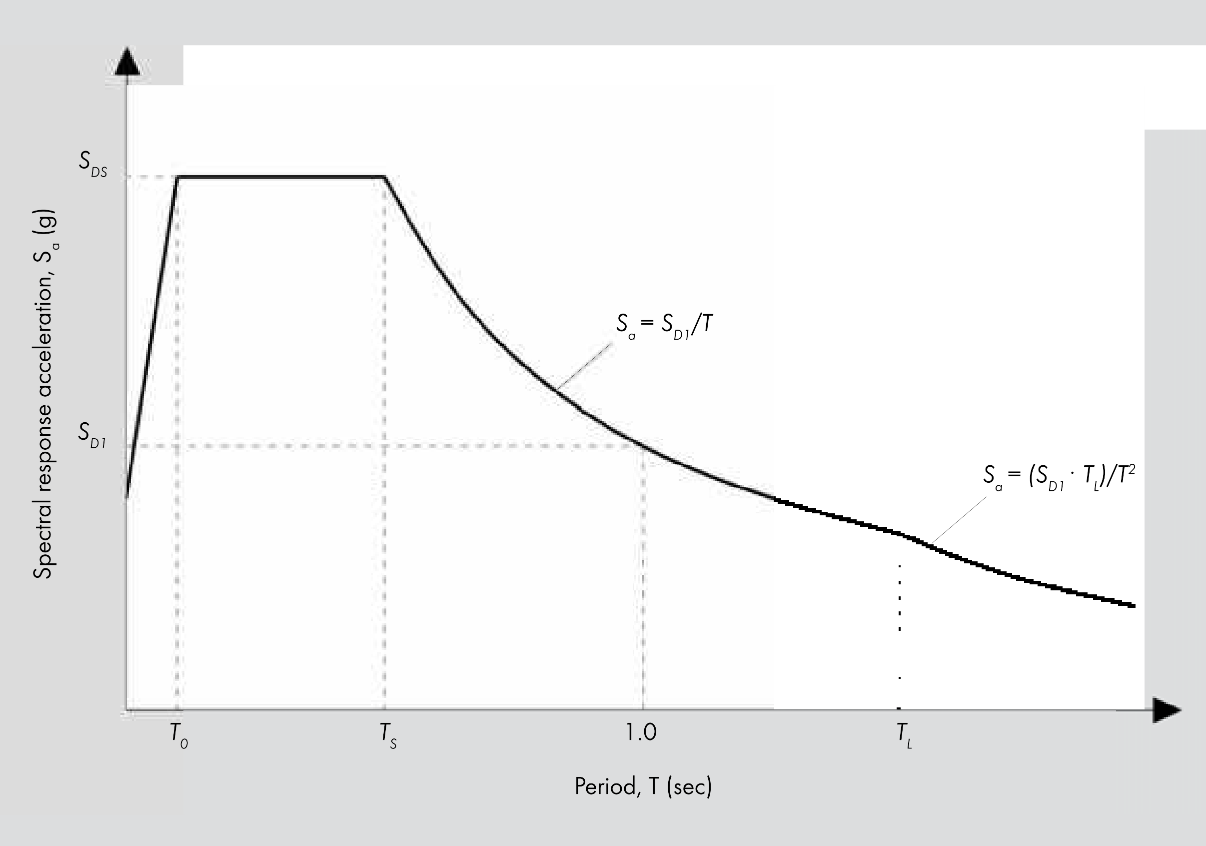


Fig. 4.47: Design response spectrum /ASCE/SEI 7-10 (2010)/

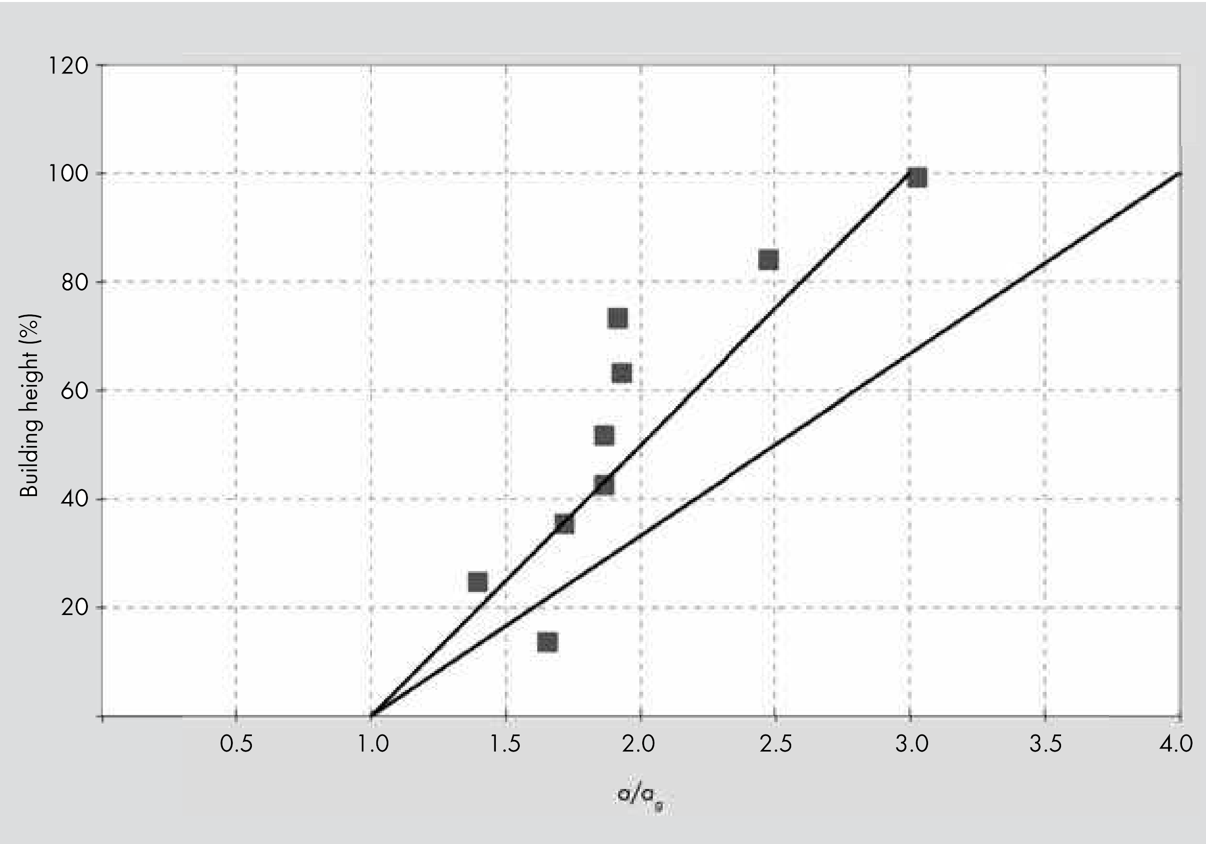


Fig. 4.48: Amplification of the peak ground acceleration versus the building height, with a_g > 0.10 g /BSSC (1997)/

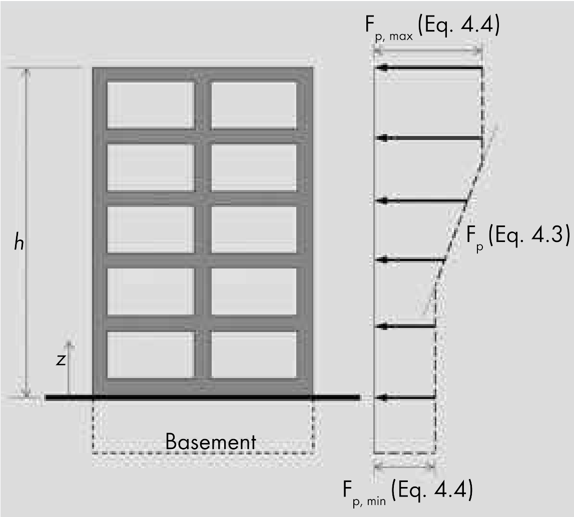


Fig. 4.49: Distribution of the design seismic force as a function of the building height

Californian areas of higher ground shaking intensity. Fig. 4.48, taken from this study, plotted the ratio between the peak floor acceleration a and peak ground acceleration a_g versus the building height expressed as a percentage. The figure shows the amplification of the peak ground acceleration from the ground to roof levels, where the peak roof acceleration is about three times the peak ground acceleration ($a \approx 3 \cdot a_g$).

In addition, according to ASCE/SEI 7-10, the design seismic force F_{ph} must satisfy the following lower and upper limits:

$$0.3 \cdot S_{DS} \cdot I_p \cdot W_p \leq F_p \leq 1.6 \cdot S_{DS} \cdot I_p \cdot W_p \qquad (4.4)$$

The minimum and maximum values of the design seismic force are obtained by considering a rigid non-structural component (with fundamental period less than 0.06 s) and a flexible non-structural component (with fundamental period greater than 0.06 s), respectively. In particular, the maximum value is set in a way to avoid an unreasonably high design force due to the components' non-linear response.

Therefore, in compliance with the above comments, the Eq. (4.3) represents a trapezoidal distribution of floor accelerations within the structure, linearly increasing from the acceleration at the ground to the acceleration at the roof (Fig. 4.49).

The component amplification factor a_p represents the dynamic amplification of the non-structural component response as a function of the fundamental periods of the structure T_s and component T_p . The dynamic amplification is caused by the resonance between the non-structural and structural responses and it occurs if the component's period closely matches that of any vibration mode of the supporting structure. In general, the dynamic amplification of non-structural components is highly affected by the primary vibration mode for short-period structures (i.e. most ordinary buildings), otherwise it is influenced by higher vibration modes in the case of long-period structures (i.e. tall buildings). In order to define the component amplification factor, FEMA P-750 /4.22/ provided a formulation of a_p as a function of the ratio between the structural and component periods T_p/T_s (Fig. 4.50). In particular, for Eq. (4.3), the component amplification factor ranges between 1.0, for rigid components with a fundamental period less than 0.06 s (for which dynamic amplification is not expected), and 2.5, for flexible components with periods greater than 0.06 s (Tab. 4.11).

The component response modification factor R_p reduces the horizontal design seismic force acting on non-structural components. This reduction factor represents the energy absorption capability of the non-structural components and attachments depending on their overstrength and ductility. Since the non-structural components have generally lower ductility and overstrength than the structural systems, the response modification factor is usually smaller than the reduction factor used for structural systems.

Tab. 4.11 shows the upper limit values of the component amplification factor and the response modification factor to be adopted for some architectural components according to ASCE/SEI 7-10. In particular, for interior non-structural partitions and all ceiling types, the component amplification factor a_p and response modification factor R_p are set equal to 1.0 and 2.5, respectively. Low, limited and highly deformable components present the assigned reduction factor equal to 1.5, 2.5 and 3.5, respectively.

The component importance factor I_p ranges between 1.0 and 1.5. In particular, it shall be taken as 1.5 if the life-safety function is required to the non-structural component after an earthquake, if the component contains hazardous materials or if the continued operation is required to the component. For all other cases, the importance factor is set

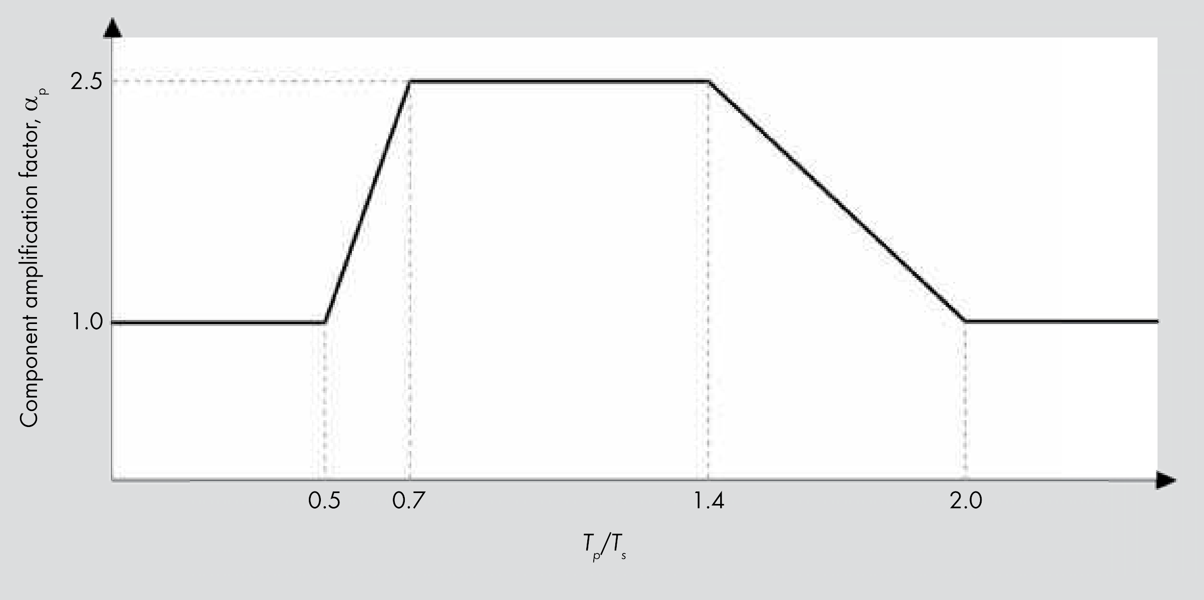


Fig. 4.50: Formulation of component amplification factor as a function of the structural and component periods /FEMA P-750 (2009)/

Tab. 4.11: Component amplification factor and response modification factor for some architectural components /ASCE/SEI 7-10 (2010)/

Architectural component	**a_p**	**R_p**
Interior non-structural walls and partitions		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever elements (unbraced or braced to structural frame below its centre of mass)		
Parapets and cantilever interior non-structural walls	2.5	2.5
Ceilings		
All	1.0	2.5
Other rigid components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Other flexible components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability elements and attachments	2.5	1.5

equal to 1.0. The horizontal equivalent static design force computed by Eq. (4.3) is used for designing the anchorage and bracing systems of the non-structural components when the importance factor I_p = 1.0, otherwise, when I_p = 1.5, the non-structural components themselves are designed for the obtained design seismic force.

The horizontal equivalent static design force F_{ph} shall be applied independently in at least two orthogonal horizontal directions in combination with vertical design force and service loads. For vertical cantilevered systems, e.g. partial-height partitions, the design seismic force shall be acting in any horizontal direction.

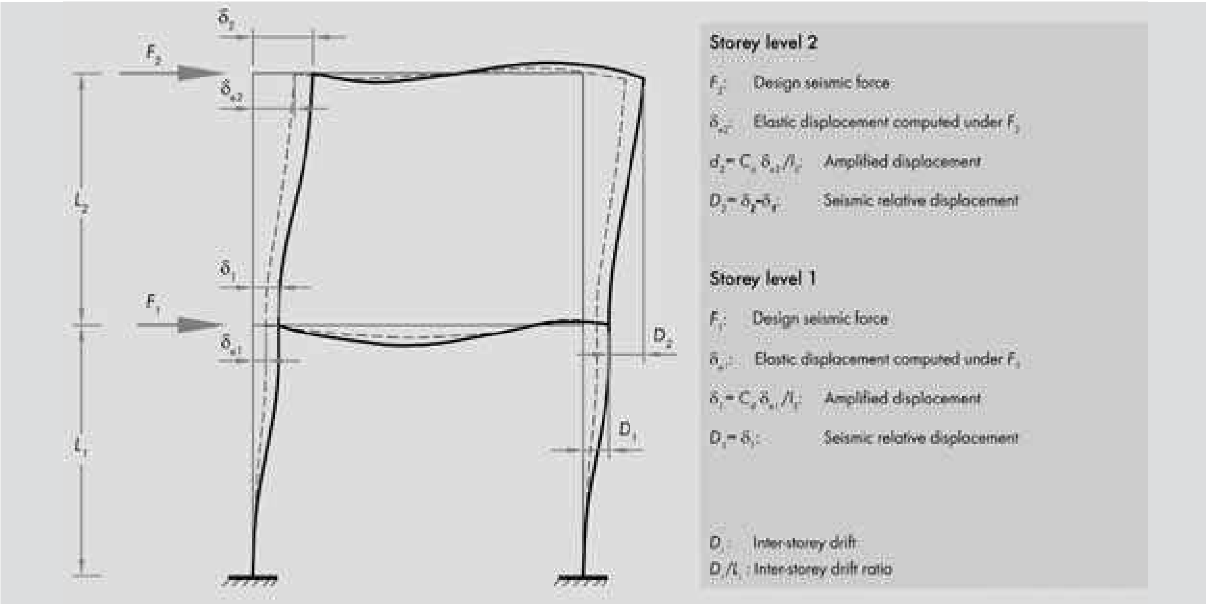


Fig. 4.51: Definition of the seismic relative displacement for non-structural components attached on the same structure A or the same structural system

In addition, according to ASCE/SEI 7-10, a vertical equivalent static design force F_{pv} must be considered for designing the non-structural components, except for suspended lay-in tile ceilings, and it is given by:

$$F_{pv} = \pm\, 0.2 \cdot S_{DS} \cdot W_p \tag{4.5}$$

Anyway, ASCE/SEI 7-10 allows determination of the horizontal and vertical design seismic forces by dynamic analysis that considers the interaction between structural and non-structural components instead of using procedures shown in Eqs. (4.3) and (4.5).

Definition of the seismic relative displacement demands

The ASCE/SEI 7-10 standard recommends that the deformation-sensitive components, susceptible to structural deformation, should be designed to satisfy the seismic relative displacement demand. The seismic relative displacement demand D_{pl} between the non-structural components and structural systems should be determined with an analysis of both structure and components attached to it and in combination with displacements caused by other loads. The displacement demand on non-structural components is defined as follows:

$$D_{pl} = D_p \cdot I_e \tag{4.6}$$

where D_p is the seismic relative displacement of the non-structural component relative to the structural system and it is defined for components attached on the same structural system or attached on separate structural systems; I_e is the seismic importance factor of the building, which depends on the risk category of the building under consideration, and it ranges between 1.0 and 1.5.

In the first case, for non-structural components (e.g. in the case of partitions or glazing systems) attached on the same structure A or the same structural system with two connection points at different heights, the seismic relative displacements should be defined as (Fig. 4.51):

$$D_p = \delta_{xA} - \delta_{yA} \tag{4.7}$$

where δ_{xA} and δ_{yA} are the amplified structural displacements at building level x and level y of the structure A, respectively.

Generally, the amplified structural displacements δ_j at building level j are computed as the displacements δ_{ej} obtained from a linear elastic analysis of the structure under design seismic forces multiplied by a deflection amplification factor C_d that takes into account the inelastic response of the structure and depends on the seismic force-resisting system types. The amplified structural displacements are defined as follows:

$$\delta_j = \frac{C_d \cdot \delta_{ej}}{I_e} \tag{4.8}$$

In the second case, for non-structural components attached on separate structures A and B or separate

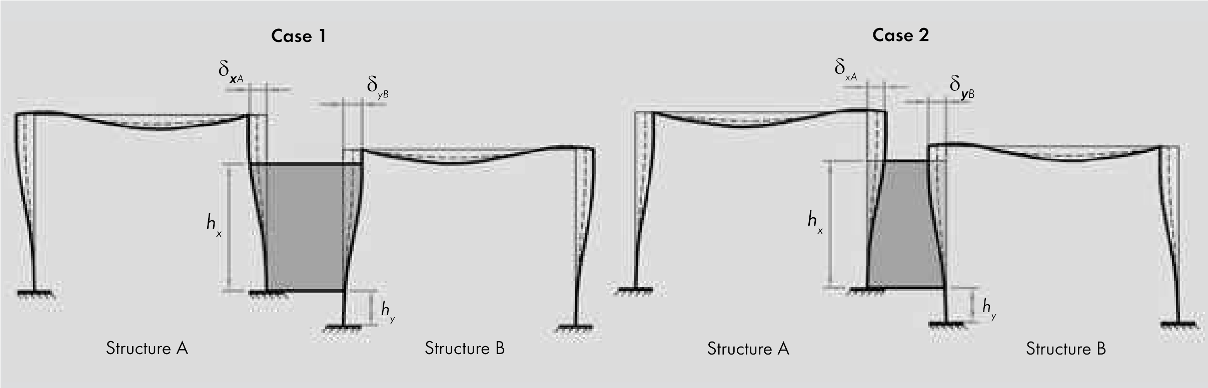


Fig. 4.52: Definition of the relative seismic displacement for non-structural components attached on separate structures A and B or separate structural systems

structural systems with two connection points at different heights, the seismic relative displacement should be defined as the sum of the absolute amplified structural displacements (Fig. 4.52):

$$D_p = |\delta_{xA}| + |\delta_{yB}| \qquad (4.9)$$

where δ_{xA} and δ_{xB} are the amplified displacements at level x of the structure A and level y of the structure B, respectively, defined according to Eq. (4.8).

However, if the amplified structural displacements obtained by linear elastic analysis are unknown, the ASCE/SEI 7-10 standard provides the maximum allowable design inter-storey drift for several structures. In this case, the Eqs. (4.7) and (4.9) used to define the seismic relative displacement D_p are replaced by the following Eqs. (4.10) and (4.11), respectively. In particular, the seismic relative displacements for components attached on the same structure A or the same structural system are defined as follows:

$$D_p = (h_x - h_y) \cdot \frac{\Delta_{aA}}{h_{sx}} \qquad (4.10)$$

Furthermore, the seismic relative displacements for components attached on separate structures A and B or separate structural systems, are defined as follows:

$$D_p = h_x \cdot \frac{\Delta_{aA}}{h_{sx}} + h_y \cdot \frac{\Delta_{aB}}{h_{sx}} \qquad (4.11)$$

In these formulations, h_x and h_y are the heights of level x and level y of the building, to which the upper and lower connection points of non-structural components are attached, respectively; Δ_{aA} and Δ_{aB} are the allowable design inter-storey drift for the structure A and B; h_{sx} is the storey height used in the definition of the allowable inter-storey drift.

In particular, Δ_a/h_{sx} is defined as the inter-storey drift ratio, and it ranges between 0.7 % and 2.5 % for several structure typologies (defined in structures of four or less storeys with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the storey drifts; masonry shear wall structures and all other structure types) depending on their risk category.

Therefore, the deformation-sensitive components should be designed in order that the relative anchor movements are equal to the seismic relative displacements defined according to the above procedure.

4.5.4 American code for existing buildings

The ASCE/SEI 41-13 standard /4.7/ sets in chapter 13 the seismic retrofit requirements for architectural, mechanical and electrical components and systems that are permanently installed in or located within existing buildings. The seismic requirements are provided for existing non-structural components that are retrofit to the position retention, operational and life safety non-structural performance levels (see section 4.4.2 for more details about the performance objectives). The new components installed in existing buildings should conform to the requirements listed in this code, however, the use of the ASCE/SEI 7-10 standard is allowed in this case. In particular, the requirements for operational non-structural performance level should be consistent with ASCE/SEI 7-10 chapter 13 requirements.

The ASCE/SEI 41-13 specifies in section 13.4 the analytical procedure for evaluating the design seismic forces and seismic relative displacement demands on

Tab. 4.12: Applicability of life safety and position retention requirements according to seismicity levels and identification of the evaluation procedures for architectural components /ASCE 41-13 (2013)/

Architectural component type	**Seismicity**						
	High seismicity		Moderate seismicity		Low seismicity		**Evaluation procedure**
	PR	LS	PR	LS	PR	LS	
Partitions							
Light	Yes	No	Yes	No	No	No	F/D
Ceilings							
Suspended gypsum board ceilings	Yes	Yes	No	No	No	No	F
Suspended acoustic lay-in tile ceilings	Yes	No	Yes	No	No	No	P

PR: Position Retention non-structural performance level; LS: Life Safety non-structural performance level
F: Analytical procedure with Force analysis should be performed; P: Prescriptive procedure should be permitted
F/D: Analytical procedure with Force and Deformation analysis should be performed.

non-structural components and the prescriptive procedure with the indication of general requirements. In particular, the non-structural components should be rehabilitated using the evaluation procedures specified by ASCE/SEI 41-13 and illustrated in Tab. 4.12. The table identifies the evaluation procedures (i.e. analytical and prescriptive) for different architectural component types that are selected based on the life safety and position retention retrofit requirements for several seismicity levels (high, moderate and low). The requirements for the operational non-structural performance level are not included in the code. In Tab. 4.12, "Yes" indicates that the non-structural retrofit is required by the code for the considered performance levels. According to Tab. 4.12, the analytical procedure, which allows performing force analysis and deformation analysis or a combination between them, is applicable to lightweight gypsum board partitions and suspended gypsum board ceilings. The prescriptive procedure consists of published design concepts and construction features that the non-structural components should have in order to be seismically protected, such as in the case of suspended acoustic lay-in tile ceilings. Alternatively, the computation of the seismic forces and relative displacements should be performed by test methodologies based on recognized procedures.

Definition of the design seismic forces

According to ASCE/SEI 41-13, the force analysis consists of general equations for determining the horizontal and vertical seismic forces, F_{ph} and F_{pv}, acting on non-structural components. The provided formulations are quite similar to Eqs. (4.3) and (4.5) of the ASCE/SEI 7-10 Standard (see section 4.5.3).

Lightweight steel gypsum board partitions and suspended gypsum board ceilings should be capable of resisting the forces computed by using a component importance factor I_p equal to 1.0 or 1.5, for position retention or operational non-structural performance levels, respectively.

Suspended acoustic lay-in tile ceilings should be retrofitted by the prescriptive procedures listed in ASCE/SEI 7-10 Standard (see section 4.4.3 for more details).

Furthermore, the code allows the use of linear dynamic analysis of the building for determining the actual storey accelerations based on different vibration modes of structure, taking into account the range of vibration periods of the non-structural components under consideration. Linear dynamic analysis procedures are considered sufficiently accurate for estimating the storey accelerations, and then the seismic forces, for life safety and position retention performance levels. Otherwise, non-linear dynamic analysis may be preferred for the operational performance level, since the prediction of floor accelerations should be more accurate.

Definition of the seismic relative displacement demands

The deformation analysis according to ASCE/SEI 41-13 allows computing the drift ratios and seismic relative displacements between the non-structural components and structural systems. The provided general equations are quite similar to Eqs. (4.7) and (4.9) of the ASCE/SEI 7-10 standard (see section 4.5.3).

According to the code, the drift ratios of lightweight steel gypsum board partitions should be limited to 2 % or 1 %, for position retention or operational non-structural performance levels, respectively.

4.6 Ongoing research activities

4.6.1 State of the art of research

As described in detail in the previous sections, recent earthquakes caused significant damage to non-structural components highlighting the low fragility and the importance of these constructive systems within the building, both for the safety of the occupants and for economic safeguarding. In fact, the damage of non-structural components can severely limit the building functionality, with consequent economic losses related to the business interruption and the damage repair and even more limit the funcional integrity of critical infrastructure being of particular importance in case of desasters. In addition, it is important to note that the non-structural elements represent a large portion (more than 80 %) of the total economic investment in typical buildings.

The importance of a rational concept of non-structural elements has also been recognized in the development of modern seismic regulations through the introduction of specific design requirements in terms of strength and deformation for these elements, as described in section 4.5. Nevertheless, the knowledge of the seismic performance of non-structural elements is still poorly understood.

Among the non-structural components, the prediction of the seismic response of lightweight steel drywall systems is a rather complex issue and cannot be easily solved by traditional methods. These systems exist in partitions or exterior walls and ceilings, which are usually composed of a frame made of cold-formed metal profiles having a low thickness (0.6 – 1.0 mm) and one or more cladding layers, generally made of gypsum boards. The material and number of layers depend on the required performance. The boards are attached to the metal frame with specific screws or nails. It is clear that the mechanical response of such types of systems is strongly influenced by the different material used and by the interaction between the board and the metal frame through the connections. Therefore, the main way to accurately assess the seismic response of these systems involves the execution of specific experimental campaigns.

In the perspective of adressing the design shortcomings regarding the non-structural drywall systems, a large number of research studies have been undertaken over the last years for investigating their seismic behaviour by means of "component-level" tests on lightweight metal stud partitions and suspended ceilings tested in isolated configuration. In particular, the seismic response of drywall partitions were evaluated under quasi-static (Lee et al. /4.23/, Restrepo and Bersofsky /4.24/, Tasligedik et al. /4.25/, Araya-Letelier and Miranda /4.26/) and dynamic loading conditions (Retamales et al. /4.27/, Magliulo at al. /4.28/). Furthermore, studies on the seismic behaviour of suspended ceilings were carried out by means of dynamic tests on shaking tables (Badillo-Almaraz et al. /4.29/, Gilani et al. /4.30/, Magliulo et al. /4.31/, Ryu et al. /4.32/, Soroushian et al. /4.33/). The "component-level" tests are usually preliminary to the "system-level" tests, which are equally essential for understanding the interaction between these components and the primary structural system and/or other non-structural components. Shaking table tests on full-scale single or multi-storey buildings were carried out on systems composed of drywall partitions and suspended ceilings (Filiatrault et al. /4.34/, McCormick et al. /4.35/, Matsuoka et al. /4.36/, Wang et al. /4.37/).

In general, the research objectives of the above cited studies were to provide information about the seismic performance of drywall partitions and suspended ceilings by investigating the following aspects: (i) the damageability and fragility using damage limit states,

(ii) the cyclic and dynamic behavior, (iii) the effects of construction details, (iv) the estimation of repair costs, (v) the interaction with the supporting structural system or other non-structural components.

A precursor of the evaluation of the non-structural seismic damageability and fragility using damage limit-state definitions was Rihal /4.38/ who investigated the damageability of drywall partitions using various damage thresholds. According to the research results, the physical damage progression is observed inspecting the specimen components and thus it is associated to the damage limit states that are distinguished according to the required repair level into: Damage state (I), characterized by minor damage of the gypsum boards, which could be easily repaired by patching, re-taping, sanding and painting; damage state (II), characterized by severe damage of the gypsum boards, which requires their replacement; damage state (III), characterized by severe damage of the metal frame and board-to-frame connections, which requires the replacement of whole or sections of non-structural systems. The recorded damage limit states are usually correlated to the structural response parameters (i.e. measured inter-storey drift levels) in the case of deformation-sensitive components or to the ground motion intensity in the case of acceleration-sensitive components. In particular, the study carried out by Restrepo and Bersofsky /4.24/ highlighted that lightweight steel drywall partitions developed the damage state (I) at inter-storey drift ratios ranging between 0.05 % and 1 %, the damage state (II) at inter-storey drift ratios ranging between 0.5 % and 1.5 %, and the damage state (III) at inter-storey drift ratios ranging between 0.5 % and 3 %. With the aim of providing a comprehensive experiment-based tool for damage assessment within the performance-based design framework /4.27/, the experimental results are presented in several studies in form of seismic fragility curves, which are defined as the conditional probability of reaching or exceeding a damage limit state at a specified level of inter-storey drift or ground motion intensity. One of the purposes of fragility analysis is to define the seismic vulnerability of non-structural components, by identifying the regions of undesirable and unsafe performance /4.29/. For example, based on the fragility curves developed using the experimental data, Ryu et al. /4.32/ pointed out that the ceiling system becomes more vulnerable using heavier tiles, incrementing the ceiling area, removing the lateral restraints or subjecting it to multi-directional input motions.

The cyclic and dynamic behaviour of drywall partitions was investigated in terms of load-displacement curves. The results showed that the response is characterized by non-linear hysteretic loops with significant pinching and stiffness and strength degradation /4.23, 4.24, 4.25/. Furthermore, the hysteretic behaviour allows moderate energy dissipation, particularly before reaching the peak lateral force /4.24/. A more ductile behavior characterize these partition types, also when compared to timber framed drywall partitions, as demonstrated by Tasligedik et al. /4.25/. As regard the comparison between loading conditions, the partition damage is not amplified by dynamic loading comparing to that observed in quasi-static tests, as pointed out by Lee et al. /4.23/. Concerning the dynamic behavior of suspended ceilings, Magliulo et al. /4.31/ demonstrated that these systems could be classified as rigid non-structural components having a fundamental vibration period ranging between 0.03 s and 0.06 s in the horizontal direction.

Since in several studies the drywall partitions and suspended ceilings realized with the same construction techniques and materials showed significant differences in their seismic performance and failure mechanisms /4.27/, the effect of construction details was recognized as an important key issue. Regarding this aspect, different variables were considered in the above mentioned experimental campaigns, such as frame thickness, connection type, wallboard thickness, screw spacing, the effects of wall discontinuities (i.e. openings, door frames and partial-height walls), and intersection details between walls and/or ceiling or supporting structure /4.23, 4.24, 4.27, 4.35/. In particular, concerning the wall behaviour, these studies highlighted that the damage was concentrated around the openings and door frames /4.35/, at the wall intersecting corners /4.27/ and at the contact perimeter between partitions and/or ceilings or supporting structure /4.23/. For these reasons, some experimental research studies were devoted to develop seismic mitigation details for drywall systems in order to prevent or reduce their damage in future practice /4.26, 4.27/. Comparing the behaviour of conventional partitions with the performance of seismically designed partitions, some studies demonstrated the effectiveness of using a gap between the drywall partitions and the structural supporting structure for reducing the wall damage /4.27, 4.28, 4.36/. Another example of seismic mitigation detail was the sliding/frictional connection, proposed by Araya-Letelier and Miranda /4.26/, which isolates the drywall partitions from the structural lateral deformations by increasing the drift demands at which damage occurs.

The estimation of repair costs associated to damage is another important issue for the performance-based design. The repair costs of drywall systems, which are defined as the costs to replace the damaged part, corresponding to specific inter-storey drifts were estimated in several studies /4.23, 4.26/. In particular, Lee et al. /4.23/ demonstrated that the repair of drywall partitions is not required up to drift levels of 0.25 %. At drift levels of 2 %, the repair costs of drywall partitions equal the initial costs, while at drift levels of 8 % they are twice the initial costs. This observation confirms the importance of loss estimation of non-structural systems placed within a building when subjected to seismic actions.

The interaction between drywall systems, i.e. partitions and suspended ceilings, and the supporting structural system or other non-structural components was evaluated by means of both cyclic and dynamic tests. In particular, Lee et al. /4.23/, Tasligedik et al. /4.25/ and McCormick et al. /4.35/ demonstrated that the strength offered by the drywall systems is not negligible with respect to the structural strength and the added damping and stiffness may contribute to the performance of the overall structure. Test frames properly designed for simulating the seismic effects at a generic building storey and specimens subjected to increasing levels of shaking for investigating a wide range of inter-storey drift demand and seismic damage were considered in several studies /4.34, 4.35, 4.36, 4.37/. Among these researches, the experimental study carried out by Wang et al. /4.37/ involved shaking table tests on full-scale five-storey buildings that were isolated or fixed at the base and completed with non-structural components and systems. The obtained results showed that the base isolation is effective for minimizing the seismic damage of drywall systems by significantly reducing the storey drift in the building.

The main research outcomes and recommendations about the characterization of drywall systems by testing, briefly summarized in this section, have been considered for the planning and the execution of a very comprehensive experimental study presented in the following section.

4.6.2 A current experimental campaign

In order to overcome the lack of information about the behaviour and design of lightweight steel drywall systems under seismic actions, an important collaboration between the Knauf Group and University of Naples "Federico II" commenced over the last few years. In particular, this collaboration has led to the planning of detailed research involving an extended experimental campaign, which is currently ongoing at the Department of Structures for Engineering and Architecture. The main objective of the research is to investigate the seismic performance of non-structural lightweight steel drywall systems provided by Knauf. The experimental campaign is principally focused on the seismic behaviour of lightweight steel gypsum board partitions and on their interaction with other non-structural components, i.e. exterior walls and suspended continuous gypsum board ceilings and structural elements. The research results will allow identification of the best solutions for optimizing the seismic performance of the investigated systems, also considering the design requirements provided in the modern seismic code for non-structural elements, i.e. EN 1998-1.

The experimental activity planned for the research foresees three different levels of tests: Subsystems, single drywalls partitions and components.

The global seismic response of subsystems composed of more drywall partitions (interior and exterior) together with suspended ceilings will be investigated through dynamic tests on shaking tables. The global response of the single drywall partitions will be evaluated by two

Tab. 4.13: Board typologies for the bending tests

Panel typology	**Thickness (mm)**
Gypsum board	12.5
Gypsum fibre board	12.5
Impact resistant special gypsum board	12.5
Impact resistant special gypsum board	15.0
Cement-based board	12.5

types of quasi-static test: In-plane reversed cyclic and out-of-plane monotonic tests. In addition, tests for the out-of-plane dynamic identification of drywall partitions are planned. Both subsystems and single drywall tests are also conceived for investigating the interaction at the interface between the drywall partition and the building structure. In fact, the set-up structures are designed to allow the interposing of concrete bricks between the specimen and the testing structure, in such a way as to give the possibility to simulate the interface of a reinforced concrete building structure. Finally, since the response of drywall systems is strongly influenced by the local response of the different materials composing these systems and by their interaction, the experimental campaign is completed by a large number of tests on material, products and components. In particular, tests on steel, cladding boards, self-drilling screws and board-to-frame connections are planned. The whole experimental campaign involves a total of 174 tests.

Local behaviour: Tests on material and components

The local response of the lightweight drywall systems is investigated by experimental tests on the materials and components. Therefore, all the materials and products used for the systems selected for full-scale tests are tested. In particular, tensile coupon tests on different steel materials adopted for profiles, bending tests on used cladding panel types, shear tests on self-drilling screws are performed. In addition, since the fundamental influence of the interaction between panels and steel frame, shear tests on panel-to-frame connections considering all the combination between type of panel, screw and steel thickness are considered for tests.

As far as the steel material is concerned, conventional tensile coupon tests carried out according to ISO 6892-1 are conducted on 0.6 and 0.8 mm thick DX51D+Z steel grade, which is typically used for profiles of non-structural systems.

Bending tests on all the cladding panel typologies used in full-scale subsystem tests are performed with the aim of defining their mechanical properties, such as modulus of elasticity and bending strength. The tested panel typologies are shown in Tab. 4.13. The tests are carried out according to the basic requirements of EN 520 in normal environmental conditions. The test consists of a three point bending test on panel specimens sampled in both the transverse and in the longitudinal directions (Fig. 4.53). The test results showed that, in case of gypsum based boards, the strength of the panels oriented in longitudinal direction is higher than the transverse direction (Fig. 4.54). On the contrary, the cement-based and gypsum fibre board typologies show similar strength values in both orientations.

Tests for the evaluation of the shear strength of drywall screws are carried out on the screws adopted for panel-to-frame connections of the investigated drywall subsystems and also on the screws used in the connection between exterior walls and partitions. The main information on tested screw typologies are summarised in Tab. 4.14. Shear tests on screws are carried out by using an ad hoc experimental test set-up, proposed by Fiorino et al. /4.39/ (Fig. 4.55a). Test results in terms of typical load-displacement curve are depicted in Fig. 4.55b.

The connection between cladding boards and the steel frame has a fundamental role in the global response of the drywall systems. In particular, the shear response of panel-to-frame connections influences the in-plane and out-of-plane behaviour of the wall. Therefore, in order to investigate this important issue, all the configurations used

Fig. 4.53: Bending tests on cladding panels

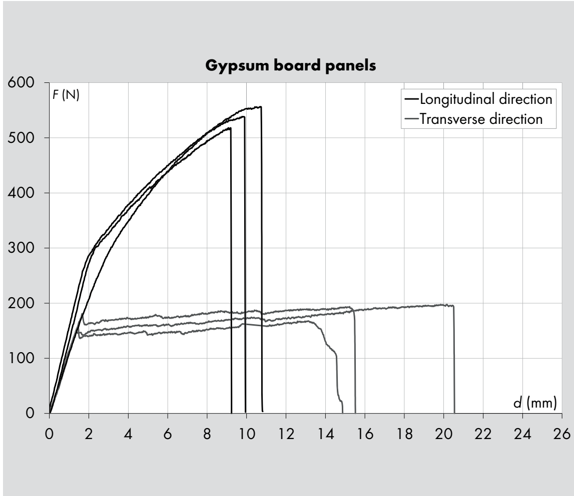
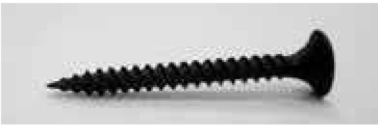
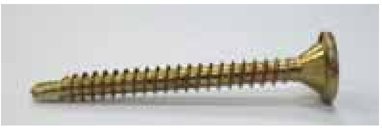
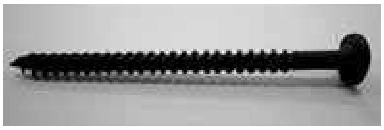
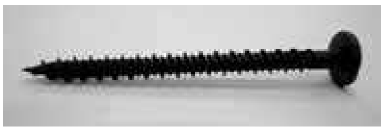
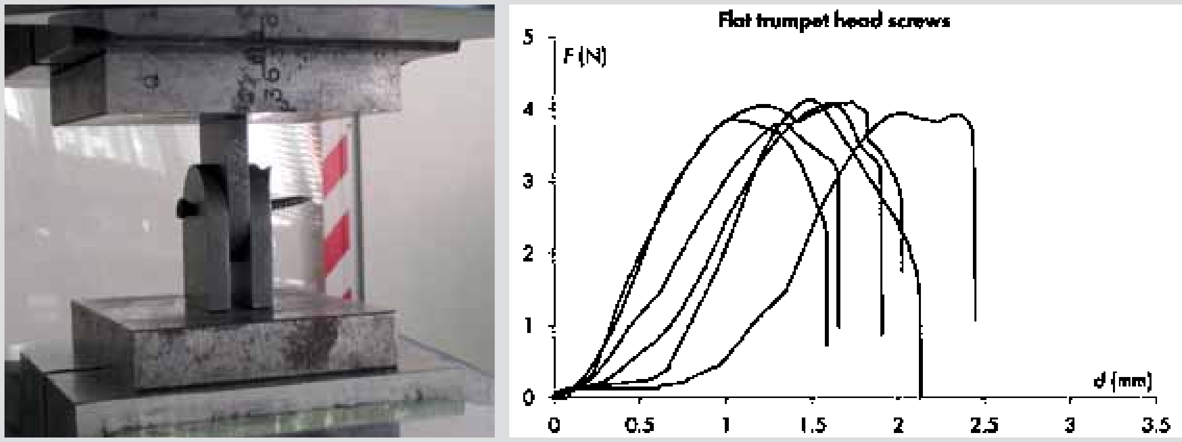


Fig. 4.54: Typical response of gypsum based boards

Tab. 4.14: Screw typologies for the shear tests

Screws	Head type	Diameter (mm)	Length (mm)
	Flat trumpet head	3.5	35
	Trumpet head with milling ribs	3.9	45
	Flat trumpet head	3.9	38
	Screw head plate with milling ribs	4.2	39
	Screw head with surrounding underneath edge	3.9	55
	Lath head	4.3	65

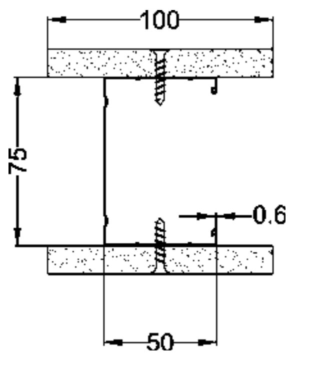
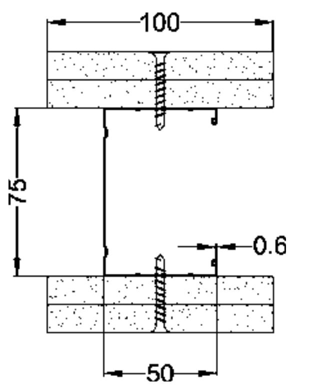
in full-scale subsystem are considered through different combinations of type and number of panels (single or double), screws and steel profile thickness. The assumed edge distance is equal to 15 mm, which is the typical distance used in drywalls. The tested configurations are described in Tab. 4.15. The tests are carried out by using a test set-up proposed by EN 520 for panel-to-wood frame connections, adapted for lightweight steel profiles (Fig. 4.56a). The experimental results show that the prevalent failure mode is the breaking of the panel edge with the tilting of the screws (Fig. 4.56b). Typical experimental force-displacement curves are shown in Fig. 4.57.



a) *Set-up* b) *Typical response of flat trumpet head screws*

Fig. 4.55: Shear tests on self-drilling screws

Tab. 4.15: Tested panel-to-frame connection typologies

	Panels	**Panel thickness (mm)**	**Stud thickness (mm)**	**Screw**	**Screw diameter (mm)**
	Gypsum board	12.5	0.6	Flat trumpet head	3.5
	Gypsum fibre board	12.5	0.6	Trumpet head with milling ribs	3.9
	Impact resistant special gypsum board	12.5	0.6	Flat trumpet head	3.9
	Impact resistant special gypsum board	12.5	0.8	Screw head with surrounding underneath edge	3.9
	Cement-based board	12.5	0.8	Screw head plate with milling ribs	4.2
	2x gypsum board	12.5	0.6	Flat trumpet head	3.5
	2x gypsum fibre board	12.5	0.6	Trumpet head with milling ribs	3.9
	Impact resistant special gypsum board + gypsum board	12.5	0.6	Flat trumpet head	3.9

Out-of-plane tests on lightweight partitions

In order to provide an answer to the requirements of EN 1998 in terms of verification of non-structural elements, out-of-plane tests on full-scale drywall partitions are programmed in the experimental campaign. As described in detail in section 4.5, EN 1998 requires the verification of the non-structural element under a horizontal force applied to its centre of mass. Therefore, the partition walls can be schematized as a simply supported beam subjected to concentrated force at mid-span. This horizontal force depends on the height position of the non-structural element with respect to the foundation level

a) Set-up b) Typical failure

Fig. 4.56: Test on panel-to-frame connections

and the ratio between the fundamental vibration periods of the whole building and of the non-structural element. In conclusion, for this verification, two parameters of the drywall partition are necessary: Bending resistance and fundamental vibration period.

In order to evaluate the bending resistance, quasi-static monotonic tests are carried out on drywall partitions. In addition, dynamic tests, namely step-relaxation tests, are performed for the evaluation of the fundamental vibration period and damping ratio of the systems.

The tested systems are drywall partitions having a width equal to 1800 mm and sheathed with two gypsum boards for each side. The parameters under investigation are the wall height (600 or 2700 mm), the stud spacing (300 or 600 mm) and the joint between drywall partitions and reinforced concrete building structure. This joint can be fixed or sliding and, in both cases, the fasteners used for the connection between track profiles and the surrounding structure can be plastic or steel dowels placed at different spacing (600 or 900 mm). The sliding joint (deflection head) detail is depicted in Fig. 4.58. Different values of the gap distance *a* (20 or 30 mm) between the panel and structure are the subject of investigation.

These tests consist in three line bending tests performed by using a specifically designed set-up structure (Fig. 4.59). The edge restraints of the set-up are composed of two reinforced concrete blocks and the load at mid-span is applied by a system of steel beams. This system allows to perform both quasi-static and dynamic tests by means of electromagnetic restraints.

The results of quasi-static monotonic tests show that the response of the partition, in particular its strength, is influenced by the studs (Fig. 4.60). In fact, the local buckling of the stud represents the main failure mode, which corresponds to the curve peak (Fig. 4.61).

The preliminary elaborations of the step-relaxation test results showed that the fundamental vibration period and the damping ratio are very similar for the different configurations tested and, therefore, they are not influenced by the joint type and the stud spacing (Fig. 4.62).

In-plane tests on lightweight partitions

In order to provide an answer to the requirement of the

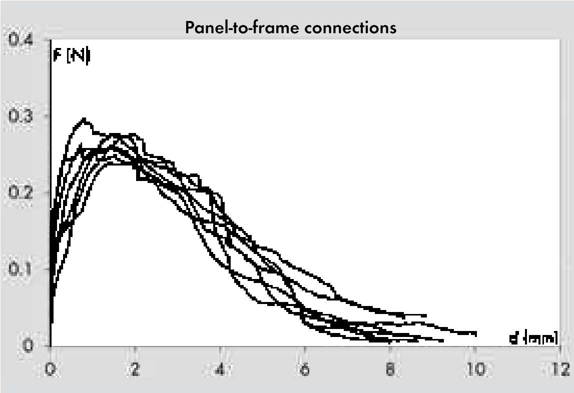


Fig. 4.57: Typical result of tests on panel-to-frame connections

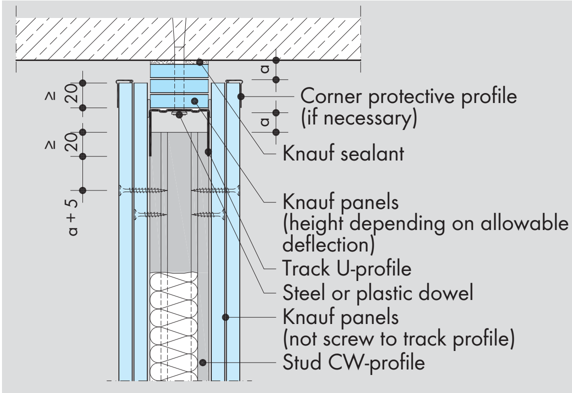


Fig. 4.58: Sliding joint (deflection head) detail

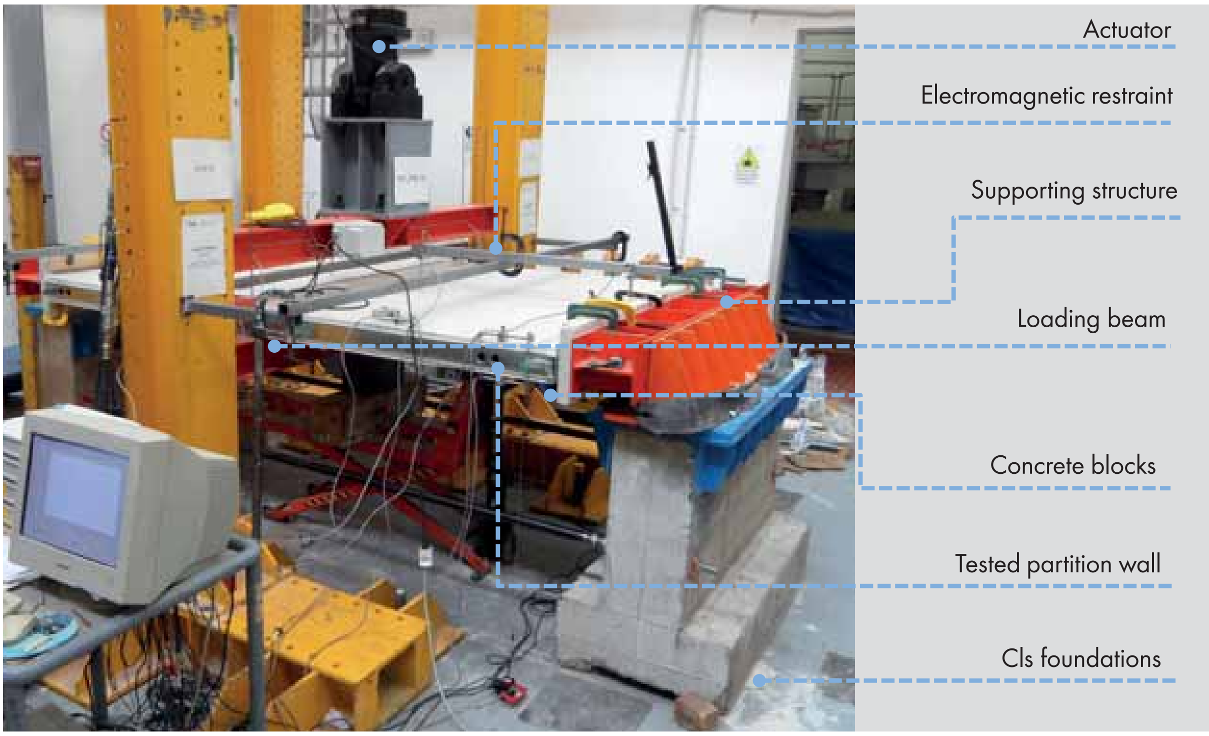


Fig. 4.59: Test set-up for out-of-plane monotonic tests

current seismic code about the admissible inter-storey drift limits for lightweight drywall partitions, their seismic response and their damage under lateral actions are investigated by means of quasi-static reversed cyclic tests on full-scale specimens subjected to horizontal in-plane loads. Two specimen typologies are considered: Single drywall partitions and subsystems made by a drywall partition connected at the lateral edge to transverse drywalling exterior walls (Fig. 4.63). The variables under investigation are the stud spacing, panel typology and joint type between partition drywall and the reinforced concrete building structure. This joint can be fixed or sliding, and three possible conditions are investigated: All fixed joint, sliding joint only on the wall top and sliding joint on top and lateral edges.

A specifically designed 2D testing hinged steel frame, without lateral resisting elements, will be adopted for this experimental activity. During the test, the damage of the different drywall components will be accurately assessed and recorded at different drift stages.

Shaking table tests on full-scale systems

In order to assess the response under seismic excitations of the lightweight subsystems, dynamic tests on shaking

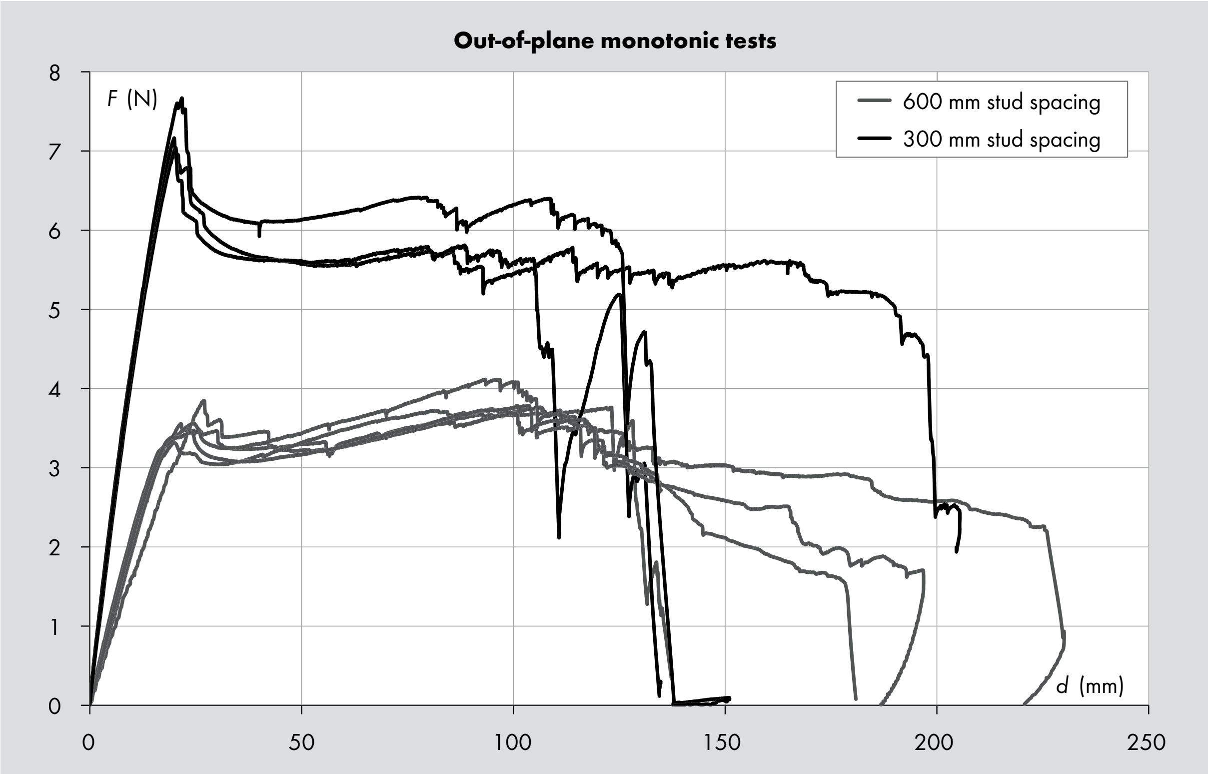


Fig. 4.60: Typical out-of-plane monotonic test results on 2700 mm height partitions and fixed joints

Fig. 4.61: Typical failure mode of out-of-plane monotonic tests

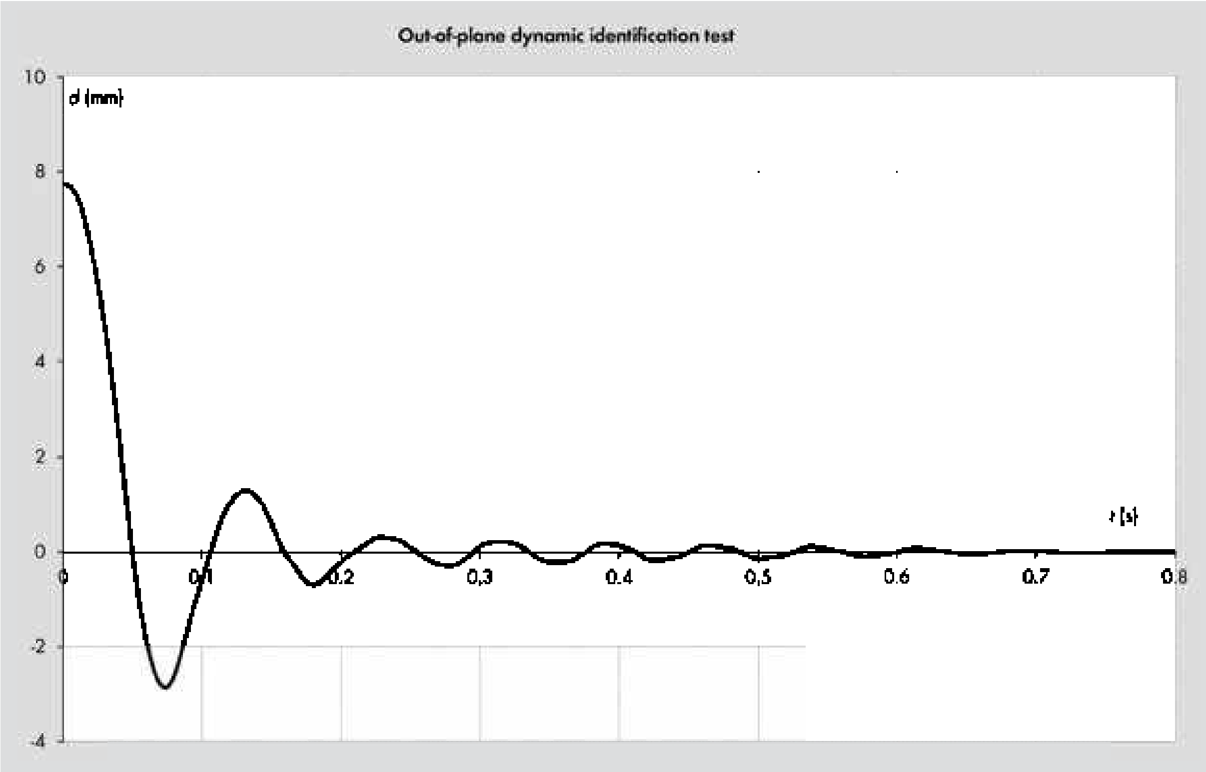


Fig. 4.62: Typical out-of-plane dynamic identification test results

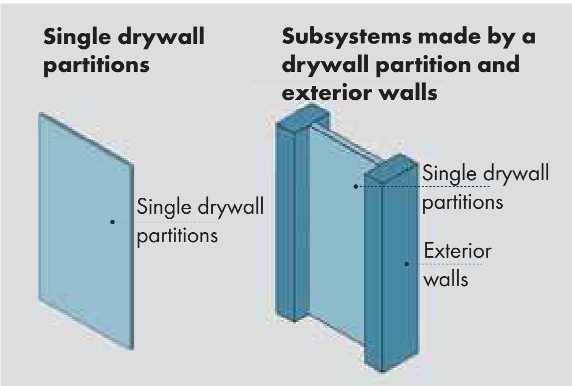


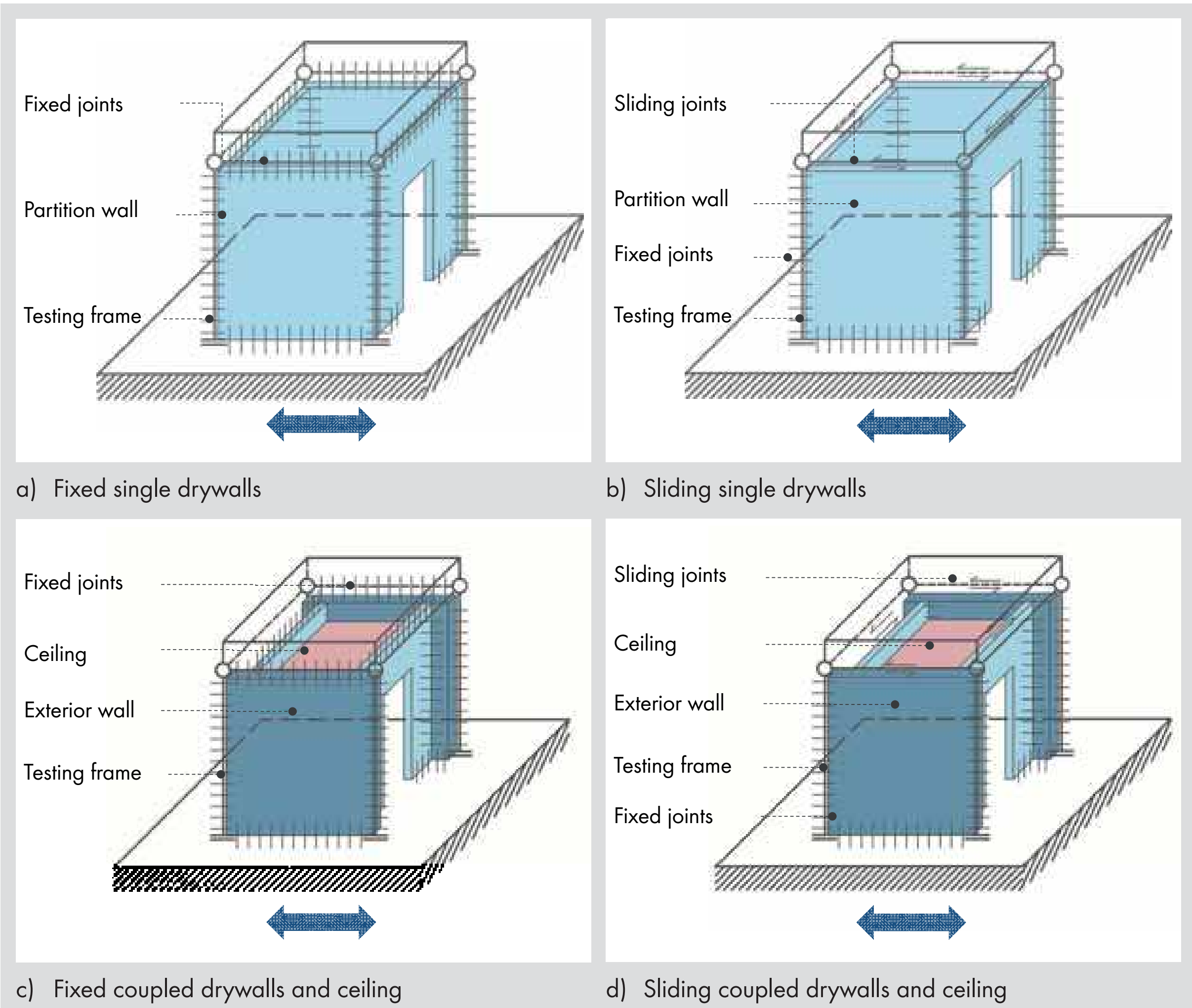
Fig. 4.63: Specimen typologies for in-plane tests

tables are included in the experimental campaign. These tests will be performed on one of the two shaking tables (3.0 x 3.0 m) of the Test Laboratory at the Department of Structure for Engineering and Architecture, which can apply loads along two translational degrees of freedom with a maximum displacement of 500 mm, maximum load of 200 kN and maximum acceleration of 1g (Fig. 4.64). For this activity, three dimensional specimens will be tested by applying the seismic input in only one horizontal direction.

Two different specimen configurations are considered. The first specimen typology aims at investigating only the behaviour of drywall partitions and consists of four drywall partitions (Fig. 4.65a, b), whereas the second configuration is representative of a constructive system consisting of two drywall partitions, two exterior walls and a suspended ceiling (Fig. 4.65c, d). For both specimen typologies, two joint configurations are considered: Fixed joints all along the partitions perimeter (Fig. 4.65a, c) and the sliding joint at the partition top (Fig. 4.65b, d).

The shaking table tests will be performed by using a purposely designed set-up structure. The set-up is a versatile 3D steel frame having a special eccentric bracing system with pre-tensioned diagonals, which allows simulation of the elastic behaviour, in terms of mass and stiffness, of different inter-storey levels of building.

Fig. 4.64: *Shaking tables at the University of Naples "Federico II"*



a) Fixed single drywalls

b) Sliding single drywalls

c) Fixed coupled drywalls and ceiling

d) Sliding coupled drywalls and ceiling

Fig. 4.65: *Shaking table tests specimen configurations*

4.7 Example of the seismic design of drywall partitions

4.7.1 General

EN 1998 defines the design criteria for evaluating the effect of the seismic action on non-structural components. In particular, the code requires that non-structural components should be verified for a design seismic force obtained by means of the equivalent static design force method (see Section 4.5.2 for more details). According to EN 1998-1 Section 4.3.5, the design seismic force F_a acting on a partition wall has to be compared with the wall design resisting force F_{Rd} . This verification can be expressed with the following condition:

$$\frac{F_a}{F_{Rd}} \leq 1 \tag{4.12}$$

In order to clarify the procedure adopted by EN 1998 for the seismic verification of drywall partitions, various 2.70 m height lightweight steel gypsum board partition walls have been considered to be placed in a multi-storey residential building. Therefore, a case study has been developed, and thus the investigated drywall partitions have been verified according to the prescribed specifications as set down by the European codes.

4.7.2 Definition of the case study

The case study is assumed to be an eight-storey residential building having a structural system composed of a reinforced concrete frame. A rectangular plan with an area of 200 m^2 and an inter-storey height of 3.20 m are considered as geometrical parameters for the investigated building (Fig. 4.66).

The structural systems are completed with lightweight steel drywall systems, e.g. exterior walls, suspended ceilings and interior partitions. The investigated interior partitions are composed of a single lightweight steel frame and double-layer cladding. Partition walls are made of C-section studs of dimensions 75 x 50 x 7.5 x 0.6 mm (web depth x flange width x lip size x thickness) spaced at 600 mm and cladded with two layers of 12.5 mm thick gypsum boards on each side. The total thickness of the partition wall is equal to 125 mm. DX51 + Z steel grade is adopted for the wall frame (with ultimate tensile strength ranging between 270 and 500 MPa according to EN 10346). The gypsum boards are connected to the steel frame with 3.5 x 35 mm (diameter x length) drywall screws spaced at 250 mm.

The dead and permanent loads, uniformly distributed on the floors, are assumed equal to 5.05 kN/m^2 and 4.45 kN/m^2 for the intermediate floors and roof floor, respectively. A live load of 2.00 kN/m^2 has been assumed for both intermediate and roof floors. The adopted values for the acting loads are given in Tab. 4.16.

The building, which is assumed to be located in a medium/high intensity seismicity zone in Europe has been designed according to EN 1992-1-1 and EN 1998-1. According to this latter code, the structure has been designed for medium ductility class. In particular, the peak ground acceleration is set equal to 0.25 g for a rare seismic event with a 475-year return period in 50 years. The assumed foundation soil is Type C, and the soil factor is set equal to 1.30. The main parameters for the calculation of the seismic action acting on the building at the Ultimate Limit State are summarized in Tab. 4.17. The fundamental vibration period of the building is equal to 0.75 s.

4.7.3 Verification according to the European codes

Evaluation of the wall design seismic force

The design seismic force F_a is supposed to be applied at the centre of mass of partition walls in the most unfavourable direction, which is the out-of-plane direction (Fig. 4.67), and it is defined according to the following relationship:

$$F_a = (S_a \cdot W_a \cdot \gamma_a)/q_a \tag{4.13}$$

where w_a = 0.43 kN/m^2 is the wall unit weight; h = 2.7 m is the wall height; γ_a = 1.0 is the importance factor of the partition wall; q_a = 2.0 is the behaviour factor for the partition wall; and S_a is the seismic coefficient (i.e. the design acceleration normalized with respect to the acceleration of gravity) obtained as follows:

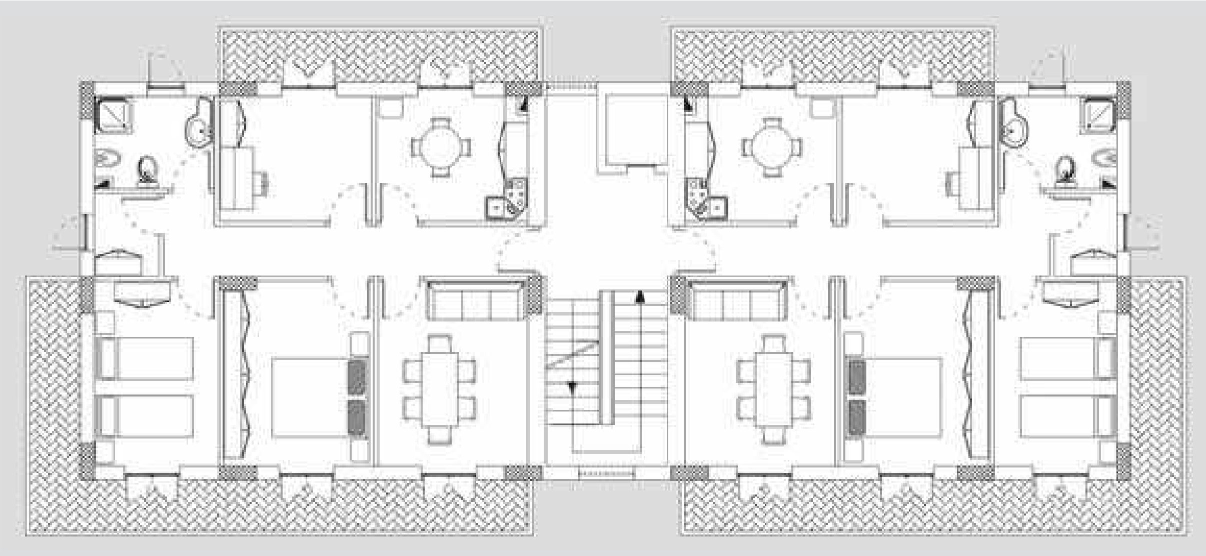


Fig. 4.66: Generic plan view of the investigated building

Tab. 4.16: Adopted values for the building loads

Structural element	**Dead loads (kN/m^2)**	**Live loads (kN/m^2)**
Floors (kN/m^2)	5.05	2.00
Roof (kN/m^2)	4.45	2.00

Tab. 4.17: Parameters for the definition of seismic action

Parameters for the definition of seismic action	
a_g (g)	0.25
T_c (s)	0.37
S	1.30
a_g: Peak ground acceleration T_c: Starting period of the constant acceleration of the horizontal spectrum S: Soil factor	

$$S_a = \alpha \cdot S \cdot \left[\frac{3 \cdot (1 + z/H)}{1 + (1 - T_a/T_1)^2} - 0.5\right] \geq \alpha \cdot S \qquad (4.14)$$

where a is the ratio between the peak ground acceleration a_g and the acceleration of gravity g, set equal to 0.25; S = 1.30 is the soil factor; H = 25.60 m is the total building height; z is the height of the partition's centre of gravity measured from above the foundation level; T_1 = 0.75 s is the fundamental vibration period of the building; and T_a is the fundamental vibration period of the partition wall, which should be evaluated by means of experimental tests or provided by the manufacturer. In this case, the fundamental vibration period of the partition wall is assumed equal to 0.07 s as a typical value.

Taking into account the equations 4.13 and 4.14, the wall design seismic force can be expressed as follows:

$$F_a = \alpha \cdot S \cdot \left[\frac{3 \cdot (1 + z/H)}{1 + (1 - T_a/T_1)^2} - 0.5\right] \cdot \frac{W_a \cdot h \cdot \gamma_a}{q_a} \geq \alpha \cdot S \cdot \frac{W_a \cdot h \cdot \gamma_a}{q_a} \qquad (4.15)$$

For example, by considering a partition wall located at the fourth floor of the investigated reinforced concrete frame building (i.e., z = 10.95 m):

$$F_a = 0.25 \cdot 1.30 \cdot \left[\frac{3 \cdot (1 + 10.95/25.60)}{1 + (1 - 0.07/0.75)^2} - 0.5\right] \cdot \frac{0.43 \cdot 2.7 \cdot 1.0}{2.0}$$

$$= 0.35\ kN/m \geq 0.25 \cdot 1.30 \cdot \frac{0.43 \cdot 2.7 \cdot 1.0}{2.0} = 0.19\ kN/m$$

Thus, in this case, the wall design seismic force is equal to: F_a = 0.35 kN/m

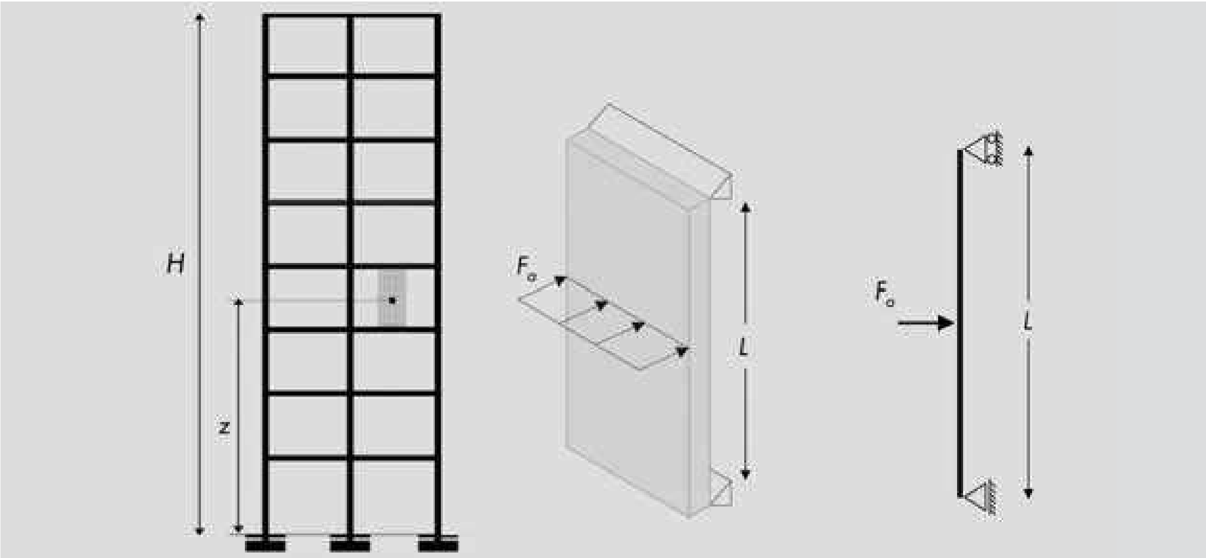


Fig. 4.67: Structural model of the partition wall

Evaluation of the wall design resisting force

In order to perform the verification required by EN 1998-1, the wall design resisting force could be evaluated by means of experimental tests or provided by the manufacturer. In this case, the study of the wall design resisting force is evaluated by means of the effective width method according to EN 1993-1-3 (Eurocode 3) /4.3/. The common hypotheses adopted are:

- The influence of the cladding panels is neglected in the evaluation of the design resisting force
- The cladding boards represent a fully effective restraint against global buckling modes (i.e. lateral-torsional buckling)
- The studs are schematized by simply supported beams subjected to concentrated forces acting at the mid-span
- The DX51 + Z steel grade is equivalent to the S320 GD+Z steel grade, with characteristic yield and ultimate tensile strengths set equal to 320 MPa and 390 MPa, respectively

The effective width method (Fig. 4.68) is used to evaluate the effective section of the stud, obtained by removing from the gross cross-section those parts that do not contribute to resistance of profile because of local and distortional buckling. Therefore, the resistance of the members is calculated by using the effective cross-sectional properties. In particular, the methodology provided by EN 1993-1-3 has been applied by neglecting the flange and web intermediate stiffeners for C-section studs.

The evaluation of the wall design resisting force F_{Rd} can be obtained with the following relationship:

$$F_{Rd} = \frac{n \cdot 4 \cdot M_{c,Rd}}{L} \qquad (4.16)$$

where n = 1000/600 = 1.667 is the number of studs for a wall with a unit length (1 meter), L = 2700 mm is the stud height and $M_{c,Rd}$ is the design bending resistance of a single stud, which can be obtained with the following formula:

$$M_{c,Rd} = \frac{W_{eff} \cdot f_{yk}}{\gamma_{M0}} \qquad (4.17)$$

where W_{eff} = 1344 mm^3 is the effective section modulus, f_{yk} = 320 MPa is the characteristic basic yield strength and γ_{M0} = 1.0 is the partial factor for resistance of the cross-section.

Taking into account the equations 4.16 and 4.17, the wall design resisting force can be expressed as follows:

$$F_{Rd} = \frac{n \cdot 4 \cdot W_{eff} \cdot f_{yk}}{L \cdot \gamma_{M0}} \qquad (4.18)$$

Therefore, the value of the wall design resisting force to be adopted for the verification required by the code is:

$$F_{Rd} = \frac{1.667 \cdot 4 \cdot 1344 \cdot 320}{2700 \cdot 1.0} = 1.06 \text{ kN/m}$$

Comparison between the wall design seismic and resisting forces

By comparing the obtained values of the wall design

Tab 4.18: Comparison between the wall design seismic and resisting force

Storey	**z (mm)**	**F_a (kN/m)**	**F_{RD} (kN/m)**	**F_a/F_{Rd}**
1	1.35	0.23	1.06	0.22
4	10.95	0.35	1.06	0.33
8	23.75	0.51	1.06	0.48

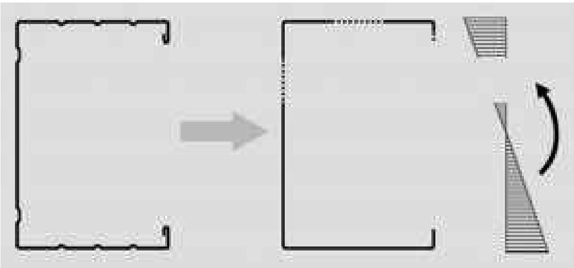


Fig. 4.68: Evaluation of the wall design resisting force

seismic and resisting forces, the verification can be carried out according to equation (4.12) as follows:

$$\frac{F_a}{F_{Rd}} = \frac{0.35 \text{ kN/m}}{1.06 \text{ kN/m}} = 0.33 < 1$$

Therefore, the seismic verification for the considered partition wall is satisfied.

Furthermore, in order to compare the design seismic forces obtained for several partition walls located at different building storeys, Tab. 4.18 lists the results of seismic verification for partition walls located at the ground, fourth and eighth storeys. It can be observed that the ratios between the design seismic forces and the design resisting forces obtained for walls placed at different building storeys range between 0.22 and 0.48 and the seismic verification is always satisfied.

The above presented procedure only involves verification of the wall bending. In general, the verification of the connections between the partition wall and surrounding structure should also be taken into account. In particular, to perform this verification the connection strength should be evaluated by means of experimental tests or provided by the manufacturer. However, the commonly adopted connection solution made of 6 x 35 mm (diameter x length) plastic dowels spaced at 900 mm satisfies the seismic verification required by EN 1998 for the case study under consideration.

5 Seismic design of structural steel drywall systems

Ornella Iuorio, Luigi Fiorino, Raffaele Landolfo

In this chapter, structural drywall systems made of lightweight steel profiles and a cladding made of gypsum, cement or wood-based boards are analyzed. The main structural typologies are presented, highlighting advantages and disadvantages, and among them, the stick-built constructions are discussed in detail. The designs under vertical and horizontal loads are discussed. In particular, two different approaches named "all-steel" and "cladding-braced" are presented. The first approach does not consider the presence of cladding boards, and the interaction between the boards and cladding is neglected. Therefore, in this case, the boards have only a finishing function. However, in the case of the cladding-braced approach, the presence of cladding is considered to calculate the load bearing capacity of walls and floors. Moreover, for the design under horizontal loads, the interaction between profiles, boards and their connections can represent the real lateral resisting system. An overview of the main codes for the structural design of this system, such as the EN 1993-1.3 (Eurocode 3 Part 1.3) and the North American standards with main reference to the AISI S213-07/S1-09 is provided, together with the introduction of design manuals, such as "Prescriptive Method For Residential Cold-Formed Steel Framing" by NASFA and "Workpack design for Steel House" by LSK. Finally, research outcomes and developments are discussed.

5.1 Lightweight steel constructions

5.1.1. Peculiarities and typologies

The demand for low-cost high performance constructions is propogating the adoption of cold-formed steel (CFS) systems as a competitive and eco-friendly solution. CFS systems provide the benefits associated with dry constructions (short execution time, quality of products and reduced disruption and noise on site as well as minimum site waste), typical distinctive features of CFS systems (e.g. lightness, high structural performance and good behaviour under seismic actions) and economic value, due to the simplicity of assembling and erection, short execution time, and few man-hours. In addition, the use of recyclable materials, the flexibility of systems and the possible reuse of elements assure a low environmental impact (Fig. 5.1).

The CFS systems can be categorized into three large families on the basis of the pre-fabrication level: Modular, panelised and stick-built systems. In particular, modular constructions (Fig. 5.2a) use pre-engineered modular units, made by assembling completed frames made of any finishing elements (e.g. doors, windows and any finishing material) in the workshop and by the vertical and horizontal addition of the units on site. Panelised constructions (Fig. 5.2b) are made of two-dimensional elements (wall and floor sub-frames and roof trusses), which are pre-fabricated in the workshop. Thermal insulation and some of the lining and finishing materials may also be applied to the steel sub-frame to form boards and to reduce execution times. This system is particularly suited to build houses characterized by repetitive elements. Stick-built constructions (Fig. 5.2c) are obtained by assembling on site, a modest number of members (e.g.

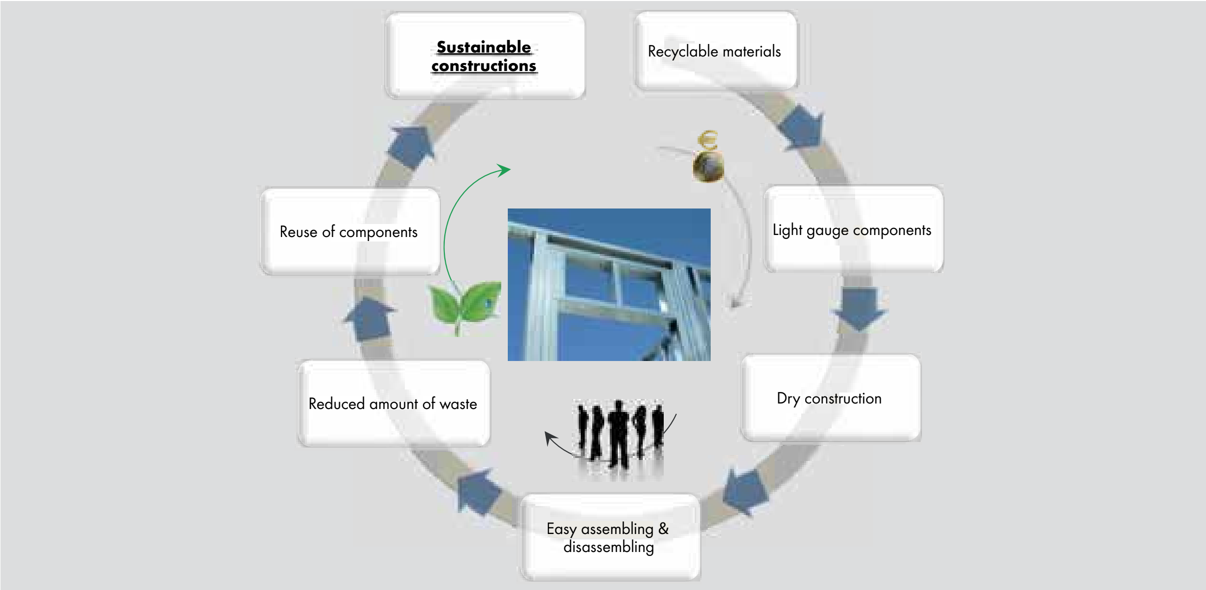


Fig. 5.1: Ecological value

a) Modular construction: Residence for elderly people, Obere Mühle, (2014) /Cocoon/

b) Panelized construction: Beaufort Court, Lillie Road, London (2003) /Sergio Russo Ermolli/

c) Stick- built construction: Casa Buna, Comanesti – Romania (2008)

Fig. 5.2: Cold-formed steel construction systems

Fig. 5.3: Typical steel framing

studs, joists and rafters) and cladding boards, which are fastened together by screws, nails or bolts.

This chapter mainly focuses on stick-built systems that represent the most widespread typology, due to the simplicity of realization. Therefore, a brief overview of the main structural subsystems is provided, together with the analysis of the structural behaviour and the main research outcome concerning the seismic response and design procedures.

5.1.2 Stick-built structural subsystems

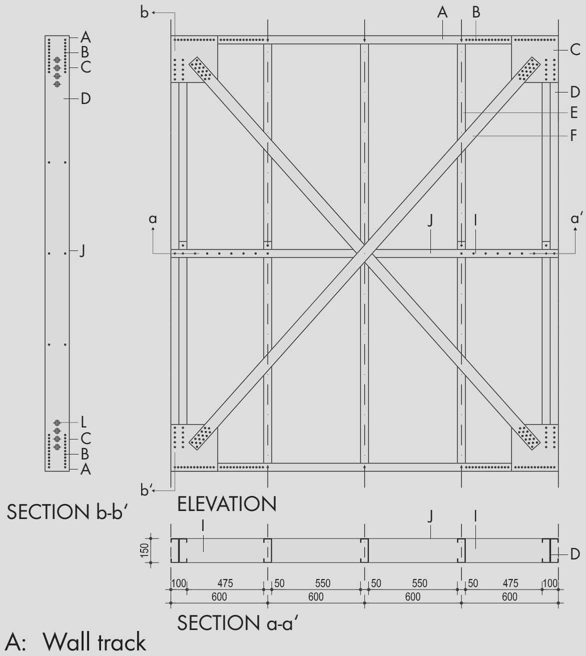
The main structural subsystems of a CFS stick-built construction can be identified in the foundation, walls and floors. The lightness of CFS systems allow the erection of low-rise buildings on minimal foundations, and therefore, the construction can be easily set on poured concrete walls or slab-on-ground foundations (Fig. 5.3).

The walls can be subdivided into load bearing and non-bearing walls. The load bearing walls are comprised of studs, i.e. vertical load bearing members spaced (s) at 300 – 600 mm, in line with floor joists. The studs are fastened at each end to wall tracks, which have the function of supporting the studs laterally and to distribute loads among the studs. At mid-wall height, straps may be installed and connected to both flanges of the studs, and some lipped channel profiles (blocking) can be introduced at the ends, with the aim of reducing the stud in-plane unbraced length (Fig. 5.4). In a seismic area, the ability to resist horizontal in-plane actions can be achieved by different systems:

- X-bracing (Fig. 5.4)
- Mixed solutions obtained by the introduction of both cladding boards and X-bracing. Moreover, in order to prevent the wall from up-lift due to horizontal in-plane actions, hold-down anchors have to be introduced at the end of each resisting wall (Fig. 5.5)
- Fastening structural cladding boards on one or both wall sides (Fig. 5.6)

The result is a sandwich construction, where each board can bear perpendicular pressure on its surface as well as in-plane loads. The internal wall cavity is ideal for inserting cables, pipes and insulation. An unlimited range of materials can be used as finishing of both the inner and the outer surface: Paint, wallpaper, coating, fabric, etc., as suggested in Fig. 5.7.

Floors are realized with horizontal load bearing members (joists) and a cladding made of gypsum and wood-based boards. Joists are usually C or Z shaped members, located in line with the wall studs, fastened at each end to a floor track (Fig. 5.8). Floor spans can range from about



A: Wall track
B: Steel-to-steel screws
C: Gussett
D: Back-to-back chord studs
E: Single stud (inner)
F: Flat strap for bracing
I: Wall blocking
J: Flat strap
K: Hold-down
L: Anchor bolts between studs and hold-down

Fig. 5.4: Typical X-braced wall

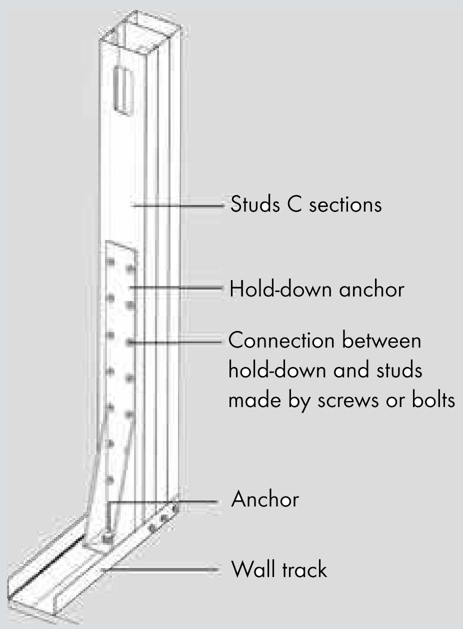
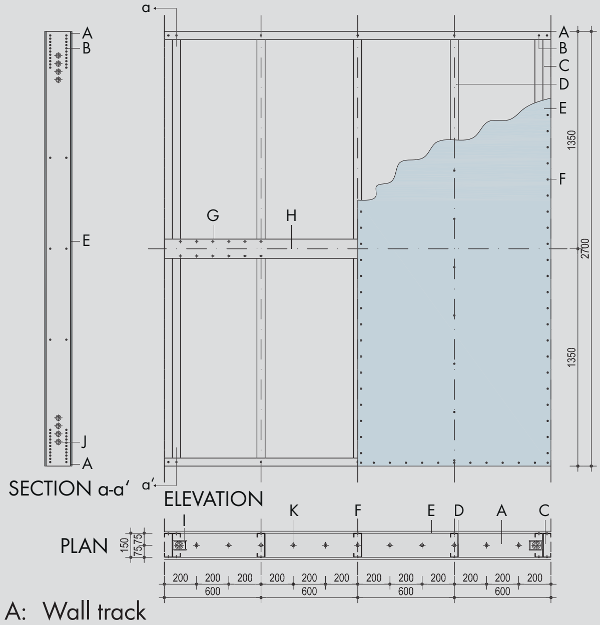


Fig. 5.5: Hold-down anchor



A: Wall track
B: Steel-to-steel screws
C: Back-to-back chord studs
D: Single stud (inner)
E: OSB structural board
F: Cladding-to-frame screws
G: Wall blocking
H: Flat strap
I: Hold-down
J: Anchor bolts between chord studs and hold-down
K: Shear anchors

Fig. 5.6: Typical cladding-braced wall

a) Masonry finishing /Stratco (Australia) Pty Ltd www.stratco.com.au/

b) Panel finishing /BFS school/

c) Plaster finishing /Cocoon/

Fig. 5.7: Finishing

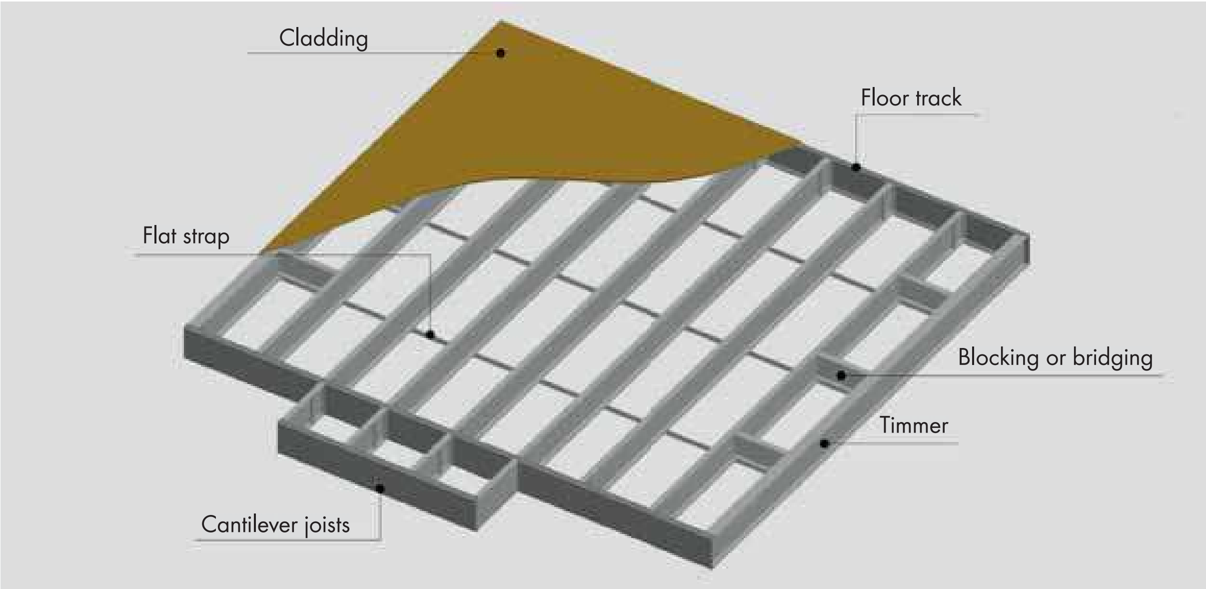
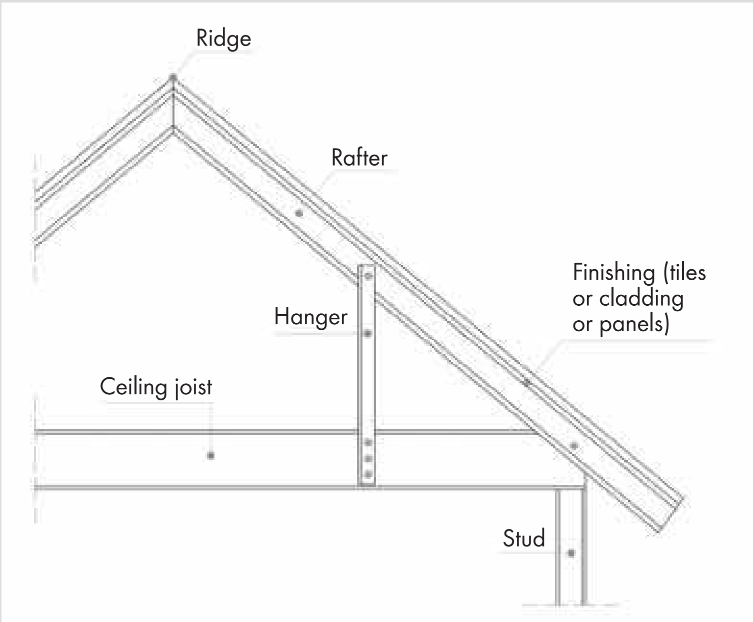


Fig. 5.8: Typical floor framing

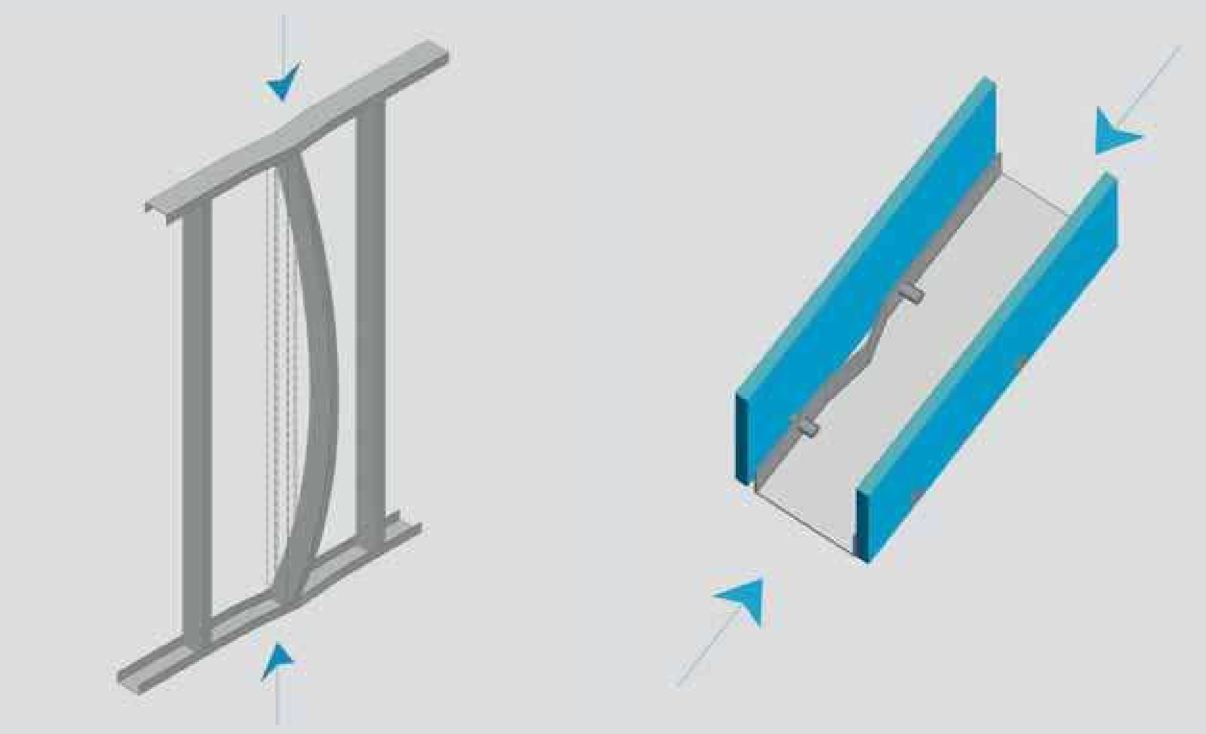


a) Schematic drawing with indication of the main structural components

b) All steel solution /Stratco (Australia) Pty Ltd www.stratco.com.au/

c) Cladding-braced solution /Cocoon/

Fig. 5.9: Typical roof framing systems



a) Global buckling of an "isolated" stud

b) Local buckling of a profile with cladding

Fig. 5.10: Difference between global and local buckling

4 up to 8 meters depending on the depth and type of the joist. A lightweight steel building can feature pitched or mono-pitched, flat or curved roofs. In any case, the main structural components of roof framing are (Fig. 5.9):

- Rafters, which are structural framing members (usually sloped C section profiles) that support roof loads
- Ceiling joists, i.e. horizontal, structural CFS profiles that support the ceiling and attic loads (typically C section profiles)
- Ridge members, i.e. horizontal members placed at the intersection between the top edges of two sloping roof surfaces.

At regular distances, hangers (typically C profiles) are usually installed to connect rafters and ceiling joists. Moreover, as for walls and floors, blocking and flat straps can be introduced to strengthen the in-plane members. Where possible, the roof frames should be aligned with the studs of the supporting walls. Where this is not possible, the use of a robust, load carrying, top track will permit trusses or other roof framing to be located with a reasonable degree of flexibility.

5.2. Structural Concept

5.2.1. All-steel and cladding-braced approaches

As for the structural design of traditional buildings, also in the case of CFS structures, there are two main performance requirements: To transfer the vertical loads and the horizontal forces acting on the structure to the ground /5.1/.

The design under vertical loads does not represent a very complex issue. In fact, considering that the construction systems consist of dry assemblies, in which boards and profiles are connected by pinned joints, the structural analysis for vertical loads is the resolution of a statically determined pendulum scheme, where the internal forces for each element can be easily obtained by the acting loads.

An interesting feature is the possibility to carry out the structural checks according to two different approaches: "all-steel design" and "cladding-braced design", also

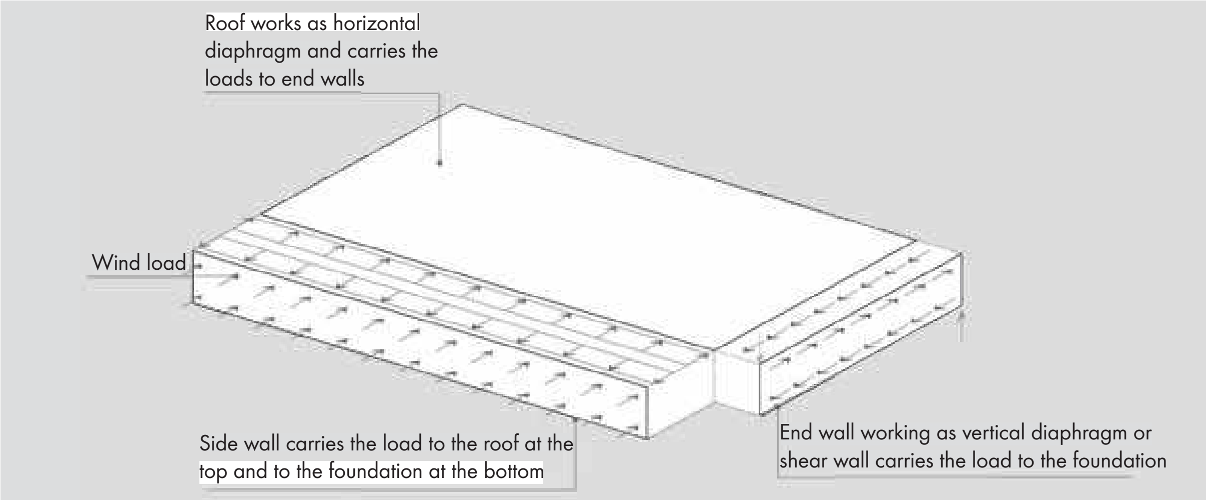


Fig. 5.11: Load pattern in case of horizontal actions

Fig. 5.12: X-bracing systems and corner detail /Condino Engineering/

known in scientific context as "sheathing-braced design". The first approach does not consider the presence of cladding boards, and the generic profile is assumed as isolated (free-standing), by neglecting the interaction between the profile itself and the cladding (Fig. 5.10a). In this case, the load bearing capacity of the member is calculated only considering the end conditions and intermediate restraints, if they are present. Therefore, the buckling length of a member is evaluated neglecting the stabilizing effect provided by the cladding board.

The latter approach calculates the load bearing capacity of a member taking into account the presence of the cladding. In fact, when the cladding has sufficient strength and stiffness and it is effectively connected to steel profiles, the bending resistance (for beams) and the axial resistance (for studs) are increased because of the interaction with the cladding boards. This phenomenon is due to the bracing effect of cladding on profiles (Fig. 5.10b), which mainly improves the strength against global and distortional buckling modes.

Fig. 5.13: Cladding system and cladding–to–frame connection

The design under horizontal loads, mainly wind and seismic loads, represents a more delicate issue and has been the object of many research projects as will be illustrated in the following (Section 5.4). In fact, when the building is subjected to a horizontal load, floors and roofs have to be able to act as a diaphragm and transfer the loads to the walls, which, in turn, have to resist these loads and transfer them to the foundations (Fig. 5.11). Therefore, the global lateral response of the building is strongly connected to the structural behaviour of floors and walls under in-plane actions.

The in-plane resistance of these structures can be achieved either using steel bracing (usually X-bracing) or taking into account the cladding-to-frame interaction. Therefore, both for the vertical load design, as well as for design under horizontal loads, it is possible to distinguish between the all-steel design and cladding-braced design approaches. When the in-plane resistance is assured by X-bracings, steel straps are generally used to obtain the diagonal elements. In floors and roofs, steel straps are connected to the bottom flanges of joists, while in walls they are connected to the external faces of stud flanges (Fig. 5.12). As an alternative to resist horizontal loads, the effects of cladding-to-frame interaction can be taken into account, and then, the interaction of steel framing, cladding and their connections represents the real lateral resisting

Fig. 5.14: Floor: Indication of joists and web stiffeners

system. When this approach is used, floor and walls can be considered as diaphragms, and the structural response depends on their elements and relevant connections (Fig. 5.13).

5.2.2 Design under vertical loads

The design under vertical loads mainly consists of the selection of the load bearing elements of floors and walls. The typical CFS profiles generally regard C, Z and U cross-sections that are classified as Type 4 cross sections, (acc. to EN 1993 / Eurocode 3), in which the behaviour is generally governed by stability phenomena. In the following, the design under vertical loads according to the two methodologies is briefly illustrated.

All-steel design for vertical loads

The evaluation of the strength capacity of a CFS member is complex, since the behaviour is strongly non-linear. The current codes allow the possibility to adopt, both, an experimental (design assisted by testing) and / or numerical approach. In fact, in case of CFS structures, design assisted by testing is often used instead of design by calculation, because it allows the structural identification and the study of the structural response of complex systems. When the numerical approach is adopted, then geometrical and mechanical non-linearity must be taken into account and the required time involved should be considered. Therefore, the current codes adopt simplified calculation methods that are mainly based on a semi-empirical evaluation of experimental results. In particular, the element model is adopted to study the local buckling. The model evaluates the buckling of each plane element that made up the CFS member on the basis of the theory of compressed plates' stability. Hence, the effective width method is adopted to define the effective/resistant cross section. Moreover, the distortional buckling can be significant for thin-walled members. Therefore, the current European code (EN 1993-1-3) provides a multi-step procedure. More details about the models can be found in /5.2/.

The main load bearing elements for floors are the joists, which can be schematized as simple supporting members. At ultimate limit states, the structural design of joists consists in checking the resistance of the cross-section (bending, shear and local transverse force) and the instability of the member (lateral-torsional buckling). While, at serviceability limit states, the control of deflections and vibrations has to be carried out.

Tab. 5.1: Summary of checks for floors

ULS		
Design bending resistance	$M_{c,Rd} = \frac{W_{eff} \cdot f_{yk}}{\gamma_{M0}}$	Eq. 6.4 EN 1993-1-3
Design shear resistance	$V_{b,Rd} = \frac{h_w \cdot t \cdot f_{bv}}{\gamma_{M0} \cdot \sin(\phi)}$	Eq. 6.8 EN 1993-1-3
Design resistance for local transverse force	$R_{w,Rd}$ (these eqs. are not listed here, as they are extensive)	see eq. 6.15 and 6.16 EN 1993-1-3
Design resistance for lateral torsional buckling	$M_{b,Rd} = \chi_{LT} \cdot W_{eff} \frac{f_{yk}}{\gamma_{M1}}$	Eq. 6.55 EN 1993-1-3
SLS		
Deflection - evaluation of effective second moment of inertia	$I_{eff} = I_{gr} - \frac{\sigma_{gr}}{\sigma}\left(I_{gr} - I(\sigma)_{eff}\right)$	Eq. 7.1 EN 1993-1-3
Vibration - evaluation of effective frequency (*)	$f = \pi^2 \sqrt{\frac{E \cdot I_{eff}}{\mu \cdot L^4}}$	

* for double simply supported beam with uniform distributed acting mass

f_{yk}: is the characteristic basic yield strength
W_{eff}: is the effective section modulus
γ_{M0}: is the partial factor for resistance of the cross section
h_w: is the web depth
t: is the material thickness before cold-forming
f_{bv}: is the shear strength considering buckling
ϕ: is the slope of the web relative to the flanges
$R_{w,Rd}$: is the local transverse resistance of the web
χ_{LT}: is the reduction factor for lateral-torsional buckling
γ_{M1}: is the partial safety factor for resistance of members to instability assessed by member checks
I_{gr}: is the second moment of area of the gross section
σ_{gr}: is the maximum compressive bending stress, based on the gross cross section
$I(\sigma)_{eff}$: is the second moment of area of the effective cross section
E: is the Young modulus
μ: is the uniform distributed acting mass
L: is the joist length

The bending resistance ($M_{c,Rd}$) at mid-span cross-section of joists, which is the most stressed element in bending, can be determined by eq. 6.4, whereas to check against the shear force acting at joist support, the shear resistance ($V_{b,Rd}$) can be calculated considering only the web contribution, and it can be evaluated through eq. 6.8. In addition, it is always necessary to check the resistance against the local transverse forces at the joist support according to eq. 6.13. This check can be neglected if specific reinforcing profiles, called "web stiffeners", are attached to the web at joist ends (Fig. 5.14).

The calculation of the resisting moment against the lateral-torsional buckling can be carried out by applying the eq. 6.55 EN1993-1-1. It has to be noted that this check is strongly influenced by the arrangement of lateral torsional restraints. In particular, at joist ends, lateral torsional restraints are represented by the "web stiffeners", while, along the joists, the restraints can be obtained by connecting a steel flat strap to the bottom flange of all joists and fixed by at least one field between two joists by means of a profile called "blocking".

At serviceability limit states, the calculation of maximum deflection needs to take into account the effects of local buckling. In fact, local buckling can influence the behaviour of slender sections as well at serviceability limit states, since it can occur before reaching the maximum elastic stress. The effects of local buckling consist of a reduction of the effective section, which depends on the

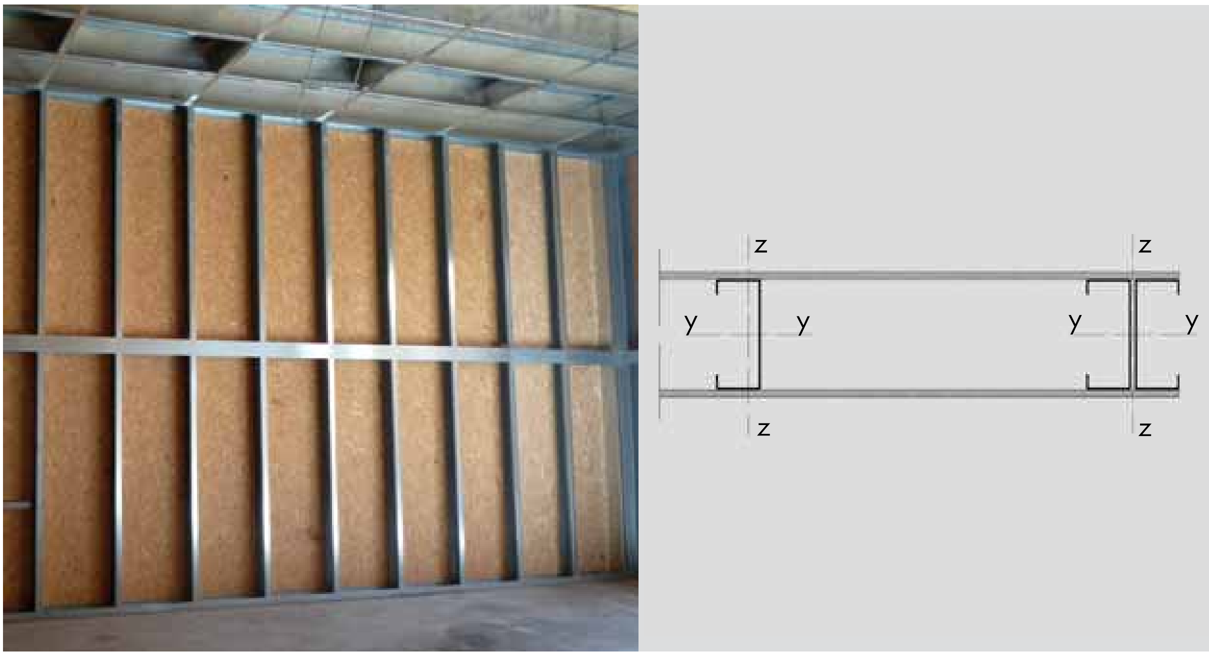


Fig. 5.15: Wall studs

stress magnitude. Therefore, deflections can be calculated considering an effective second moment of area, which can be taken variably along the span. Alternatively, a uniform value based on the maximum acting bending moment can be assumed. The effective second moment of area at the serviceability limit state can be calculated by eq. 7.1.

Finally, for the vibration check, the assessment of the lowest natural frequency is required, where this last case can be considered equal to the frequency for a single member. In /5.2/, a lower limit of 3 Hz is recommended for floors, over which people walk regularly, with a more severe limit of 5 Hz for floors used for dancing or jumping, such as gymnasiums or dance halls. Tab. 5.1 summarizes the main equations for joists verification.

Load bearing walls can have many structural functions. Their primary function is to carry vertical loads from the floors and roof to foundations. In this case, the structural scheme consists in a pinned column subjected only to axial compression coming from the wall weight and the loads transmitted by the upper structures (walls and floors). When walls have also a bracing function, they have also to resist in-plane lateral loads, due to wind or an earthquake, which have to be transmitted to the foundations. In the case of external walls, they also have to resist the lateral pressure action of the wind and

Fig. 5.16: Wall lateral restraint

transmit it to the floor and roof diaphragms. This action is usually schematized as a uniform load acting on the stud perpendicular to the wall surface. Hence, the studs of external walls are subjected to combined compression and biaxial bending moments (Fig. 5.15). Therefore, in general, at the ultimate limit state, the resistance of a cross-section, which also takes into account the effects of local and distortional bucking, and the global buckling resistance under compression and biaxial bending moments have to be calculated.

The buckling resistance is strongly influenced by the buckling length. When studs have no intermediate restraints, the buckling length for both in-plane and out of plane directions is equal to the wall length. Instead, when

Tab. 5.2: Summary of checks for walls

ULS		
	Equations	
Strength verification	$\frac{M_{y,Ed}+\Delta M_{y,Ed}}{M_{cy,Rd}}+\frac{M_{z,Ed}+\Delta M_{z,Ed}}{M_{cz,Rd}}\pm\frac{N_{Ed}}{N_{c,Rd}}\leq 1$	Eq. 6.25 EN 1993-1-3
Buckling resistance	$\frac{N_{Ed}}{\chi_y\frac{N_{Rk}}{\gamma_{M1}}}+k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}}+k_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}}\leq 1$ $\frac{N_{Ed}}{\chi_z\frac{N_{Rk}}{\gamma_{M1}}}+k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}}+k_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}}\leq 1$	Eqs. 6.61 and 6.62 EN 1993-1-3

$M_{y,Ed}$ and $M_{z,yd}$: are the design acting bending moments about y-y and z-z axis, respectively
$\Delta M_{y,Ed}$ and $\Delta M_{z,Ed}$: are the design additional moment due to shift of local axis about y-y and z-z axis, respectively
N_{Ed}: is the design acting compression load
$M_{cy,Rd}$: is the design bending moment resistance about the y-y axis
$M_{cz,Rd}$: is the design bending moment resistance about the z-z axis
$N_{c,Rd}$: is the design compression resistance
N_{Rk}: $= A_{eff} f_{yb}$, $M_{y,Rk} = W_{eff,y} f_{yb}$, $M_{z,Rk} = W_{eff,z} f_{yb}$
A_{eff}: is the effective area of the cross section
$f_{y,b}$: is the basic yield strength
$W_{eff,y}$: is the effective modulus about the y-y axis
$W_{eff,z}$: is the effective modulus about the z-z axis
γ_{M1}: is the partial safety factor for resistance of members to instability assessed by member checks
χ_y and χ_z: are the reduction coefficients due to the flexural buckling about y-y and z-z axes, respectively
χ_{LT}: is the reduction coefficient due to lateral - torsional buckling;
k_{yy}, k_{yz}, k_{zy} and k_{zz}: are the interaction coefficients

steel flat straps are attached lengthwise to stud flanges at mid-height of the wall together with a "blocking" profile, the in-plane buckling length of the studs is reduced by a half (Fig. 5.16). Tab. 5.2 summarizes the main equations for stud verification.

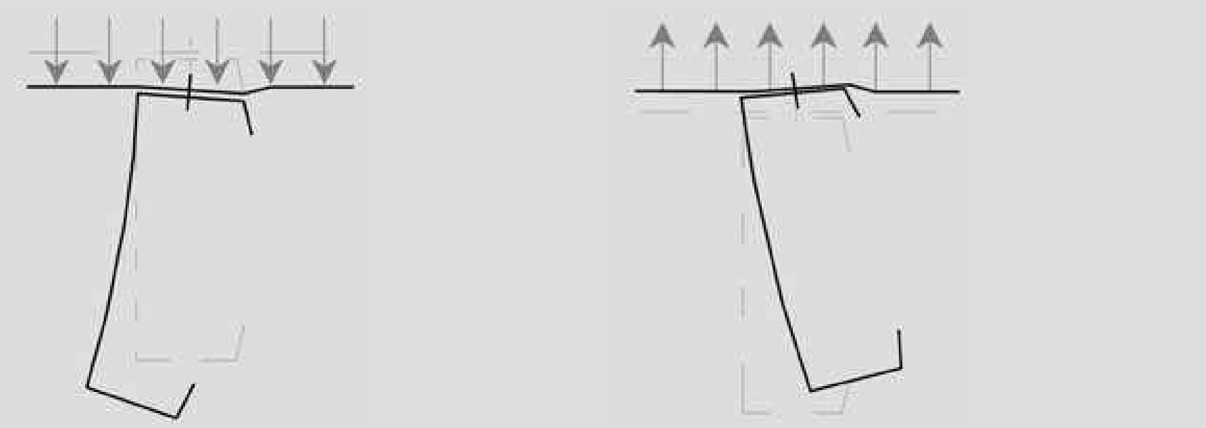
Cladding-braced design for vertical loads

As mentioned beforehand, when two-dimensional elements, such as boards or sheets, are connected to steel profiles of floors (joists) or walls (studs), the structural performance is improved, because the influence of buckling modes is mitigated by the constraining effect of the cladding.

In floors, the cladding can restrain the steel profile on the flange in compression or in tension, depending on the arrangement of two-dimensional elements as well as on the direction of loads and the structural scheme. For example, in the configuration of Fig. 5.17a, the stabilizing effect is the highest and it completely opposes lateral-torsional buckling, while in the latter configuration in Fig. 5.17b, the effect of cladding is lower, but, the load bearing capacity is increased by the torsional restraint provided by the cladding-to-frame connections.

The evaluation of stabilizing effects needs specific design procedures, because they depend on several factors, such as the shape and thickness of the profiles, flexural stiffness of the cladding and the stiffness of cladding-to-frame connections. The EN 1993-1-3 dedicates its section 10 to this topic.

For the design of walls, a remarkable reference is represented by the North American code AISI S211 "North American Standard for Cold-Formed Steel Framing – Wall Stud Design", which provides a design methodology mainly based on the interpretation of the result of research carried out in the United States /5.3, 5.4/. This methodology can be used only for the design of walls with cladding on both sides using the same cladding type.



a) Cladding on flange in compression *b) Cladding on flange in tension*

Fig. 5.17: Stabilizing effect of cladding on C-profile under bending

The design approach takes into account the stabilizing effect given by cladding reducing the buckling length only for global bucking in the wall plane. Therefore, for flexural global buckling out of wall plane, the buckling length is considered conservatively equal to the stud length, and the presence of cladding is completely neglected, whereas, for in-plane flexural and torsional buckling, the buckling length is assumed as two times the spacing of cladding to stud connections. In addition, in order to avoid the failure of the connections, the maximum value of axial resistance is limited depending on the cladding type and the spacing of connections.

5.2.3 Design under horizontal loads

Hereafter, the main steps to calculate displacements and strength of the CFS structure under horizontal loads according to the two methodologies are shown. In particular, even if the explanation is mainly focused on the behaviour of vertical walls subjected to in-plane lateral loads, these methodologies can also be applied to evaluate the response of floor decks subjected to in-plane loads.

All-steel lateral bracing

If lateral bracing is obtained by only using steel elements, and the presence of the cladding is neglected, then the lateral resisting system consists usually of concentric diagonal X-bracing, and the structural behaviour of both floors and walls is similar to that of a steel truss.

For walls, the main structural components are the steel frame composed by studs, tracks, diagonal bracings, diagonal-to-frame connections and the connections between steel framings and external structures. In particular, the bracing can be realized by steel straps, which, due to their great slenderness, are considered active only in tension. Therefore, the lateral load applied on a wall is absorbed only by the diagonal in tension, which transmits a significant axial compression force to the ends of the wall. For this reason, the design of members and connections located at wall ends is very important, especially for end studs, diagonal connections and tension anchors (Fig. 5.18).

The lateral displacement at the top of a wall (d) subjected to a horizontal load (H) can be evaluated taking into account the contribution of the main structural components (Fig. 5.19): Diagonal in tension (d_d), frame to foundation anchors (d_a) and diagonal to frame connections (d_c). In particular, the lateral wall displacement can be calculated as follows:

$d = d_a + d_d + d_c$

As well as for the evaluation of displacements, the resistance of a wall subjected to in-plane loads can be evaluated through the strength associated to each wall component. In particular, for each wall component it is possible to individually specify one or more failure mechanisms and the smallest associated strength value defines the lateral wall resistance. Therefore, the lateral wall resistance (H_c) is given by:

$H_c = \min (H_{c,s} ; H_{c,t} ; H_{c,a} ; H_{c,d} ; H_{c,c})$

in which $H_{c,s}$, $H_{c,t}$, $H_{c,a}$, $H_{c,d}$ and $H_{c,c}$ are the strengths

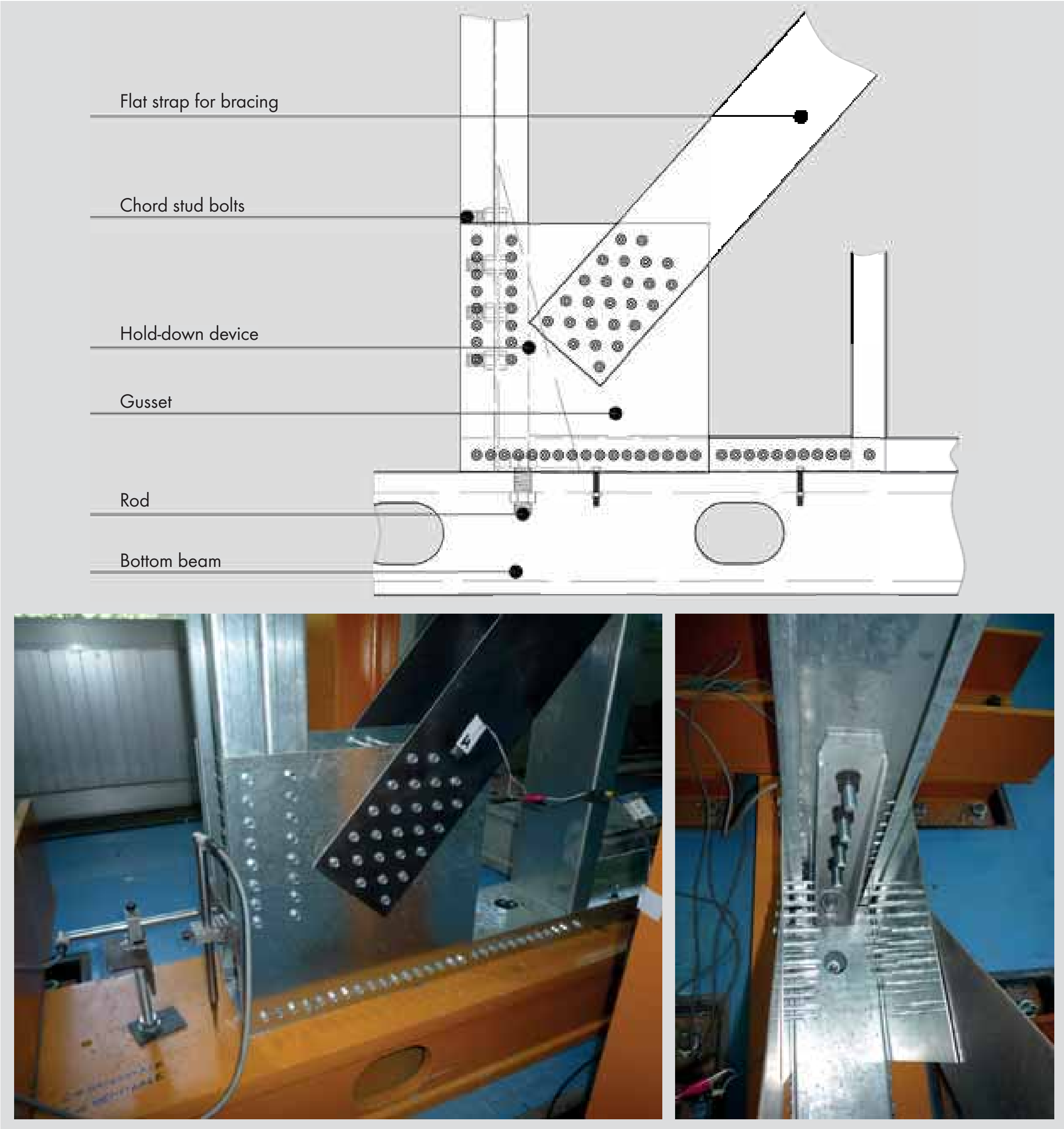


Fig. 5.18: Detail of a corner in "all-steel" walls

associated to studs, tracks, frame-to-foundation anchors, diagonals in tension and diagonal-to-frame connections, respectively.

Cladded diaphragm

When the design is carried out according to the cladding-braced methodology, floor decks and walls act as a diaphragm. In particular, floors can be considered as simple supported horizontal diaphragms subjected to a uniform load, while walls are cantilever vertical diaphragms subjected to a uniform horizontal force acting on the top edge (Fig. 5.20). The structural behaviour of diaphragms can be assumed as that of a composite I beam, in which cladding boards are the web and the chord profiles are the flanges. In particular, cladding boards absorb the shear actions, while the compression and tension axial load due to bending are resisted by chord profiles (Fig. 5.21).

Therefore, it is possible to individually specify two main components: Web boards and chord profiles. Both components are obtained as an assembly of different elements. In fact, the cladding system consists of several

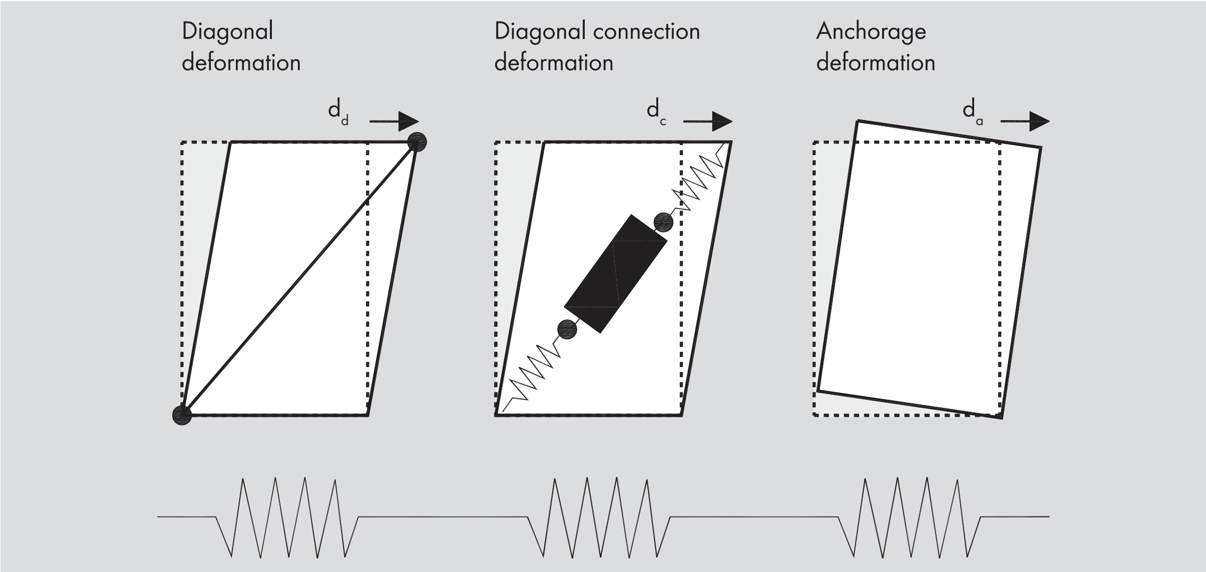


Fig. 5.19: Deformation contributions of an X-braced wall under lateral loads

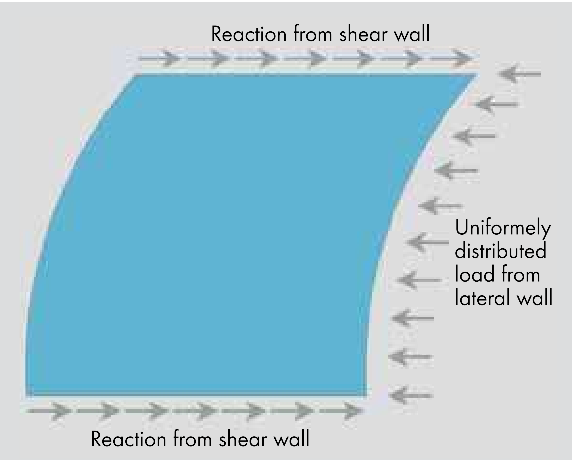


Fig. 5.20: Structural scheme of floor diaphragms

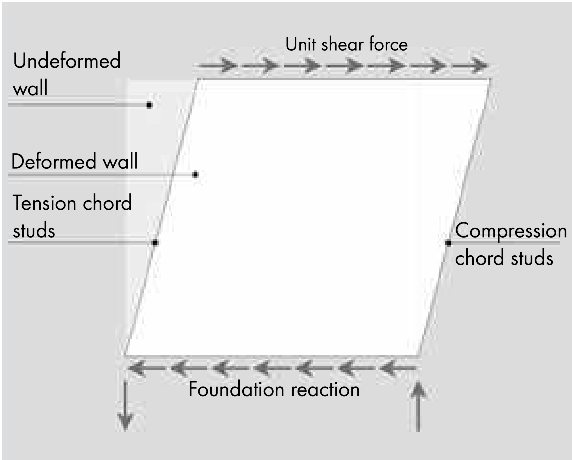


Fig. 5.21: Structural scheme of walls diaphragms

boards connected to each other, and chord profiles can be obtained by coupling two profiles.

In addition, the connections between boards and profiles strongly influence the diaphragm behaviour and, then, the evaluation of their structural response represents an important step in the design procedure.

As mentioned beforehand, with particular regards to walls, it is possible to individually specify the main structural components such as steel frame, cladding boards, cladding-to-frame connections and connections between steel frame and external structures. It is clear that the global structural response of the diaphragm (wall or floor) depends basically on the local structural response of its components.

As well as for the all-steel design, the evaluation of lateral displacement at the top of the wall under horizontal loads can be obtained by adding in series the deformation contribution of each structural component (Fig. 5.22) as follows:

$d = d_s + d_a + d_p + d_f$

in which d_s, d_a, d_p and d_f are the deformation contributions of steel frame, frame-to-foundation anchors, cladding boards and cladding-to-frame connections, respectively.

If the local behaviour of cladding-to-frame connections governs the global lateral response of walls, as generally happens, the relevant deformation produces wall lateral displacement greater than those produced by other components (Fig. 5.23):

$d_s << d_f$; $d_a << d_f$; $d_p << d_f$ (5.5)

Also the evaluation of wall lateral strength (H_c) can be

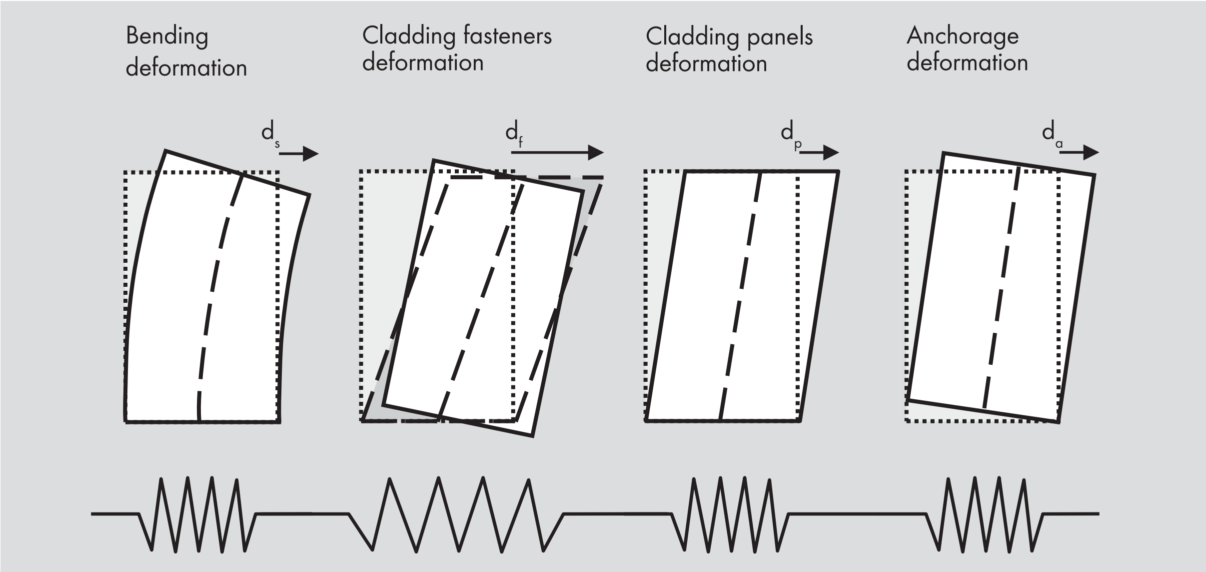


Fig. 5.22: Deformation contributions of a wall with cladding under lateral loads

carried out in a similar way to the case of the all-steel approach. Therefore, it can be obtained by the wall lateral strength associated to the failure of chord studs ($H_{c,s}$), frame-to-foundation anchors ($H_{c,a}$), boards ($H_{c,p}$) and cladding-to-frame connections ($H_{c,f}$) as follows:

$$H_c = \min (H_{c,s}\,; H_{c,a}\,; H_{c,p}\,; H_{c,f}) \qquad (5.6)$$

Generally, the failure mechanism of cladding-to-frame connections is the most ductile one.

Beside the theoretical methodologies, the lateral wall resistance can be evaluated by means of an experimental approach, based on a large number of test results carried out on full scale specimens having different configurations. This approach consists of "design tables", provided by codes that can be used only for walls consistent with fixed limitation depending on the reference experimental results.

5.3 Codification

5.3.1 General

Extensive research and product development in the past has led to national design specifications for CFS sections and structures in many countries. In particular in the following, the EN1993-1.3and AISI S100-2007 are briefly discussed. These codes can be adopted to design CFS structures in Europe and North America, respectively. Moreover, some design manuals such as "Prescriptive Method For Residential Cold-Formed" /5.3/, "The Lightweight Steel Frame House Construction Handbook" /5.5/ and "WiSH Workpack design for Steel House" /5.6/ are introduced in order to provide the readers with references for a pre-dimensioning of some CFS construction typologies for housing. Finally, an overview on the possibility to design lightweight steel-framed constructions in seismic areas according to European (EN1998) and North America (AISI S213- 07/S1-09) is given.

5.3.2 Standards for cold-formed steel structural design

The structural design of CFS thin gauge members and sheeting in Europe can be carried out according to rules given in EN1993-1-3. EN1993-1-3 applies to CFS products made from coated or uncoated thin gauge hot or cold-rolled sheet or strip that have been cold-formed by such processes as cold-rolled forming or press-braking. It is intended to be used for the design of buildings or civil engineering works in conjunction with EN1993-1-1 and EN1993-1-5. EN1993-1-3 permits only design by the

Fig. 5.23: Global collapse mechanism of a wall due to failure of cladding-to-frame connections

limit states method (LSD). The code provisions are limited to steel in the thickness range 1.0 – 8.0 mm for members, and 0.5 – 4.0 mm for sheeting. Thicker material may also be used when the load-bearing capacity is determined by testing. As application support of this code, the European Convention for Constructional Steel Work, ECCS, published in 2008 "Worked examples according to EN1993-1-3".

The North American Specification for design of cold-formed steel structural members (AISI S100) represents one of most advanced standards on CFS structural members. The first edition of this specification was published by American Iron and Steel Institute (AISI) in 1946. Starting from the 2001 edition, the AISI S100 standard, called North American Specification, was applicable in the USA, Canada and Mexico. This standard was revised and expanded in 2007 (AISI S100-2007). In the AISI S100-2007, the "Allowable Strength Design" (ASD) and "Load and Resistance Factor Design" (LRFD) methods are used in USA and Mexico, while "Limit State Design" (LSD) method is used in Canada. The specification is intended for the design of CFS structural members not more than 25.4 mm thick to be used for load-carrying purposes in buildings. Member design provisions in the AISI S100 specification are not very dissimilar from the EN1993-1-3 code, even though the notations and format of formulas are different. In some areas such as plane elements in compression and with edge or intermediate stiffeners, the AISI S100 design provisions are less complex respect to those given by EN1993-1-3. In order to provide a record of a reasoning behind, and the justification for, varies provisions of specification, the AISI published Commentaries on several editions of AISI S100. Apart from North American Specification for design of cold-formed steel structural members (AISI S100) the AISI specifications include standards for specific design of wall studs, headers, floor and roof systems and trusses.

5.3.3 Design manuals for the lightweight steel housing

The "Prescriptive method for residential cold-formed steel framing" /5.3/, developed by the North America Steel Framing Alliance (NASFA), provides the guidelines to design low-rise dwellings. The specifications given by the prescriptive method are given by means of tables, figures and information about design and execution details. These guidelines are valid for buildings consistent with fixed geometrical limitations and for acting loads, as reported in Tab. 5.3. In terms of materials, steel structural elements have to be obtained by structural steel sheets according to the requirements of ASTM.

Design guidelines for low-rise dwelling similar to the "Prescriptive Method" are provided by the Canadian Sheet Steel Building Institute (CSSBI). The document is the "The Lightweight Steel Frame House Construction Handbook" (CSSBI-59-05) /5.5/ and provides information about the design and the execution of main structural parts, such as walls, floors, roofs, openings and foundation. The

Tab. 5.3: *Geometrical limitations and range of validity for the application of NASFA /NASFA (2000)/*

Geometrical limitations	
Maximum number of floors	2
Maximum plan dimensions	11 x 18 m
Maximum wall height	3 m
Maximum roof slope	20 to 100 %
Maximum cantilever span	0.61 m
Range of validity for loads	
Floor dead load	0.48 kN/m^2 (except roofs)
Roof dead load	0.72 kN/m^2
Wall dead load	0.48 kN/m^2
Design wind speed	49 m/s
Wind exposure	A/B (suburban and wooded) C (open terrain)
Ground snow load	3.35 kN/m^2
Floor live load (First floor)	1.92 kN/m^2
Floor live load (Second floor)	1.44 kN/m^2
Roof live load	3.35 kN/m^2

different possible solutions for these structural parts are widely described through detailed drawings and their dimensioning is aided by means of specific design tables. In addition, information about non-structural features, i.e. plants, thermal insulation and finishing, are also provided. These guidelines are applicable to the construction of detached or terraced houses up to three storeys.

Another example of design manuals for the Lightweight Steel Housing is the "WiSH workpack design for steel house" /5.6/. This manual offers several "easy-to-use" tools for the design of low-rise dwellings. It aims to be a comprehensive and user-friendly design package based on the Eurocodes and addressed to architects, builders and design offices. The workpack provides design specification about finishing, architectural details, thermal and acoustic insulation; a list of construction details and an on-line software for design according Eurocodes. The workpack has been developed for two-storey dwellings with and without attic. The structure has to be realized with CFS members made with S350GD+Z coated steel grade, while self-tapping screw and bolts are used for connections. The software allows the design of different building parts such as roof, floors and walls, and it is set up to be used in Belgium, France and Spain. The user enters the input parameters relating to geometry and loads of the element selected for design. Then the software gives the adequate profile section according to checks given in EN1993-1-3 and the reaction forces to design the other elements. The software also provides the drawings of construction details.

5.3.4 Standards for the seismic design of lightweight steel-framed constructions

EN 1998-1 is the European code for seismic design of building structures. This code covers the following aspects: Seismic performance levels, types of seismic action, types of structural analysis, general concepts and rules which should be applied to all types of structures beyond those generally used for buildings. EN 1998 takes into account the capacity of structures to dissipate energy through inelastic deformations. In fact, an elastic analysis based on a response spectrum reduced with respect to the elastic one, called the "design spectrum", can be performed. The reduction is accomplished by introducing the behaviour factor q (EN 1998-1 3.2.2.5(2)), whose values are given for various materials and structural systems according to the relevant ductility classes within EN 1998. Indeed, as summarized in Fig. 5.24, seismic

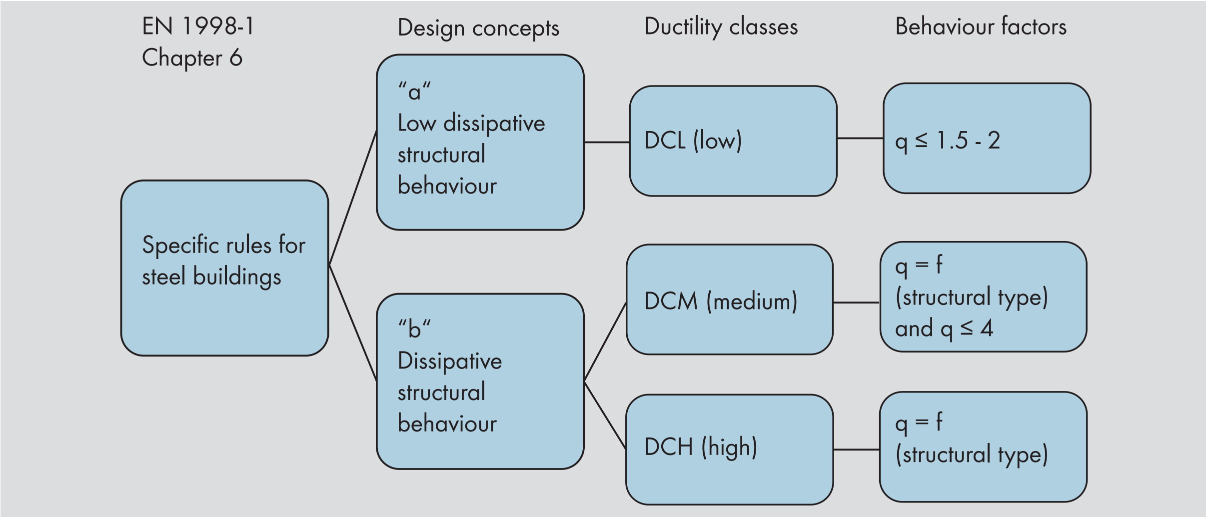


Fig. 5.24: Design Concepts according to EN 1998-1

resistant steel buildings may be designed in accordance with one of the following concepts (EN 1998-1 6.1.2(1) P): (a) Low-dissipative structural behaviour; (b) Dissipative structural behaviour.

In concept (a), the action effects may be calculated on the basis of an elastic global analysis neglecting the non-linear behaviour. In this case, the behaviour factor assumed in the calculation must be less than 2. Structures designed in accordance with concept (a) belong to the low dissipative structural class "DCL" (Ductility Class Low). Hence, the resistance of members and connections should be evaluated in accordance with EN 1993 without any additional requirement (EN 1998-1 6.1.2(4)). In this case, no restrictions on cross-section class is recommended. Hence, this implies that also members with class 4 cross-sections (lightweight steel structures) can be used. However, it should be noted that for non-base-isolated structures this simplified design is recommended only for low seismicity regions. Although the designation of low seismicity zone should be established by the competent National Authorities, a threshold value of design ground acceleration equal to 0.1 g is recommended.

In concept (b), the capability of parts of the structure (dissipative zones) to undergo plastic deformations in case of an earthquake is taken into account. The behaviour factor q assumed in the calculation is greater than 2 and depends on the type of seismic resistant structural scheme. Structures designed in accordance with concept (b) may belong to a medium structural ductility class "DCM" (Ductility Class Medium) or to a high ductility class "DCH" (Ductility Class High). These classes correspond to increased ability of the structure to dissipate energy through inelastic behaviour. Depending on the ductility class, specific design requirements are provided for both local and global structural aspects.

It is important to highlight that for DCM and DCH it is expected to have moderate and large plastic engagement in dissipative zones, respectively. Therefore, EN 1998 prescribes specific design rules both at global and local level in order to guarantee sufficient ductility in dissipative elements. In both cases, there are some rules common for all structural schemes and other specifically conceived for each typology. In addition, under these conditions, EN 1998 recommends use of a q factor greater than 2. This assumption can be applied provided that the dissipative elements in compression or bending under seismic loading satisfy a set of cross-section requirements, namely by restricting the local slenderness ratios to limit local buckling phenomena under large deformation demand. For this purpose, EN 1998 adopts the EN 1993 classification for cross-sections relating the restrictions to the value of q factor for each Ductility Class, as summarized in Tab. 5.4. Therefore, according to the current provisions of EN 1998, seismic design of diagonal strap braced walls (all-steel CFS walls) is possible by considering them as common steel structures made of Class 4 cross-sections belonging to the DCL.

Tab. 5.4: Requirements on cross-sectional class of dissipative elements depending on Ductility Class and reference behaviour factor

Design concept	**Ductility class**	**Upper limit value for behaviour factor q**	**Required cross-sectional class**
Design concept	DCL (low)	≤ 1.5 – 2 (Recommended value = 1.5)	Class 1, 2 or 3 for q > 1.5
Concept (a) Low dissipative structural behaviour	DCM (medium)	q ≤ 4 Also depending on structural type	Class 1 or 2
Concept (b) Dissipative structural behaviour	DCH (high)	Only depending on structural type	Class 1

On the contrary, seismic design of sheathed shear walls (cladding-based CFS walls) is not covered by the current Eurocodes, and there is therefore actually a gap between the European code specifications and the application of cladding-braced CFS solutions in seismic areas.
"North American Standard for Cold Formed Steel Framing - Lateral Design" (AISI S2013) represents the only specific reference for the design of CFS framing structures under seismic actions. Both strap-braced systems and shear walls with cladding are considered in the standard. In particular, special requirements for seismic design, such as behaviour factor values, aspect ratio limitations, capacity design rules for non-dissipative elements, are provided for both systems.
For the definition of the force reduction factor (R), or behaviour factor (q) according to European notation, the AISI defines two categories of seismic-resistant systems. For the first category, special seismic requirements (capacity design rules) are not required and the seismic resistant system is not specifically detailed for ductile performance. In this case, in the USA and Mexico, the force reduction factor should be taken equal to or less than 3, while in Canada, it should be taken equal to or less than 2 for sheathed shear walls, and equal to or less than 1.6 for diagonal strap braced walls. For the second category, the rules of capacity based design approach apply. Therefore, for this case, in the USA and Mexico, the force reduction factor can be taken greater than 3, in accordance with the applicable building code, e.g. the American code ASCE-07, and provides a factor value equal to 6.5 for shear walls cladded with wood panels and 4 for diagonal strap braced walls. In Canada, for the systems designed according to capacity design, an R factor ranging from 2.6 to 4.3 can be used for cladded shear walls depending on the cladding panel type, and an R factor equal to 2.5 can be used for diagonal strap braced walls.
In the case of shear walls, a specific formulation for the calculation of wall deflection and tabulated values of wall resistance based on experimental results are provided. The standard also provides the requirements for the seismic design of floor diaphragms made with CFS framing. In addition, to facilitate the use and the understanding of the code, a thorough commentary illustrates the research and scientific background of the standard.

5.4. State of the art of research

5.4.1 General

Over the last two decades, many theoretical and experimental studies have been addressed to capture the complex behaviour of CFS structures for improving the current calculation models and design codes. In particular, the research activities carried out at the

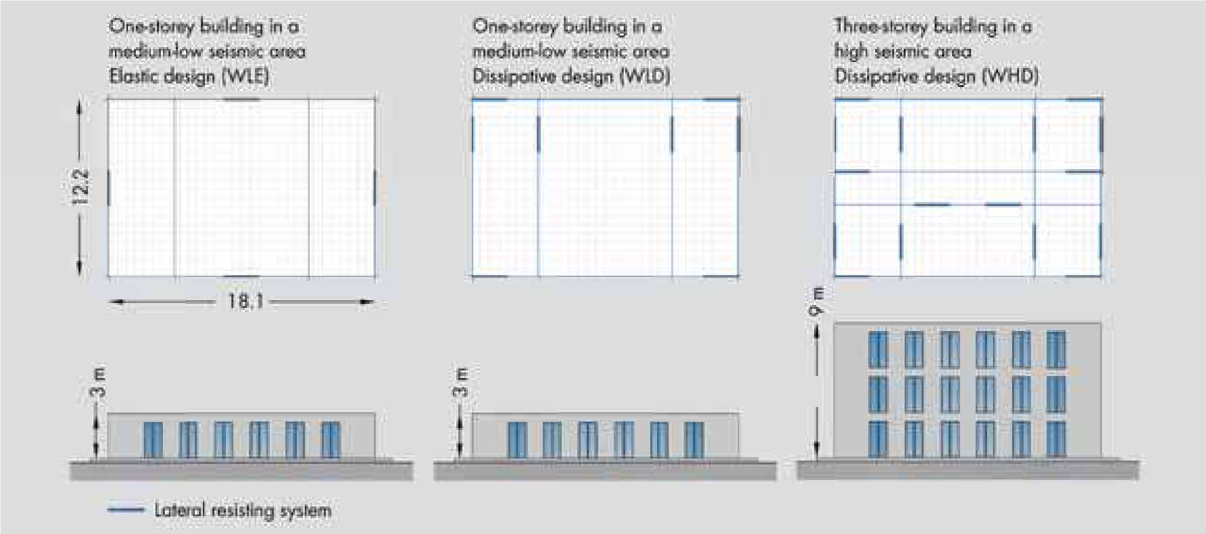


Fig. 5.25: *Plan and elevation of the three case studies with wall resisting systems highlighted*

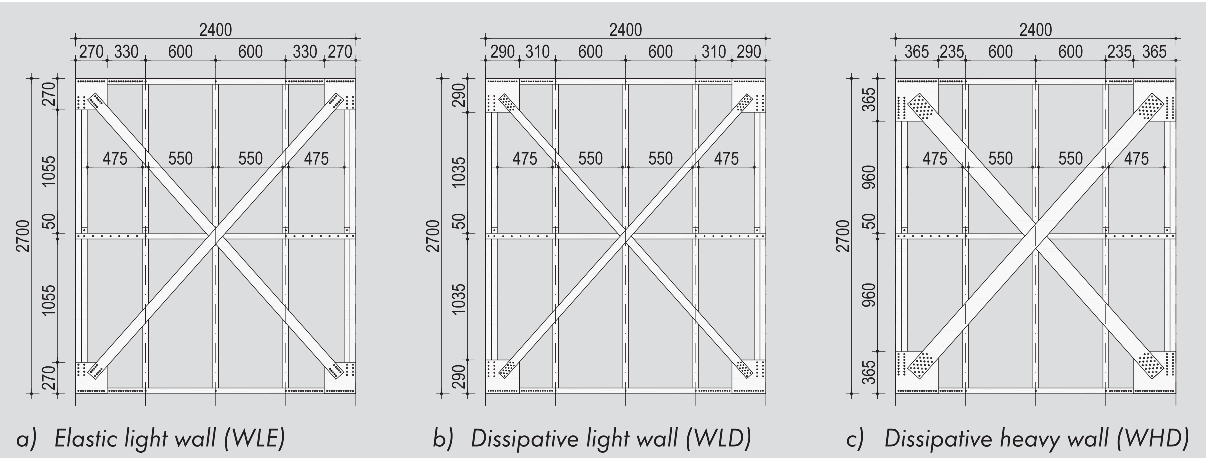


Fig. 5.26: *Schematic drawings of the three wall configurations*

University of Naples Federico II focused on the seismic behaviour of stick-built structures /5.7/. Both X-braced and cladding-braced structures have been under investigation. In particular, sophisticated models for the prediction of the wall behaviour under horizontal loads have been developed, based on the experimental global response of walls and experimental local response of connections (i.e. steel-to-steel connections for X-braced and cladding-to-steel connections for cladding-braced designs). The behaviour factors have been evaluated for both solutions and procedures for the seismic design have been proposed. In the following sections, an overview of the research developed to evaluate the global seismic response of one floor buildings (Section 5.4.2), to model the shear walls (Section 5.4.3) and to propose simplified design procedures for the seismic design (Section 5.4.4) is provided. It is worthwhile underlining that the complexity and extent of the research makes it difficult to be fully included in this book. Therefore, only the main goals and achievements will be presented, referring to the bibliography for further information.

5.4.2. Global seismic response of buildings

The feasibility of using CFS members in seismic zones and the development of design criteria and methodologies have been the subject of previous Italian National Research Projects (PRIN 2001 and PRIN 2003) /5.8/ and are among the main topics developed within the Italian University Network Reluis (Reluis 2010-13). The main objective of the research was the definition of the seismic response of stick-built constructions braced by flat straps in an X-configuration (X-braced solution)

Fig. 5.27: WHD general view and corner detail

Tab. 5.5: Nominal design dimensions and material properties of the tested wall components

	WLE		**WLD**		**WHD**	
	Section (mm)	**Grade**	**Section (mm)**	**Grade**	**Section (mm)**	**Grade**
Studs	C150x50x20x1.5[a]	S350	C150x50x20x1.5[a]	S350	C150x50x20x3.0[a]	S350
Tracks	U153x50x1.5[b]	S350	U153x50x1.5[b]	S350	U153x50x1.5[b]	S350
Diagonal straps	90x1.5[c]	S350	70x2.0[c]	S235	140x2.0[c]	S235
Gusset plates	270x270x1.5[d]	S350	290x290x1.5[d]	S350	365x365x1.5[d]	S350
Track reinforcements	C150x50x20x1.5[a]	S350	C150x50x20x1.5[a]	S350	C150x50x20x3.0[a]	S350
Blocking members	C150x50x20x1.5[a]	S350	C150x50x20x1.5[a]	S350	C150x50x20x3.0[a]	S350
Flat straps	50x1.5[c]	S350	50x1.5[c]	S350	50x1.5[c]	S350

[a] C-section: outside-to-outside web depth x outside-to-outside flange size x outside-to-outside lip size x thickness;
[b] U-section: outside-to-outside web depth x outside-to-outside flange size x thickness;
[c] width x thickness;
[d] height x width x thickness

Tab. 5.6: Adopted connection systems

	WLE	**WLD**	**WHD**
Screws	AB 04 63 040	CI 01 48 016	AB 04 63 040
Shear anchors	M8 class 8.8 bolts spaced at 300 mm on centre		
Hold-down–to-chord stud fasteners	no.4 M16 class 8.8 bolts		
Hold-down–to-steel beam fasteners	M24 class 8.8 bolt rods		

or by cladding boards (cladding-braced solution). In both cases, the most important issues for the seismic performance have been evaluated. In the following, research on X-braced and cladding-braced solutions are presented separately.

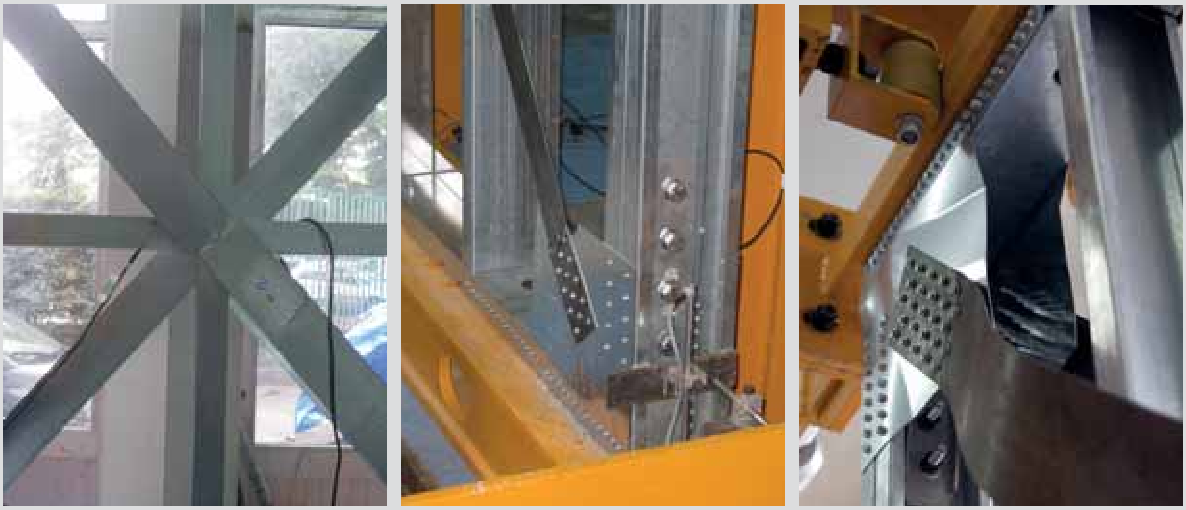
X-braced buildings

In order to investigate a large range of possible CFS solutions for low-rise dwellings, three buildings to be located in different seismic areas were designed. Each of them has a rectangular layout with dimensions

a) *Net section failure for WLE-M1*

b) *Brace yielding for WHD-M1*

Fig. 5.28: Collapse mechanism for the monotonic tests



a) *WLE-C1*

b) *WLD-C1*

c) *Strap yielding registered in WHD-C1*

Fig. 5.29: Collapse mechanisms registered in the cyclic tests: Net cross-section fracture

12.2 m x 18.1 m and an inter-storey height of 3 m. One-storey to three-storey dwellings were designed, and for each of them the main lateral resisting system was defined. Fig. 5.25 shows the schematic plans and elevation developed for the three case studies with indication of the lateral resisting walls. These last case studies are CFS strap-braced stud walls that were designed according to elastic or dissipative design approaches /5.4/. Therefore, three wall configurations were defined as follows: Elastic light (WLE), dissipative light (WLD) and dissipative heavy (WHD) walls (Fig. 5.26). In particular, the WLE typology represents the seismic force resisting system of a single-storey building in low-medium seismic area, in which all wall components were designed according to

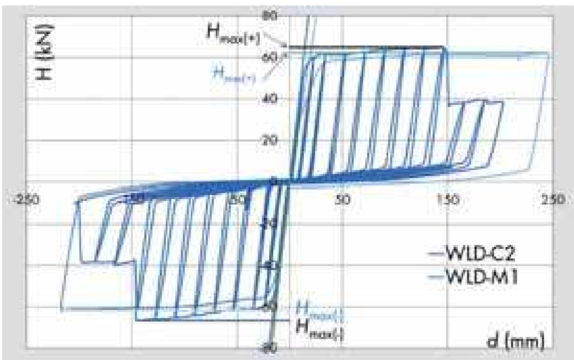


Fig. 5.30: Load vs. displacement curves: Comparison between the monotonic test on WLD-M1 and the cyclic test on WLD-C2

an elastic approach. The WLD wall represents the lateral resisting system of the same building (one-storey located

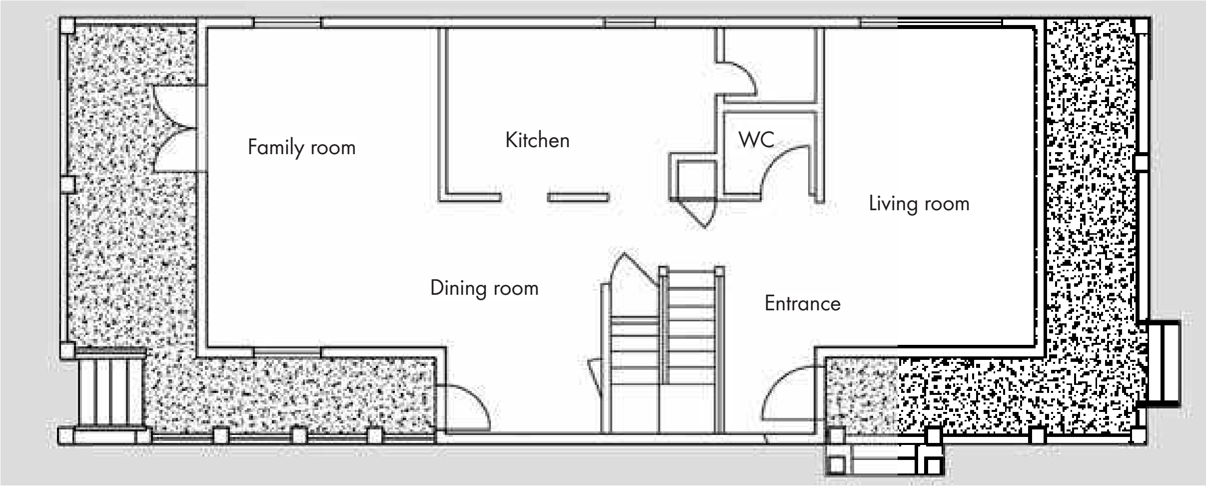


Fig. 5.31: Case study architectural plan

Tab. 5.7: Nominal design dimensions and material properties of the tested wall components

CFS profiles				
	Wall		**Floor**	
	Section (mm)	Grade	Section (mm)	Grade
Studs / Joists	C100x50x10x1.0[a]	S350 GD+Z	C260x40x10x1.5	S350 GD+Z
Tracks	U103x40x1.0[b]	S350 GD+Z	C263x40x1.0	S350 GD+Z
Bearing stiffeners			C263x40x1.0	S350 GD+Z
Cladding				
	Wall		**Floor**	
	Type	Dimensions (mm)	Type	Dimensions (mm)
Outside	OSB/3 Kronoply-3	1250x2500x9.0	OSB/3 Kronoply-3	1250x2500x18.0
Inside	Gypsum based board	1250x2500x12.5		

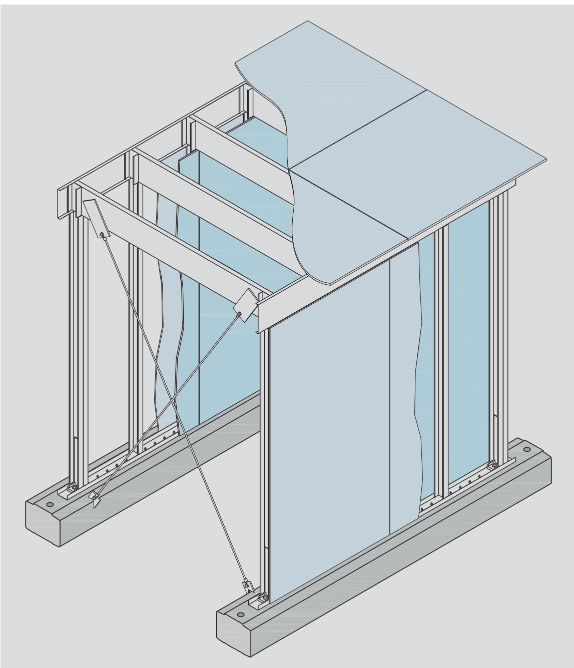


Fig. 5.32: View of the tested specimen

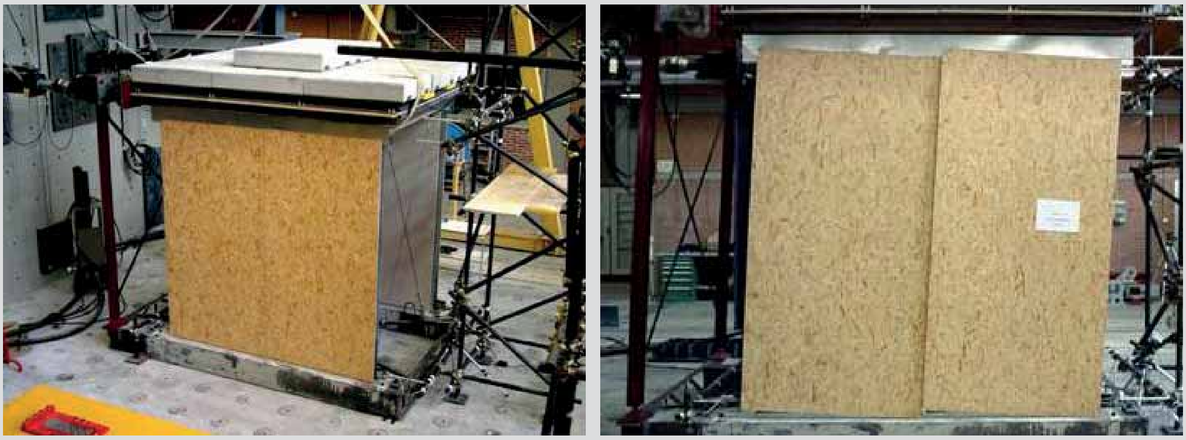
in a low-medium seismic area) but following a capacity design approach. Finally, the WHD wall represents the lateral resisting system of a three-storey building in high-medium seismic area (Fig. 5.27). In particular, the capacity design was adopted in such a way as to ensure a ductile performance by promoting the brace yielding and the overstrength of the other wall components (tracks, studs, gusset plates and hold-down). Tab. 5.5 indicates the nominal design dimensions and material properties of the wall components, and Tab. 5.6 shows the adopted connection systems.

The lateral response of these systems was investigated by testing each of the three selected configurations by two monotonic and two cyclic tests, for a total of twelve tests on full-scale wall specimens /5.9/.

The recorded experimental response was in agreement with the theoretical previsions in terms of both strength and

Tab. 5.8: Adopted connection systems

Connection	**Typology**
Steel-to-steel	Modified truss head 4.2x13 self-drilling screws
Cladding-to-studs	OSB-to-studs: Bugle head 3.5x25 mm spaced at 150 mm along the board edge and spaced at 300 mm on the inner stud GWB-to-studs: Bugle head 4.2x25 mm spaced at 150 mm along the board edge and spaced at 300 mm on the inner stud
Cladding-to-joists	4.2x32 mm spaced at 150 mm along the board edge and spaced at 200 mm on the inner stud
Shear anchors	M8 bolts spaced at 100 mm on centre
Hold-down-to-chord stud fasteners	M6 class 5.6 bolts
Hold-down-to-concrete beam foundation	Episodic chemical anchors with M20 class 8.8 bolt rods



a) Undeformed b) Deformed

Fig. 5.33: Wall specimen

stiffness, as well as in terms of failure modes. In particular, in the monotonic tests, the WLE configurations collapse was reached with the net section failure of diagonal straps (Fig. 5.28a), while the ultimate performance of WLD and WHD specimens was governed by the brace yielding up to the maximum stroke of the actuator without reaching wall failure (Fig. 5.28b). In the cyclic tests, the observed collapse mode was the net section failure of diagonal straps for all the specimens, except for WHD wall specimens, which showed the brace yielding in the pushing phase (Fig. 5.29).

The comparison between the monotonic and cyclic test results reveals that the average experimental shear strength and stiffness values registered under monotonic loads are lower than those recorded in the cyclic tests (Fig. 5.30).

Cladding-braced

The global response of cladding-braced CFS structures was investigated by testing two substructures designed starting from a one-family one-storey dwelling with plan dimensions of 7 x 11 m and a total height of 6 m (one floor plus pitched roof, Fig. 5.31). The assumptions on the main dimensions and unit loads of the structure are shown in Tab. 5.7. The structure was a stick-built construction in which both horizontal (roof and floors) and vertical diaphragms were CFS frames with structural cladding. Two full scale specimens /5.10/ composed of a foundation, two walls and one floor were tested (Fig. 5.32) as a good prototype of typical lateral load resisting systems of house structures. The structures were designed to obtain an elastic response for the seismic design loads. All the structural components (members,

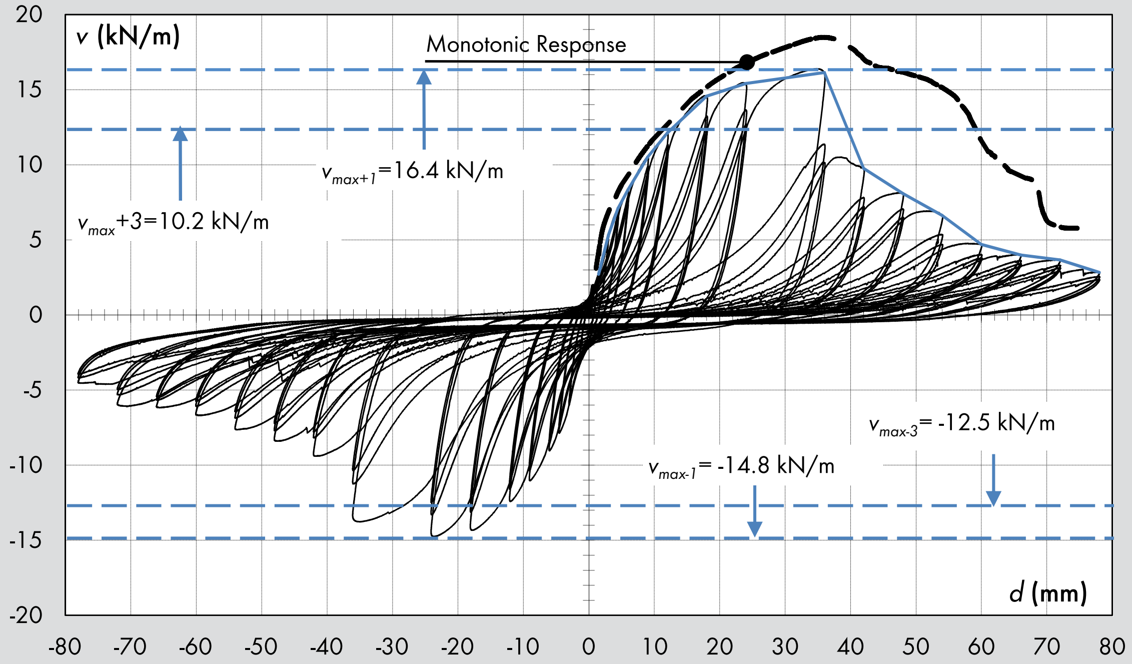


Fig. 5.34: Shear vs. displacement curve

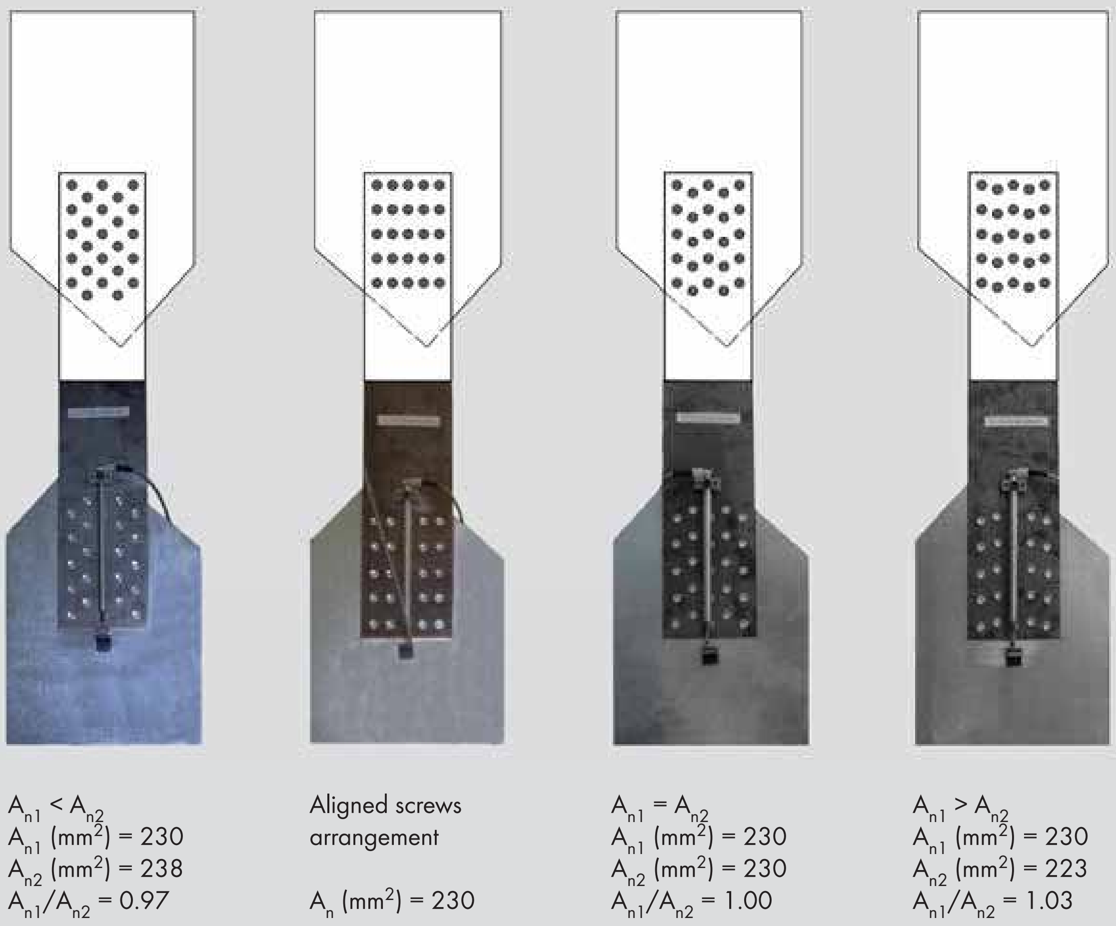


Fig. 5.35: Connection test specimens representative of WHD walls: CHD-1, CHD-2, CHD-3, CHD-4

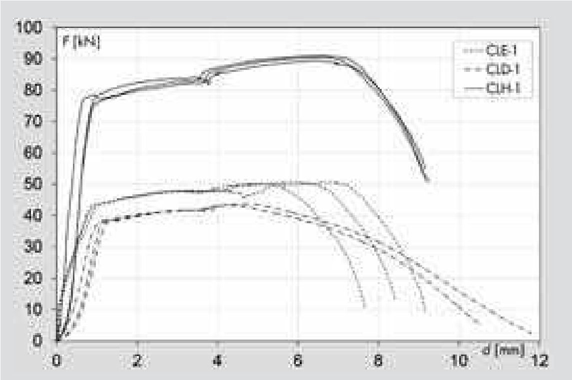


Fig. 5.36: Load (F) vs. displacement (d) curves for CLE-1, CLD-1 and CHD-1

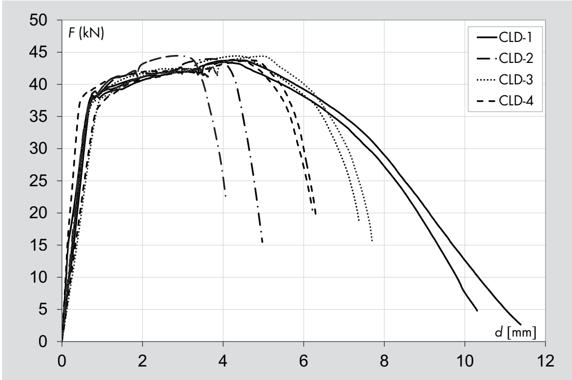


Fig. 5.37: Load (F) vs. displacement (d) curves for CLD-1, CLD-2, CLD-3 and CLD-4

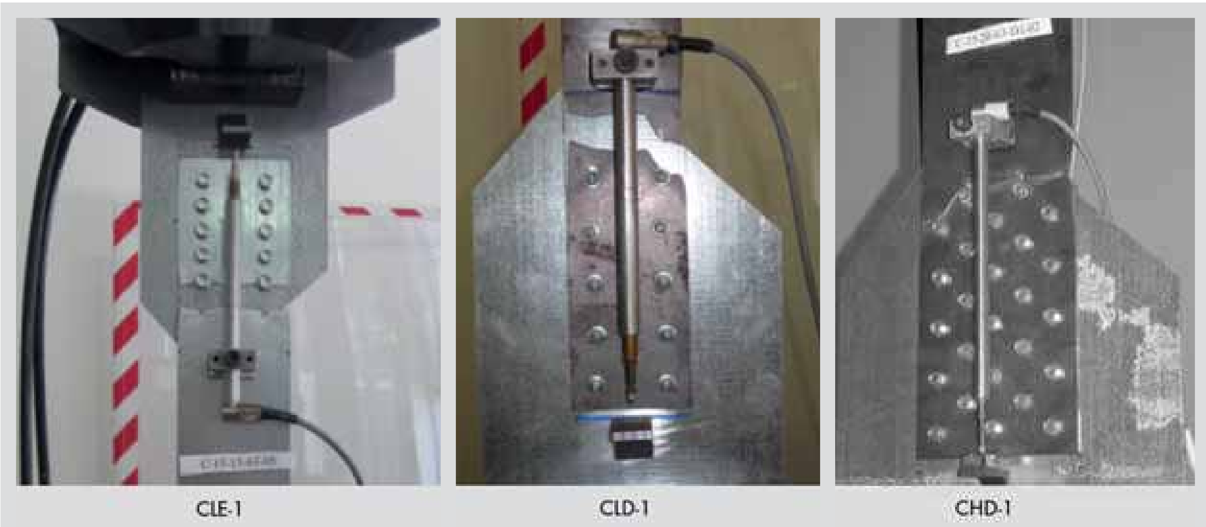
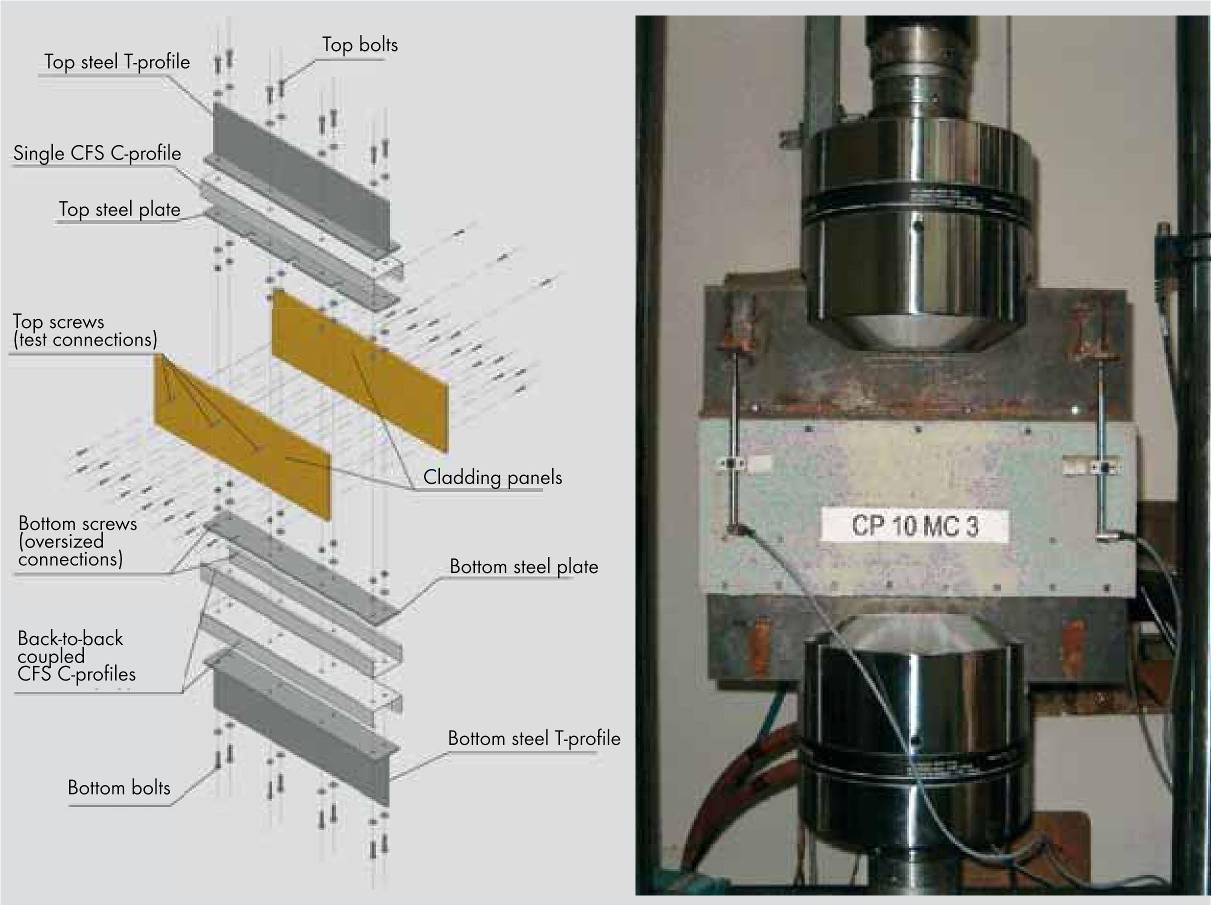


Fig. 5.38: Failure modes for CLE-1, CLD-1 and CHD-1 specimens

boards, and connections other than board-to-wall framing connections, see Tab. 5.8) were designed according to capacity design criteria, in such a way as to promote the development of the full shear strength of the cladding–to–frame connections. Two types of load were applied: Gravity and racking loads. A total gravity load of 45 kN was applied on the floor of the prototype. Horizontal loads were applied to the floor panels by means of two actuators, which were connected to the floor by an adequately designed system able to uniformly distribute the load. This testing apparatus allowed the horizontal floor panels to transmit loads to the vertical walls, which were checked, up to failure of the vertical stud-to-board connections.

The recorded experimental response was in agreement with the theoretical previsions in terms of both strength and stiffness, as well as in terms of failure modes. In particular, the last factor was due to the failure of the connections between studs and cladding, without the occurrence of either stud buckling and deformation of the connections between tracks and floor (Fig. 5.33). Moreover, a good behaviour of shear and tension anchors was recorded. Fig. 5.34 shows a comparison between the monotonic and cyclic response of the specimen.

In order to evaluate the seismic performance, once the seismic capacity was defined by experimental test, the corresponding demand in terms of deformation was investigated /5.11/. In order to study the seismic demand, a theoretical study has been developed in the following phases: Definition and calibration of a model able to capture the hysteretic behaviour; non-linear dynamic seismic analysis of walls subjected to a set of accelerograms adequately identified; design of ad-hoc protocols for cyclic tests.



a) Test set up b) Specimen

Fig. 5.39: Shear test on cladding–to–frame connections

The results obtained by the experimental and numerical phase allowed a comparison of the capacity and requirement under strict conditions. The comparison showed that the CFS systems provide an excellent seismic response when adequately designed. In particular, they showed an elastic behaviour under design earthquakes (earthquakes having a return period of 475 years); the system provides adequate ductility (for overstrength) that assures a good level of safety (limited damages) in case of more severe events (earthquakes having a return period of 475 years).

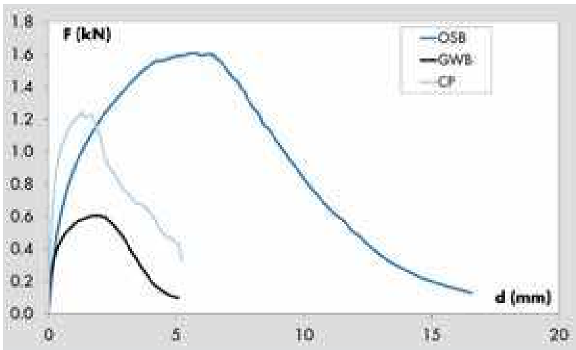


Fig. 5.40: Force – displacement curve for OSB, GWB and CP boards for edge distance equal to 15 mm

5.4.3 Local behaviour of wall components and connections

The experimental studies on the global behaviour of CFS structures demonstrated that the global lateral response of CFS stick-built structures is strongly correlated to the local behaviour of wall components and connections. Therefore, some experimental campaigns were undertaken to investigate the local response of connecting systems of both X-braced and cladding-braced solutions.

X-braced buildings

The global lateral response of CFS strap-braced stud walls and the local behaviour of their components are strongly interrelated. In particular, since the CFS strap-

a) *Breaking of the cladding edge: Insufficient edge distance (10 mm)*

b) *Pull-through cladding and tilting: Adequate edge distance (15 – 20 mm)*

Fig. 5.41: Effect of the loaded edge distance for CP boards

braced stud walls behaviour is influenced by the design of frame-to-strap connections, shear tests on connection prototypes reproducing the joints between gusset and strap-bracing were performed. The behaviour of the connections adopted for the three selected wall configurations (specimens named CLE-1, CLD-1 and CHD-1, corresponding to walls WLE, WLD and WHD, respectively) were investigated /5.10/. Furthermore, three additional connection specimens having different screw layouts were also tested. The additional specimens represent three alternative geometrical screw layouts with respect to the configuration representative of WLD and WHD walls. They are named CLD-2, CLD-3, CLD-4 and CHD-2, CHD-3, CHD-4 (Fig. 5.35), for configuration obtained by changing the screw position with respect to the specimens CLD-1 and CHD-1, respectively.

The experimental results show that the specimens representative of the WHD walls exhibit the best response in terms of strength and stiffness, with average failure load values approximately twice the values obtained for specimens representative of the WLE and WLD walls (Fig. 5.36). Regarding the connection response evaluation for different screw geometrical layouts (Fig. 5.37), the configurations do not have a significant influence in terms of strength and stiffness, but the configurations CLD-1 and CHD-1 have larger deformation capabilities. For all tests, the failure mechanism was screw tilting with subsequent net section failure of the straps (Fig. 5.38).

Cladding-braced

In the case of CFS stud walls with cladding, the global lateral response depends mainly on the shear response of fasteners between CFS frame and cladding boards (cladding fasteners). Therefore, an experimental program for the evaluation of the shear behaviour of cladding fasteners was organized and carried out in two phases /5.12, 5.13, 5.14/: In the first phase, connections between steel profiles and wood (OSB) or gypsum-based (GWB) boards were tested and in the second phase fasteners between profiles and cement-based (CP) boards were tested. The objectives of the testing program were:

- to compare the response of different board typologies (wood, gypsum and cement–based boards)
- to examine the effect of the loaded edge distance
- to evaluate the effect of different cyclic loading protocols
- to study the effect of cladding orientation (only for wood-based boards)
- to assess the effect of the loading rate

A total of 94 specimens (Fig. 5.39), grouped in series composed of 2, 3 or 4, nominally identical specimens were tested.

Acc. to the test results (Fig. 5.40), the cladding material has a significant effect on the shear connection behaviour. In particular, in case of both monotonic and cyclic tests, the CP provides largest stiffness values, the GWB reveals larger ductility, the OSB reveals higher strength and absorbed energy. In the case of OSB boards, the

Tab. 5.9: Behaviour factor for elastic design walls

Test	R_d	R_o	q
WLE-M1	1.74	1.1	2.00
WLE-M2	1.74	1.17	2.04
WLE-C1	1.80	1.21	2.19
WLE-C2	1.73	1.20	2.08

Tab. 5.10: Behaviour factor for dissipative design walls

Test	R_d	R_o	q
WLD-M1	2.6	1.4	3.7
WLD-M2	2.7	1.4	3.9
WLD-C1	2.6	1.5	3.9
WLD-C2	2.9	1.5	4.3
WHD-M1	2.3	1.4	3.1
WHD-M2	2.2	1.4	3.1
WHD-C1	2.3	1.5	3.4
WHD-C2 (Pull)	2.5	1.4	3.6
WHD-C2 (Push)	2.4	1.4	3.4

perpendicular-to-grain loaded connections show lower strength, ductility and stiffness compared with parallel-to-grain loading, while the absorbed energy is almost the same for both cases. Moreover, the increment of the loaded edge distance (Fig. 5.41) produces an increment of strength and absorbed energy with an almost linear variation.

5.4.4. Behaviour factor evaluation

As commonly known, the seismic behaviour of a generic structure can be evaluated by a linear or non-linear analysis. When a linear approach is adopted, the dissipative behaviour of a structure is considered through a "behaviour factor" (q) that reduces the design seismic force as a function of the expected overstrength and ductility levels. In other words, the behaviour factor reflects the capability of a structure to dissipate energy through inelastic behaviour and survive even severe earthquakes without collapse. In particular, the "seismic behaviour factor q" is defined by the EN 1998 as the factor used for design purposes to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures. On the other hand, when a non-linear analysis is adopted, the inelastic response of a structure has to be considered directly, by models and structural analysis that includes mechanical and geometrical non-linearity.

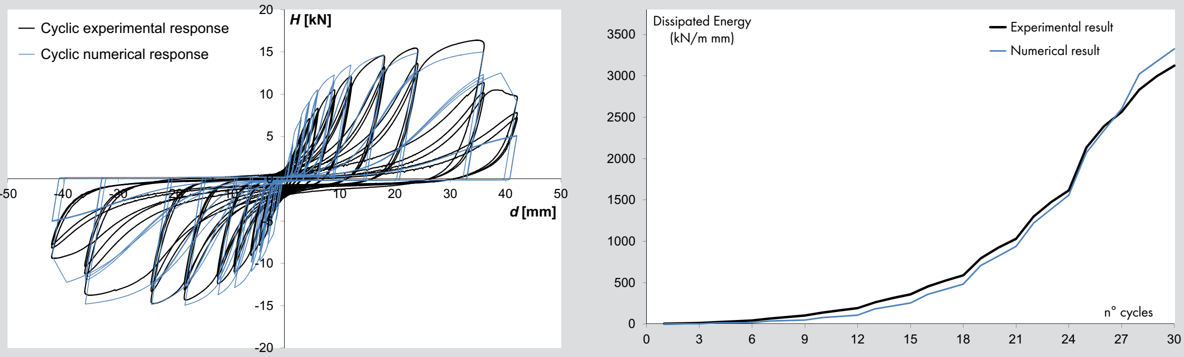
Despite the limits of the linear approach, mainly due to the approximations in the evaluation of q, the undeniable advantages to carry on a linear elastic analysis, make, this last approach the one mainly adopted for the seismic design. Therefore, broad research has been developed to evaluate behaviour factors for all-steel and cladding-braced solutions and the results have been compared with the values provided by the current seismic codes.

X-braced buildings

On the basis of the results of both monotonic and cyclic wall tests, the behaviour factors for each investigated X-braced wall system have been evaluated and then compared with those provided by the AISI S213.

The behaviour factor has been defined as the product of R_d (ductility) and R_o (overstrength) factors, as given in /5.15/. In particular, the ductility-related force modification factor R_d can be evaluated as follows:

$$R_d = \sqrt{2\mu - 1} \text{ with } \mu = \frac{d_{max}}{d_y}$$



a) Global response in terms of horizontal force vs. displacement measured at the top of wall

b) Wall response in terms of dissipated energy

Fig. 5.42: Results in terms of cyclic response and dissipated energy

where μ is the ductility; d_{max} and d_y are the maximum and the conventional elastic limit of the top wall displacement, respectively. The displacement d_{max} can be assumed as the displacement corresponding to an inter-storey-drift of 2 %. The limit 2 % has been defined according to FEMA 356 /5.16/ for traditional concentrically braced structures at the Collapse Prevention limit state. The overstrength-related force modification factor R_o can be evaluated through the formulation provided by Mitchell et al. /5.17/:

$R_o = R_{sd} \cdot R_{\Phi} \cdot R_{yield} \cdot R_{sh}$

where R_{sd} represents the overstrength due to the choices of the structural elements; R_{Φ} accounts for difference between design and nominal yield strength; R_{yield} takes into account the difference between nominal and actual (average) yield strength; R_{sh} accounts for material strain hardening.

Tab. 5.9 and 5.10 show the values of the behaviour factor obtained by the experimental results. These tables show that in the case of elastic design walls (Tab. 5.9) the behaviour factor values proposed by AISI S213 for the conventional construction category (q = 1.6) is always smaller than those experimentally obtained (q = 2.0 ÷ 2.2). As far as dissipative design walls (Tab. 5.10) are concerned, the value provided by AISI S213 in case of Limited ductility braced walls (q = 2.5) represents a lower limit of the obtained behaviour factors (q = 3.0 ÷ 4.3).

Cladding-braced

On the basis of the results of previous research on CFS cladding-braced systems, a constitutive model for cladding-braced CFS walls has been developed /5.18, 5.19/. This model is able to describe the global cyclic response of a cladding-braced CFS wall on the basis of the monotonic response of the screws between cladding and frame. The comparison between the cyclic experimental response and the numerical response, obtained by this constitutive model, provided satisfying results in terms of both cyclic response and dissipated energy (Fig. 5.42).

Once the cyclic constitutive model has been calibrated, the behaviour factors have been evaluated numerically by a wide parametric study. In particular, 72 different wall configurations have been investigated. All the walls considered were made of typical CFS frames with lipped channel-section studs spaced at 600 mm with cladding made of GWB boards on both sides (G+G) or GWB on one side and OSB boards on the other side (G+O). Wall geometry (height and length), CFS members typology, cladding board material, screw typology and spacing have been varied as summarized in Tab. 5.11. For each wall configuration obtained by combining those parameters, the stud thickness and hold-down device typology have been selected, so as to promote the cladding fasteners collapse (capacity design approach).

One-storey buildings, in which both floors and walls are realized with CFS framing with a structural cladding board, have been considered as case studies. In particular, in order to obtain a large range of solutions, a schematic construction has been considered with wall length (L) variable between 3 and 7 m and resisting wall

Tab. 5.11: Assumption for the parametric study

Wall geometry	
Height	2400, 2700, 3000 mm
Length	1200, 2400, 9600 mm
CFS members	
Studs	C 100x50x10 mm (height x width x lip length)
Cladding configurations	
G + G	Gypsum based boards with thickness equal to 12.5 mm on both sides (interior and exterior)
G + O	Gypsum based boards with thickness equal to 12.5 mm on the interior side OSB board with thickness 9.5 mm on the exterior side
Cladding–to–frame fasteners	
Gypsum	Self-tapping screws with bugle head and dimensions: 3.5 x 25 mm Screws spacing along the inner studs: 300 mm Screws spacing along the board edges: 50, 75, 100, 150 mm
OSB	Self-tapping screws with flat head and dimensions: 4.2 x 25 mm Screws spacing along the inner studs: 300 mm Screws spacing along the board edges: 50, 75, 100, 150 mm

Tab. 5.12: Behaviour factors obtained by the parametric analysis /Fiorino et al. 2012b/

Wall configuration		**q**
G + G	Average	3.2
	Standard deviation	1.0
	Coefficient of variation	0.3
G + O	Average	2.9
	Standard deviation	0.6
	Coefficient of variation	0.2
All types (G + G and G + O)	Average	3.0
	Standard deviation	0.9
	Coefficient of variation	0.3

segment with lengths (l) in the range of 0.4L and 0.7L (Fig. 5.43). Unit weights ranging from 0.4 to 1.5 kN/m^2 and from 0.3 to 1.2 kN/m^2 have been considered for floors and walls, respectively. Moreover, buildings with and without attic have been considered.

The assessment of the behaviour factor /5.19, 5.20/ has been achieved by performing a very large number (about 500,000) of non-linear dynamic analyses, in which 21 different European earthquake records for each case study have been considered. The interpretation of the results of this extensive parametric seismic study shows that a reasonable value of the behaviour factor for the analyzed structural system is about 3, as shown in Tab. 5.12.

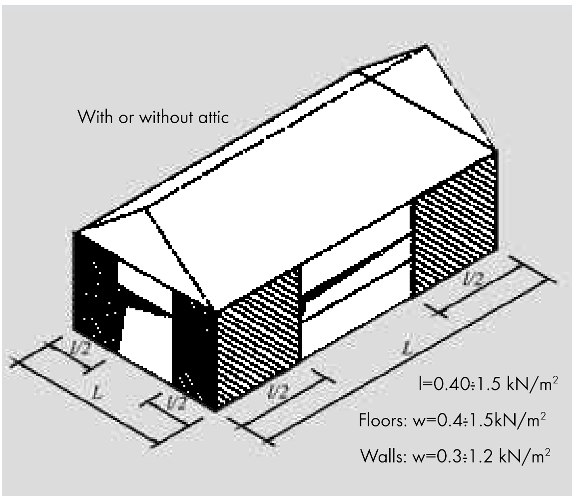


Fig. 5.43: Schematic building for a large range of solutions adopted in the parametric study

Tab. 5.13: Seismic design criteria for X-Braced CFS structures

Requirements	**Traditional steel X-braced structures**	**CFS X-braced structures**
Ductile elements	Tension diagonals	
Cross-section slenderness limits for ductile elements	Class 1 or 2	no
Member slenderness limits for ductile elements	$1.3 \le \bar{\lambda} \le 2.0$ in structures with more than two storeys	no
Member slenderness limits for ductile elements	$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \le N_{u,Rd} = \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2}}$	
Design of connections for dissipative members (local hierarchy criteria)	$R_d \ge 1.1 \gamma_{ov} N_{pl,Rd}$ with $\gamma_{ov} = 1.25$: material overstrength factor	
Deformation capacity of connections	no	$R_{v,Rd} \ge 1.2 R_{b,Rd}$ or $\sum R_{v,Rd} \ge 1.2 N_{u,Rd}$
Design of beams and columns (local hierarchy criteria)	$N_{pl,Rd,bc}(M_{Ed,bc}) \ge N_{Ed,G,bc} + 1.1 \cdot \gamma_{ov} \cdot \Omega_{min} \cdot N_{Ed,E,bc}$	$N_{pl,Rd,bc}(M_{Ed,bc}) \ge N_{Ed,G,bc} + 1.1 \cdot \gamma_{ov} \cdot \Omega_i \cdot N_{ED,E,bc}$
Global mechanism promotion (global hierarchy criteria)	$\frac{\Omega_{max} - \Omega_{min}}{\Omega_{min}} \le 0.25$	no

$\bar{\lambda}$:	non-dimensional member slenderness
$N_{pl,Rd}$:	design yield resistance of the gross cross-section of the diagonal
$N_{u,Rd}$:	design net area resistance of the diagonal
$N_{pl,Rd,i}$:	design yield resistance of diagonal i
$N_{Ed,i}$:	design value of the axial force in the diagonal i due to seismic loads
$N_{Ed,E,bc}$:	the design axial forces in the beam or column due to seismic loads
$M_{Ed,bc}$:	the design bending moment in the beam or column
R_d:	design resistance of the connection
$R_{v,Rd}$:	design shear resistance of the screw
$R_{b,Rd}$:	design bearing resistance of the connection
$N_{pl,Rd,bc}(M_{Ed,bc})$:	is the design plastic resistance of the beam or column evaluated by considering the interaction with the bending moment $M_{Ed,bc}$
A:	gross cross-section area
A_{net}:	net area of the cross section at the fasteners holes
f_y:	characteristic yield strength
f_u:	characteristic ultimate strength
$\Omega_i = N_{pl,Rd,i} / N_{Ed,i}$:	overstrength factor of diagonal i
Ω_{max}:	is the maximum value of the overstrength factor evaluated for each diagonal
Ω_{min}:	is the minimum value of the overstrength factor evaluated for each diagonal
$\gamma_{M0} = 1.0$:	partial safety factor for yielding resistance of gross cross-section
$\gamma_{M2} = 1.25$:	partial safety factor for the tensile resistance of net sections

5.4.5. Seismic design guidelines and procedure

Once the global and the local behaviour of CFS systems has been evaluated and the behaviour factors have been defined for both X-braced and cladding-braced solutions, guidelines and procedures for the seismic design can be proposed as presented in the following sections.

Guidelines for the design of X-braced systems

Recent studies /5.21, 5.22, 5.23/ have shown that for X-braced CFS structures, design guidelines compatible to those provided by EN 1998-1 for traditional concentrically X-braced steel frames can be adopted if specific prescriptions are introduced. In fact, as it is well known, the adoption of a given behaviour factor can only be done if the design requirements have been

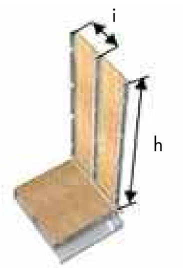
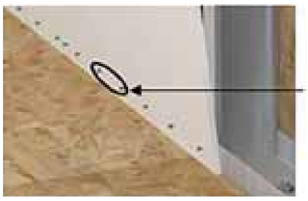
Step 1	Geometry		• Height (h) • Framing	→ Stud spacing
Step 1	Wall components		• Cladding • Framing • Connections • Hold-down anchors • Shear anchors	→ Typology → Thickness → Orientation → Steel grade → Stud dimensions → Track dimensions → Typology → Internal spacing → Edge distance → Typology → Typology
Step 2	Wall components		• Connections	→ External spacing (s)
Step 3	Wall components		• Framing • Shear anchors • Hold-down anchors	→ Studs thickness → Track thickness → Spacing → Diameter

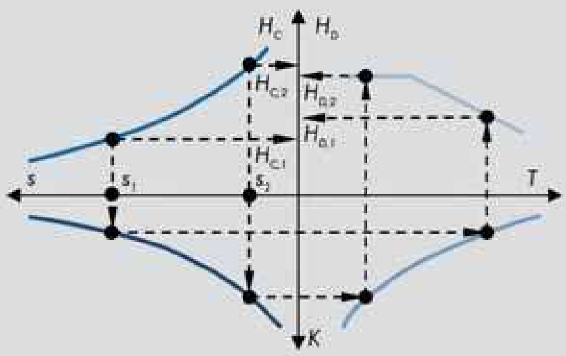
Fig. 5.44: Scheme of the proposed design procedure

defined, i.e., rules that should be used to proportion and detail the system and limits in its application. Tab. 5.13 shows a comparison of design requirements given in EN 1998-1 for traditional steel X-braced structures and those proposed by authors for CFS X-braced seismic lateral resisting systems.

For both traditional and CFS X-braced structures, the ductile elements are represented by the tension diagonals, and the ductile mechanism is identified by the simultaneous yielding of all tensile diagonals by avoiding net section brittle failure.

No limits for dissipative elements in terms of both local (cross-section) and global (member) slenderness are needed in case of CFS X-braced systems. In fact, for the common CFS solutions, strap diagonals are usually used. Concerning the local hierarchy criteria, the same rules used for traditional X-braced structures can be used also in the case of strap-braced CFS walls, i.e., for designing non-dissipative elements (connections, studs, tracks, anchorages), a material overstrength factor equal to 1.25 can be recommended. In addition, a specific recommendation adopted for CFS structures and prescribed by EN 1993-1-3 devoted to provide an adequate deformation capacity of connections typically should be used for X-braced CFS systems.

Since the maximum number of storeys of CFS buildings is generally limited to 3, the global mechanism can be promoted by proportioning all non-dissipative elements



s: Cladding fastener spacing along the board edges
K: Wall stiffness
T: Vibration period
H_D: Seismic demand
H_C: Wall capacity under horizontal load

If $s=s_1$ $H_C=H_{C,1}<H_D=H_{D,1}$ is not acceptable.
if $s=s_2$, there is a smaller cladding fastener spacing ($s_2<s_1$), and then, as result, $H_C=H_{C,2}>H_D=H_{D,2}$ is acceptable.

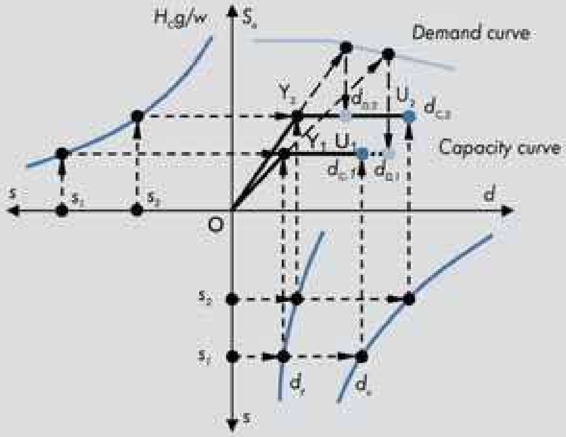
Fig. 5.45: Nomograph for linear dynamic analysis

under the assumption of simultaneous yielding of all tensile diagonals (global hierarchy criteria).

Design procedures for cladding-braced systems

Starting from the results obtained through previous research on cladding-braced systems, some design procedures devoted to the seismic design of CFS walls with cladding made of OSB or gypsum based boards have been developed. In agreement with the hypothesis adopted in the previous studies, the developed methodology is based on the idea that the optimum wall response is reached when the global response significantly depends on connections between steel members and boards. Moreover, the procedure presented hereafter, can be exclusively applied to floor dwellings that can be schematized as systems with a single degree of freedom.

The procedure /5.18, 5.24/ is organized into three consecutive phases (Fig. 5.44): It starts with the preliminary definition of the wall geometry and material (phase 1), then the cladding fastener spacing is defined by a simplified seismic analysis (phase 2) and, finally,

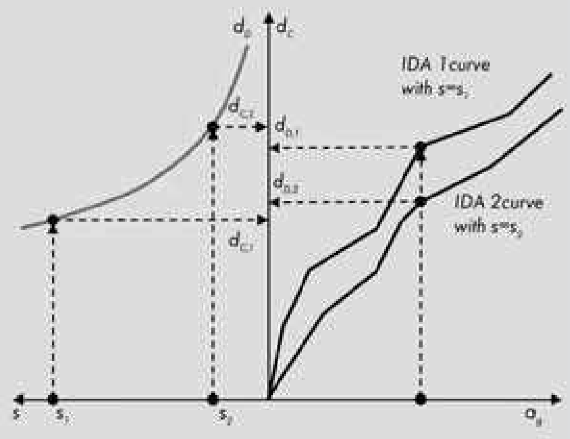


s: Cladding fastener spacing along the board edges
d: Lateral displacement at the wall top location
d_y: Elastic limit displacement
d_u: Ultimate limit displacement
S_a: Elastic spectral accelleration
H_C g/w: Acceleration corresponding to the wall strength
H_C: Wall strength under horizontal load
g: Gravity accelleration
w: Seismic weight
d_c: Wall displacement capacity (for example $d_c=d_u$)
d_D: Wall displacement demand

O-Y_1-U_1 capacity curve for $s = s_1$
O-Y_2-U_2 capacity curve for $s = s_2$
For $s=s_1$, the capacity curve O-Y_1-U is obtained. The seismic displacement demand is equal to $d_{D,1}$ and the displacement capacity is equal to $d_{C,1}$. In this case $d_C=d_{C,1}<d_D=d_{D,1}$ is not acceptable.
For $s=s_2$, the cladding fastener spacing becomes smaller ($s_2<s_1$), and the O-Y_2-U_2 capacity curve is obtained. The required displacement is $d_{D,2}$ and the displacement capacity is equal to $d_{C,2}$. Since $d_C=d_{C,2}>d_D=d_{D,2}$, the solution is acceptable.

Fig. 5.46: Non-linear static nomograph

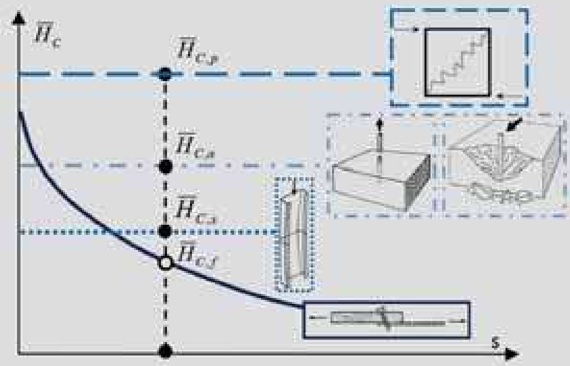
all the other wall structural components are defined according to a capacity design approach (phase 3). In particular, in the first phase, the wall height, stud spacing and some features of the main structural components (i.e. steel grade, dimensions of tracks and studs, cladding typology and thickness, screw and anchor typology) are defined. This selection usually depends directly on the structural design of the walls under vertical loads, as well as on architectural and technological reasons. The seismic analysis developed in the second phase is the core of the procedure. In fact, in the second phase, the cladding



s: Cladding fastener spacing along the board edges
a_g: Peak ground acceleration
d_c: Displacement wall capacity (for example d_c=d_u)
d_D: Wall displacement demand

For s=s_1, the IDA 1 curve is obtained. The required displacement is $d_{D,1}$ and the displacement capacity is equal to $d_{C,1}$. Since $d_C=d_{C,1}<d_D=d_{D,1}$ the solution is not acceptable.
For s=s_2, the cladding fastener spacing becomes smaller ($s_2<s_1$), and IDA 2 curve is obtained. The displacement demand is $d_{D,2}$ and the displacement capacity is $d_{C,2}$. Since $d_C=d_{C,2}>d_D=d_{D,2}$ the solution is acceptable.

Fig. 5.47: Nomograph for non-linear dynamic analysis



$\overline{H}_C$: Lateral strength wall capacity
$\overline{H}_{C,p}$: Lateral strength capacity associated to the board failure
$\overline{H}_{C,a}$: Lateral strength capacity associated to the anchors failure $\overline{H}_{C,s}$: lateral strength capacity associated to stud failure
$\overline{H}_{C,f}$: Lateral strength capacity associated to the cladding fasteners failure
s: Cladding fastener spacing along the board edges

Note: all the lateral strengths $\overline{H}_C$ are for unit wall length ($\overline{H}_C$ = H_C/L)

Fig. 5.48: Nomograph for the verification according to the capacity design criteria

fastener spacing (s) is defined by means of a simplified seismic analysis developed with graphic nomographs. In particular, three different alternative nomographs have been proposed. Each of them corresponds to a different alternative seismic analysis (i.e. linear dynamic, non-linear static or non-linear dynamic). Therefore, for each chosen seismic analysis, the corresponding nomograph allows the fastener spacing (s) to be defined.

When the linear dynamic (LD) procedure is selected for the seismic analysis, the corresponding nomograph (Fig. 5.45) allows the comparison between seismic capacity (H_C) and seismic demand (H_D) in terms of force (force-based approach). In particular, starting with an assigned screw spacing (s), it is possible, on one side, to define the lateral wall capacity (H_C), and on the other side, to evaluate in three consecutive steps the following parameters: Stiffness (K), vibration period (T) and seismic demand (H_D). The procedure can be considered concluded when, for a defined screw spacing (s) the capacity will not be smaller than the demand ($H_C \geq H_D$).

When the seismic analysis is performed by means of the non-linear static (NS) procedure, the inelastic behaviour and the structural overstrength are directly considered and the comparison between seismic capacity and demand can be achieved in terms of displacements (displacement-based approach). The corresponding nomograph facilitates obtaining, for each screw spacing (s), the wall strength (H_C) and the yield (d_y) and ultimate displacement (d_u), and then to define the capacity curve (bilinear O-Y-U curve in Fig. 5.46). Starting from the capacity curve and from the demand curve, defined by the acceleration-displacement spectrum, it is possible to define the spacing in such a way that the maximum allowable wall displacement capacity is higher than the seismic displacement demand ($d_C \geq d_D$).

The non–linear dynamic analysis represents the most

Fig. 5.49: The school complex and its external area

advanced approach to define the seismic performance of a structural system. Due to the complexity of the approach connected to the definition of the hysteretic constitutive model and the accelerogram selection, this analysis is usually adopted only for research purposes. Nevertheless, the large number of results obtained by the wide parametric seismic analysis presented in the previous section, allows development of a specific nomograph for non-linear dynamic analysis (Fig. 5.47). As in the case of non-linear static analysis, a displacement-based method is adopted. Therefore, once the screw spacing is defined, the wall displacement capacity, on one side, and the seismic displacement demand on the base of the peak ground acceleration, on the other side, can be evaluated.

When the screw spacing is defined by one of the previous analyses, the wall design can be completed defining the other wall components in such a way that the seismic verification is satisfied. In other words, the dimensions of the other wall components should guarantee that the wall collapse is associated to the cladding fasteners failure (phase 3). To this end, a specific nomograph (Fig. 5.48) has been developed, in which the wall shear strengths per wall unit length ($\overline{H}_c$) corresponding to the resistance of cladding fasteners ($\overline{H}_{C,f}$), chord studs ($\overline{H}_{C,s}$), anchors ($\overline{H}_{C,a}$) and boards ($\overline{H}_{C,p}$) are represented together as a function of the exterior spacing (s).

In conclusion, the described simplified procedure permits the study of complex problems such as the seismic design of CFS walls with cladding in an easy way. Obviously, this procedure has to be considered as a preliminary tool that has to be followed up by a complete seismic design.

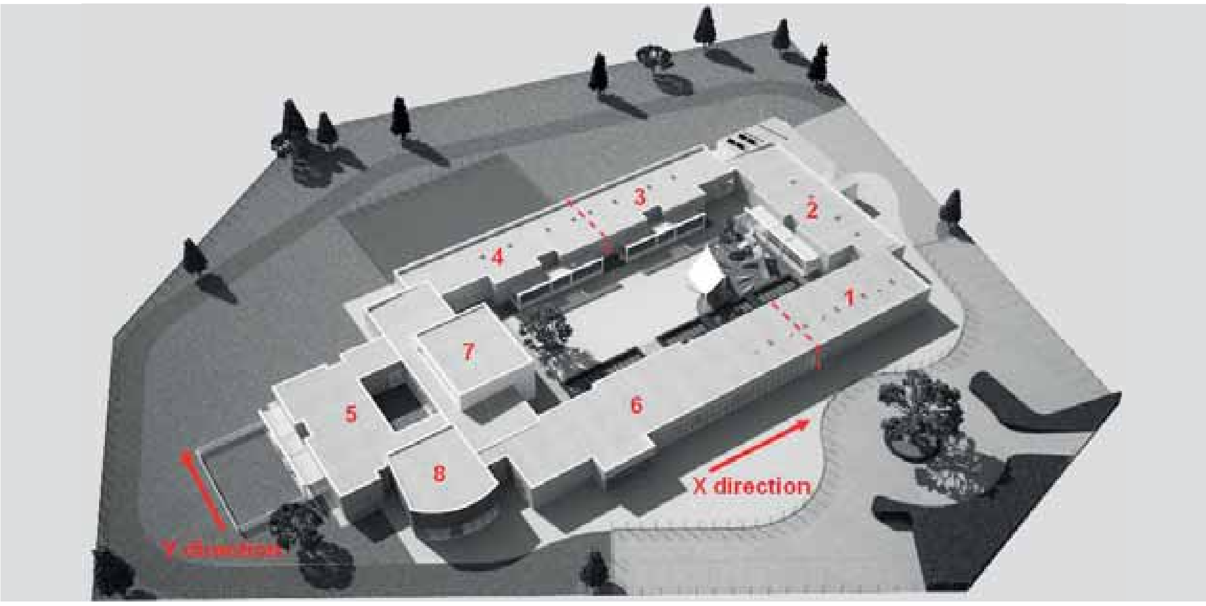


Fig. 5.50: Block distribution

Fig. 5.51: Foundation

5.5. Design and execution of an Italian CFS structure: The BFS school

5.5.1. General

The results of the described scientific studies developed at the University of Naples Federico II were adopted in several different design applications. Among them, an emblematic example for dimension and complexity is represented by the design and execution of a kindergarten and primary school for the British Command of Defense Estate in Naples. The building is a dry construction, realized with CFS members, adopted to realize both standard steel buildings and stick-built constructions. The adoption of these systems permited realization of the buildings characterized by: High quality of products,

Fig. 5.52: Foundation

elevated structural performance and durability, short execution time, large adoption of eco-friendly materials, good integration of thermal, hydraulic and electrical systems and high degree of flexibility. In order to assure these special characterisations, integration of the expertise of eleven designers coming from different fields was required during the entire design and realization process. The building received the ACAI 2011 award with the following motivation: "Work of great interest in the architectural world that emphasizes the constructive system made by CFS profiles. Compared to the traditional steel systems, the realized construction provides a number of indisputable advantages such as: Lightweight, low cost, simplicity of construction and sustainability."

5.5.2 Conceptual design

The choice of the construction typology was strongly motivated by the requirements of the client and by the characteristics of the site. The need for a fast solution that could also eventually be disassembled and reassembled elsewhere, drove the choice towards dry layered solutions, which could allow the reuse of components. On the other hand, the priority to have a safe structure from a seismic point of view, also motivated by damage inflicted by the seismic events that occurred in recent years in central Italy, required the use of a structurally reliable solution. Finally, the difficulty to access the construction site, which would not permit the passage of large trucks, finally identified the solution in CFS as optimal, by combining high structural performance with reduced execution time and meeting the strict logistic requirements.

The school building covers an area of about 3000 m^2 and is divided into eight joined constructions (Fig. 5.49 and Fig. 5.50). The structures of the first six constructions (buildings 1 to 6) are used as classrooms and are made of CFS members braced with structural wood-based cladding boards (stick-built structures with cladding), while the buildings 7 and 8, devoted to multi-purpose use and music hall, respectively, are traditional CFS framed structures.

The foundations (Fig. 5.51 and 5.52), which also represent the ground floor, are made of a reinforced concrete stiffened plate, in which the beams have a height of 800 mm and a width ranging from 500 to 1500 mm. The free fields between the foundation beams have been completed with an aerated structural floor system made of recycled plastic domes with a reinforced concrete layer. This solution provided a flat surface for the assembly of the walls and easy passage for the equipment.

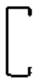
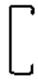
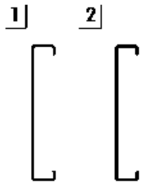
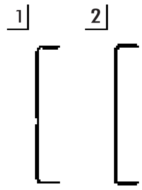
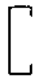
The six buildings with stick-built structure (blocks 1 to 6) are characterized by load-bearing walls with a height of about 4 m and a top flat floor covered by a pitched

Fig. 5.53: Wall system

Fig. 5.54: Roof system

ventilated roof. This last is made of corrugated sheet with multilayer protection and supported by galvanized steel studs telescopically adjustable to realize the different slopes. The load-bearing structures of the buildings consist of floors and walls made of CFS profiles, with a thickness variable between 1.5 and 3.0 mm and OSB/3 boards with a thickness of 9 mm for the walls and 18 mm for the floors.

In particular, the walls (Fig. 5.53) are made with studs having 150×50×20×1.50 mm (outside-to-outside web

Wall		
Studs	Wall tracks	Wall blocking
 C-Section 150x50x20x1.5 mm Web x flange x stiffness x thickness r = 2.25 mm	 U-Section 153x50x1.5 mm Web x flange x stiffness r = 2.25 mm	 C-Section 150x50x20x1.5 mm Web x flange x stiffness x thickness r = 2.25 mm
Floor		
Joists	Floor tracks	Web stiffners
 1. C-Section 1.5 300x50x20x1.5 mm Web x flange x stiffness x thickness r = 2.25 mm 2. C-Section 3.0 300x50x20x3.0 mm Web x flange x stiffness x thickness r = 4.5 mm	 1. U-Section 1.5 303x50x1.5 mm Web x flange x thickness r = 2.25 mm 2. U-Section 3.0 306x50x3.0 mm Web x flange x thickness r = 4.5 mm	 C-Section 150x50x20x1.5 mm Web x flange x stiffness x thickness r = 2.25 mm

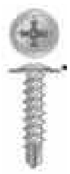
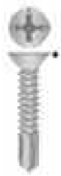
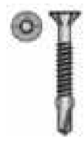
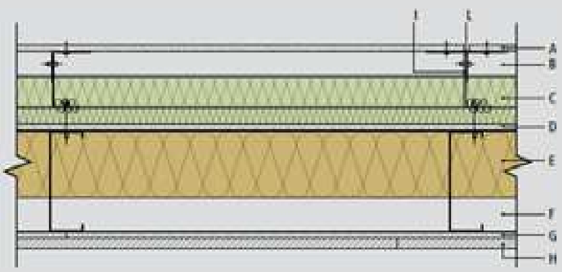
Connections			
 "CI 01 48" By TECFI S.p.A.	 "AB 04 63" By TECFI S.p.A	 "CH 01 42" By TECFI S.p.A.	 "HZ 01 55" By TECFI S.p.A.

Fig. 5.55: Abacus of steel members and screws adopted in the stick-built buildings

depth × outside-to-outside flange size × outside-to-outside lip size × thickness) lipped channel sections spaced at 600 mm on the centre. The track are realized with 152x40x1.5 mm (outside-to-outside web depth × outside-to-outside flange size × thickness) U sections. In order to reduce the in plane global buckling length, wall blocking having the same section of the studs are installed at mid-wall height and flat straps having a width of 50 mm and thickness of 1.5 mm are fastened to the external faces of both flanges of each joist to ensure continuity between studs and blocks. The steel frames have a cladding made of vertically oriented 9.0 mm thick OSB/3 boards on both faces. Cladding boards are connected to steel framing by 4.2 mm diameter bugle-head self-drilling screws spaced at 100 mm at the perimeter and 300 mm in the field. In order to avoid buckling phenomena and any wall overturning, back-to-back coupled studs and purposely designed hold-down devices are placed at the ends of



A: *Fibre-cement board type SWISSPEARL*
B: *Air cavity (t = 50 mm)*
C: *2 layers of insulation boards Celenit N50 + Celenit N25)*
D: *OSB Type 3 board (t = 9 mm)*
E: *Insulation panel Celenit LC/30 (t = 120 mm)*
F: *Air cavity*
G: *OSB Type 3 board (t = 9 mm)*
H: *1 layer of fibre-gypsum board type Vidifire by Knauf (t =18 mm)*
I: *Aluminium angle profile*
L: *Aluminium stud*

Fig. 5.56: Technical solution adopted for the external walls

Fig. 5.57: CFS frame construction of stick-built structures

Fig. 5.58: Cladding boards fixing

Fig. 5.59: Completed building

Fig. 5.60: Shear test on walls

each shear wall segment. In particular, the hold-down devices have been designed for this construction and they are made with S700 steel grade (f_y = 700 MPa and f_u = 750 MPa) and are connected to studs by four M16 8.8 class bolts and to the foundation by one HIT-RE500 with HAS-(5.8) – M24 adhesive-bonded anchors by HILTI (2012 /5.25/). In addition, tracks are connected to the foundation with HST-R-M8 mechanical anchors by HILTI spaced at 200 mm on centre.

The structures of the top flat floor (Fig. 5.54) are made of 300×50×20 mm joists with a thickness ranging from 1.5 mm to 3 mm, depending on the span, and placed in line with the studs. Web stiffeners having a 150x50x10 mm C-shaped section and a thickness equal to that of the connected joist are installed at both joists ends in order to strengthen the member against web crippling. To provide adequate bracing against the lateral-torsional buckling of joists, 250x50x20x1.5 mm C-shaped members (floor blocking) spaced at 1200 or 2400 mm on the centre are installed perpendicular to the joists, and flat straps having a width of 50 mm and a thickness of 1.5 mm are fastened to the bottom side of each joist to ensure continuity between joists and blocks. The subfloor cladding is made of 18 mm thick OSB/3 boards.

Fig. 5.55 shows all the steel members and screws adopted in the stick built buildings.

The definition of the functional "packets" and technological choices for both walls and floors were intended to ensure the eco-efficiency of the building for its life-cycle, starting from the use of eco-compatible materials with low embodied energy to the choice of energetically efficient envelope solutions. Moreover, the internal finishing was chosen in order to withstand fire safety requirements. For these reasons, gypsum fibre boards were adopted as internal finishing (Fig. 5.56). The selected solutions were developed in order to maximize performances related to the main demands of comfort and safety of the users.

5.5.3 Structural design

The structure has been designed compliant to the Italian national code /5.26/, which is very similar to the EN 1993 – Part 1.3, regarding the CFS members

a) Shear tests on OSB boards
b) Shear tests on screws
c) Tension test on hold- down devices
d) Shear test on cladding-to-frame connections

Fig. 5.61: Laboratory tests

and boards. The seismic loads have been evaluated according to Section 3.2 of the Italian national code. The seismic design of the six stick-built buildings has been carried out as a cladding-braced approach. In particular, the horizontal loads have been evaluated following the segment method approach so that only full-height walls are effective as load bearing wall segments, i.e. without openings. For the seismic action distribution, floors were considered as in-plane rigid diaphragms. Taking into account that no specific regulations for the seismic design of CFS structures are provided by the Italian code, an elastic seismic design has been developed. Therefore, a behaviour factor equal to 1 has been considered.

The seismic verification at the ultimate limit state (ULS) has been carried out comparing, for each wall segment, the maximum horizontal load demand (H_D) with the lateral strength capacity (H_c), according to the well-known equation:

$H_D/H_C \leq 1$

in which the capacity corresponds to the weakest collapse mechanism among all the component failures: Chords studs ($H_{C,s}$), cladding-to-frame connections ($H_{C,f}$), cladding boards ($H_{C,p}$) and frame-to-foundation anchors. Therefore, even if it was not necessary to satisfy any capacity design criteria, because the seismic design was carried out under the assumption of elastic behaviour, the

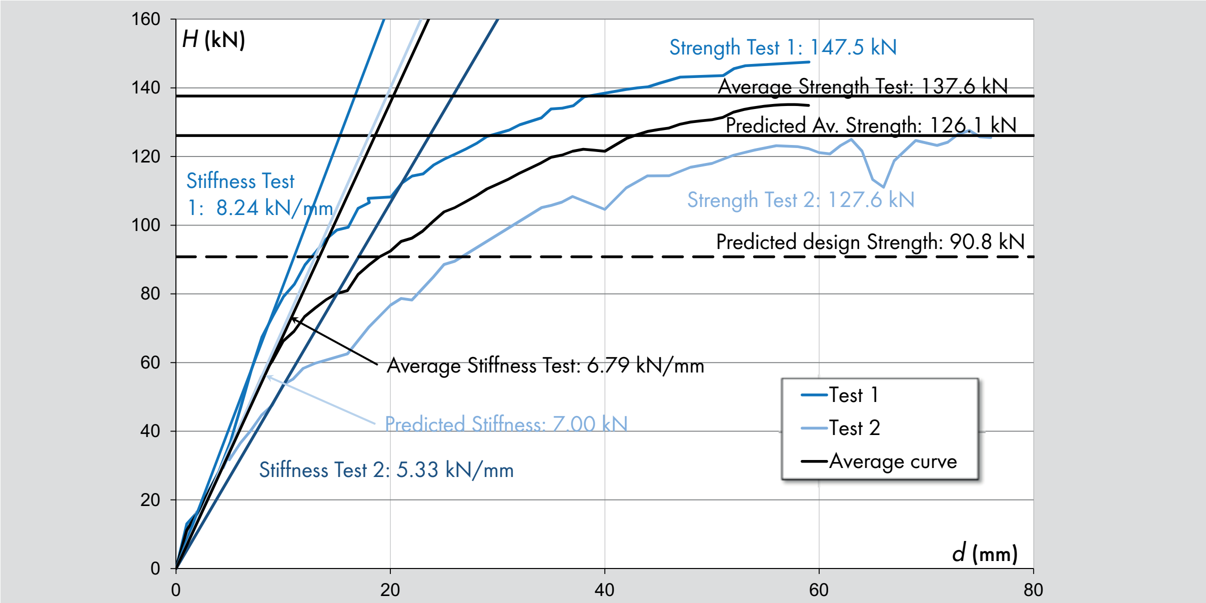


Fig. 5.62: Wall response: Comparison between experimental results and theoretical evaluations

walls were designed to guarantee that the minimum lateral strength was equal to that associated to the cladding-to-frame connections collapse ($H_{C,f}$). More details of the seismic design are provided in /5.27/.

5.5.4 Execution

The whole complex has been realized in about one year and, in particular, the CFS structures have been erected in six months. The main construction phases are shown in Fig. 5.57 to Fig. 5.59. A total of about 160,000 kg of steel was used for the steelwork, with an incidence of approximately 50 kg/m^2. In the period of larger constructive activities, over 50 people worked at the same time. This means that lightness, modularity and dry constructive systems, which are among the greatest features of these systems, may allow high on-site productivity comparable to typical levels of industrial contexts. Therefore, an important feature of this construction was to realize a non-conventional complex in short time, with an elevated level of detail.

5.5.5 On-site verifications

The construction represents a "unicum" in the Italian construction sector. It adopts a system that is not covered by the Italian national code. Therefore, it has been designed on the basis of the research results obtained at the University of Naples. On the other hand, during the execution, a cumbersome and expensive verification process was required. Indeed, in addition to ordinary on site tests, the seismic performance at the global and local scale had to be verified during the construction phase.

The testing programme included two tests on full scale walls (Fig. 5.60) and a large number of components tests (Fig. 5.61). In particular, in order to investigate the global response, full scale tests on two identical walls 4.80 m long and 3.95 m high have been carried out. At the component scale, the following tests were performed: 40 shear tests on screws, 30 shear tests on OSB boards, 44 tests on cladding-to-frame connections and 10 tests on hold-down devices. More details of the experimental campaign are provided in Iuorio et al. 2014c /5.22/.

The wall tests showed that the response, under vertical and horizontal loads, of shear walls with non-common geometry (height of about 4 m) confirmed the design calculations with accurate prediction of the response in terms of strength and stiffness (Fig. 5.62). At the local scale, the tests on materials and components provided a large experimental characterization of the main mechanical properties and demonstrated that the design assumptions were reasonable.

The presented experience could be considered as a milestone for updating the Italian code and represents a case study of great relevance for size and design solutions, with reference to the Italian construction market.

6 Building physics fundamentals

Georg Krämer

Beyond the mechanical performance, building physical issues are an inherent part of the state-of-the-art integral approach of the engineering and design of lightweight steel constructions.
Fire protection, sound insulation and thermal insulation are fundamental properties facilitating the application diversity of drywall constructions. The following chapter gives the basics for these topics and shows the high performance that can be provided by lightweight steel constructions.
One of the greatest accompanying hazards of earthquakes is fire, which makes fire protection a vitally important issue with drywall systems. The following sections deal with these issues as well as other integral topics of building physics as fundamentals in the dimensioning of constructions with reference to the European standards (EN).

6.1 Fire protection

Alongside preventative fire protection (e.g. fire protection organisation, warning, extinguishing and safety equipment, fire services), in building engineering building related fire protection by

- Selection of suitable building materials (flammability)
- Spatial segregation with fire-resistant building elements
- Encapsulation of load bearing constructions for retention of the stability

are decisive in "preventing a fire outbreak and stopping fire and smoke from spreading, and to enable the rescue of people and animals and facilitate effective firefighting".
Derived from this, for components regarding their building regulation technical fire protection classification, the building material classes of the materials employed (flammability) as well as the fire resistance for the constructional component are the decisive fire resistance parameters.

6.1.1 Reaction to fire of building materials and building products (Building material classes)

European standards
According to the European standard EN 13501-1, the classification of the reaction to fire of building materials / products are categorized into 6 classes from A – F (Tab. 6.3).
The main classification criteria are

- Ignitability
- Spread of flame
- Emitted heat

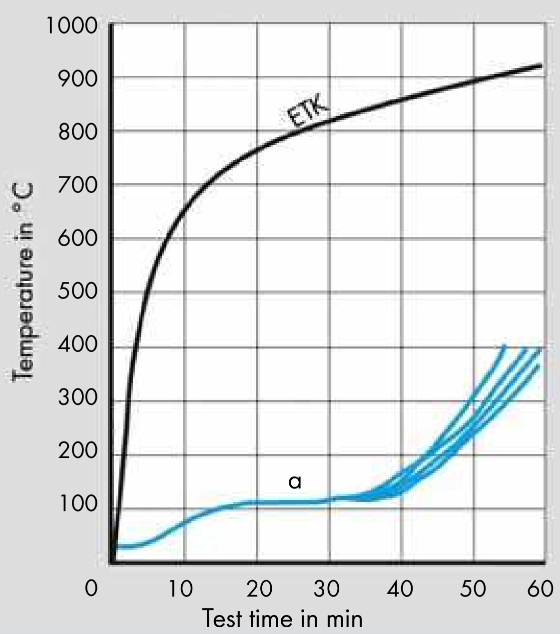
Furthermore, the parallel fire phenomena

- Smoke production with s1, s2 and s3
- Flaming debris, molten drip (droplets) d0, d1 and d2 (Tab. 6.1) are classified.

The combination of building material class, smoke and drip performance in combination define the reaction to fire of the building material (Tab. 6.2).

Tab. 6.1: *Subcategories of the effects of the fire according to EN 13501-1*

Smoke production		**Flaming droplets / particles**	
s1	None / hardly any smoke production	d0	No flaming droplets
s2	Limited smoke production	d1	Limited flaming droplets
s3	Unlimited smoke production	d2	High incidence of flaming droplets



a: Progress of the temperature increase on the side not exposed to fire on a 15 mm gypsum board

Fig. 6.1: Nominal temperature time curve for fire testing. Example: Test of a 15 mm gypsum board shell /Knauf Gips KG/

6.1.2 Fire resistance of constructional components and constructions

European standards

The fundamental fire protection classification of building elements is according to fire resistance classes. The tests are in accordance with the European normative complexes EN 1364 – 1366 in the test furnace with furnace temperatures acc. to the standard temperature-time curve in acc. to Fig. 6.1. This fire load density from the time-temperature curve corresponds approximately to a "fully developed fire".

The classification occurs according to the time period for which the constructional component or construction withstands the fire. The classes are comprised of the letters of the respective performance criteria acc. to Tab. 6.4 and the specification for the fire resistance duration in minutes.

Under thermal insulation I, a defined max. temperature rise on the side not exposed to fire on the construction is assumed.

The classification can be undertaken in stages of 15, 20, 30, 45, 60, 90, 120, 180, 240 min.

The test results can be used for a whole range of classification options for the building element.

Example of the constructional component:

Load bearing wall:

Test result:

- Stability (R) - 104 min.
- Room enclosing (E) - 76 min.
- Thermal insulation (I) - 40 min

Possible classification:

- R 90 - Stability
- RE 60 - Stability + room enclosing
- REI 30 - Stability + room enclosing + thermal insulation

Classification of constructional components and building authority requirements

The technical fire resistance quality of a component is not just characterized by the fire resistance but also by the combustibility of the building materials contained in the building element.

From the diversity of combination options, the assignments listed in Tab. 6.5 are interesting. The table shows the European fire resistance classes and their assignment to the European building authority requirements.

The building authority requirements for fire protection are generally enshrined in the national standards and guidelines. Dependent on the level of danger for the building (building height, floor space, kind of usage, number of building units), the corresponding building requirements are demanded of the building.

Tab. 6.2: Fire performance of gypsum boards, Knauf system acc. to EN 13501-1 /Knauf Gips KG/

Knauf products	**Delivery dimensions**		**European term**		**Reaction to fire**
	Thickness mm	**Width mm**	**Type**	**DIN EN**	
Knauf boards					
Fireboard	12.5 15 20 25 30	1250 1250 1250 1250 1250	GM-F	15283-1	A1
Fire-Resistant Board	12.5/15 18	1250 1250	DF (H2) DF	520	A2-s1.d0 (B)
Solid Board	20/25	625	DF (H2)		
Diamant Hard Gypsum Board	12.5/15 18	1250 625	DFH2IR		
Diamant Paneel	20	625	DFH2IR		
Diamant 1 Mann	10/12.5	1000	DFH2IR		
Silentboard	12.5	625	DF		
X-Ray Shielding Board Safeboard	12.5	625	DF		
X-Ray Shielding Board lead sheet	12.5	625	-	14190 Verf.G	
Floorboard (pre-fab floor screed)	12.5	900	DFI	520	
System sheathing unit for hollow floor system Camillo	18	600	DFH2IR		
Solid Board	18	625	A	520	A2-s1,d0 (B)
Wallboard	12.5	1250	A / H2		
Cleaneo Acoustic boards, perforated and slotted	12.5	1188- 1200	-	14190 Verf.A,C,G	A2-s1,d0 (C.4)
Brio unit (pre-fab floor screed)	18/23	600	GF-W1	15283-2	A1
GIFAfloor FHB for hollow floor system	25 28[4)] 32[4)] 38	600 600 600 600	GF-W1DIR1		A2-s1,d0 A1
GIFAfloor LEP for load elevation on hollow floor system	13/18	600	GF-W1DIR1		
Brio 18 WF composite unit (pre-fab floor screed) Brio 23 WF composite unit (pre-fab floor screed) Brio 18 EPS composite unit (pre-fab floor screed)	28 33 38	600 600 600	-	-	E

Tab. 6.3: *Building authority requirements for reaction to fire of building*

Building authority designations	**Additional demands**		**Reaction to fire acc. to EN 13501-1**	
	No smoke	**No flaming droplets / particles**		
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	

Tab. 6.4: *Classification criteria fire resistance to EN 13501-2*

	Abbreviation	**Criterion**
R	(Resistance)	Stability
E	(Étanchéité)	Room-enclosing
I	(Isolation)	Thermal insulation (exposed to fire)
W	(Radiation)	Limitation of the radiation leakage
M	(Mechanical)	Mechanical action on walls (impact stress resistance)

Tab. 6.5: *Fire resistance classes of components acc. to EN 13501-2 and EN 13501-3 and their assignment acc. to German building authority requirements /Knauf Gips KG/*

Requirements of the Building Authority	**Load bearing**		**Non-load bearing interior wall**	**Non-load bearing exterior wall**	**Raised access floor**	**Independent subceiling**
	without	**with**				
	fire barrier					
Fire retardant	R30	REI30	EI30	E30 (i→o) and EI30-ef (i←o)	REI30	EI30 (a↔b)
Highly fire retardant	R60	REI60	EI60	E60 (i→o) and EI60-ef (i←o)		EI60 (a↔b)
Fire resistant	R90	REI90	EI90	E90 (i→o) and EI90-ef (i←o)		EI90 (a↔b)
Fire resistance time 120 min.	R120	REI120	-	-		-
Fire wall	-	REI90-M	EI90-M	-		-

Fig. 6.2: Test of a 7 m high partition to F90 at the MPA Braunschweig /Knauf Gips KG/

6.1.3 Aspects of fire protection of selected drywall constructions

Suspended ceilings

Drywalling ceiling constructions can guarantee fire resistance in two different modes of action.

- If the drywall ceiling construction has a fire resistance class without the influence of the basic ceiling, it is referred to as "sole fire resistance".
- If the fire resistance is only achieved together with the basic ceiling, the classification is then "fire resistance in conjunction with the basic ceiling". The basic ceilings are assigned into different classes depending on the construction on which the common fire resistance effect is also dependent.

Furthermore, the fire protection of ceiling constructions from two directions can be guaranteed alternatively or simultaneously in conjunction with the basic ceiling or when solely effective.

- Fire protection of ceiling cladding or suspended ceilings "in conjunction with the basic ceiling" is always guaranteed for fire exposure from above and from below of the overall ceiling construction in order to simultaneously protect the other side, whereby the fire exposure in an emergency is naturally only from one of both sides. This type of fire exposure can be guaranteed by both suspended ceilings as well as ceiling linings.
- Fire protection "solely from above" means that the fire protection for exposure to fire in the plenum between basic ceiling and suspended ceiling guarantees the protection of the room underneath the suspended ceiling. This type of fire protection is only of relevance with suspended ceilings, as a ceiling lining does not have a cavity between it and the basic ceiling.
- "Solely from below" provides fire protection for fire exposure from underneath the ceiling lining or suspended ceiling, for protection of the ceiling cavity and the basic ceiling. Ceiling linings and suspended ceilings can fulfil these demands.
- Fire protection "solely from above and from below" means that both the first mentioned requirements must be fulfilled simultaneously by a suspended ceiling. Exposure to fire is expected alternatively from the room underneath the suspended ceiling or from the plenum.

Non-load bearing partition

For non-load bearing drywalling partitions, it is necessary to guarantee the relevant fire resistance class exposure of duration to fire of the room-enclosure and compliance of the surface temperature of the wall side not exposed to fire. For fire walls, the resistance to an impact load is additionally tested after exposure to fire.

Installation shaft walls

The fire protection of installation shaft walls with fire exposure from within the shaft to prevent a spread of the fire into the surrounding rooms has to be guaranteed. It is also necessary to guarantee with fire exposure from the room that functional integrity of the installations within the shaft is maintained and to avoid the spread of fire to other storeys by a corresponding design.

Load bearing constructional component

For load bearing constructional components, the level of protection is not just the fire resistance to flanking rooms

as with non-load bearing walls, but rather additional fire protection to ensure the load bearing capacity of the overall construction. Thus the bracing wall constructions, which are to be braced by the cladding, must guarantee that the bracing effect is retained for a sufficiently long time even under fire exposure.
For cladding of load bearing constructional components such as steel columns and beams, the temperature increase must be limited, so that the structural stability of the steel is not affected for a sufficiently long period.

6.2 Sound insulation

Buildings should be planned and implemented so that users of a building are protected from exterior noise or noise from adjacent rooms. For example, according to the "basic requirements for construction works" as set down by the European parliament, "noise perceived by the occupants or people nearby is kept down to a level that will not threaten their health and will allow them to sleep, rest and work in satisfactory conditions".
The corresponding fundamentals for the acoustic planning of the building are explained in the following sections.

6.2.1 The building in the sound field

Comprehensive sound insulation is mainly determined by:

- Sealing off the external noise from the building interior
- Reduction of the sound transmission from one room to the next in the interior of the building
- Prevention of sound transmission with high levels of noise in the building (industrial noise, discotheque) to the exterior
- Provision of the optimum "acoustic environment" (optimum reverberation time) particularly in large rooms (theatres, music halls)

The first three measures stated can be dealt with by building acoustics and the last measures by the room acoustics. The relationships between noise emission, sound insulation measures and characteristics of sound insulation are clear in Fig. 6.3.
Derived from the hearing response and the frequency relevant sensitivity of the human ear, the range in the frequency spectrum between 100 and 3150 Hz has been determined as the primary range to be protected in building acoustics. At these frequencies, the response of the human ear is most sensitive, and the noise volume tends to be loudest. For special applications (e.g. higher share of low-frequency noises with street noise, a wide spectrum of loud frequencies in cinemas), the rating of the sound insulation is supplemented for this range, and generally a frequency spectrum of 50 to 5000 Hz is taken as the basis. In some European states, this extended range is generally set down in building construction as the standard range, at least in the lower frequency range.

6.2.2 Important technical acoustic terms in building acoustics

Sound and sound levels

Sound consists of mechanical vibrations and waves that spread as "airborne noise" (in the air) or as "structure-borne noise" (in solid materials). The structure-borne excitation of floors and steps is referred to as "impact sound" /6.1/. The logarithmic measure for sound intensity is the sound level L, specified in decibels [dB].
Derived from the previously stated designations, the insulation of the sound waves generally between two rooms is referred to as airborne noise or footfall sound insulation.

Weighted apparent sound reduction index R_w

The sound reduction index R in dB identifies the airborne sound insulation of a component. It is determined in accordance with ISO 140-4 by measurement of the sound level difference between a sending-room and a receiving-room in a building acoustics test stand (Fig. 6.4). A correction of this measured value is undertaken considering the parameter variables surface of the wall and absorption surface in the receiving-room.
Taking into account a frequency-dependent correction of the human hearing sensitivity and following a reference

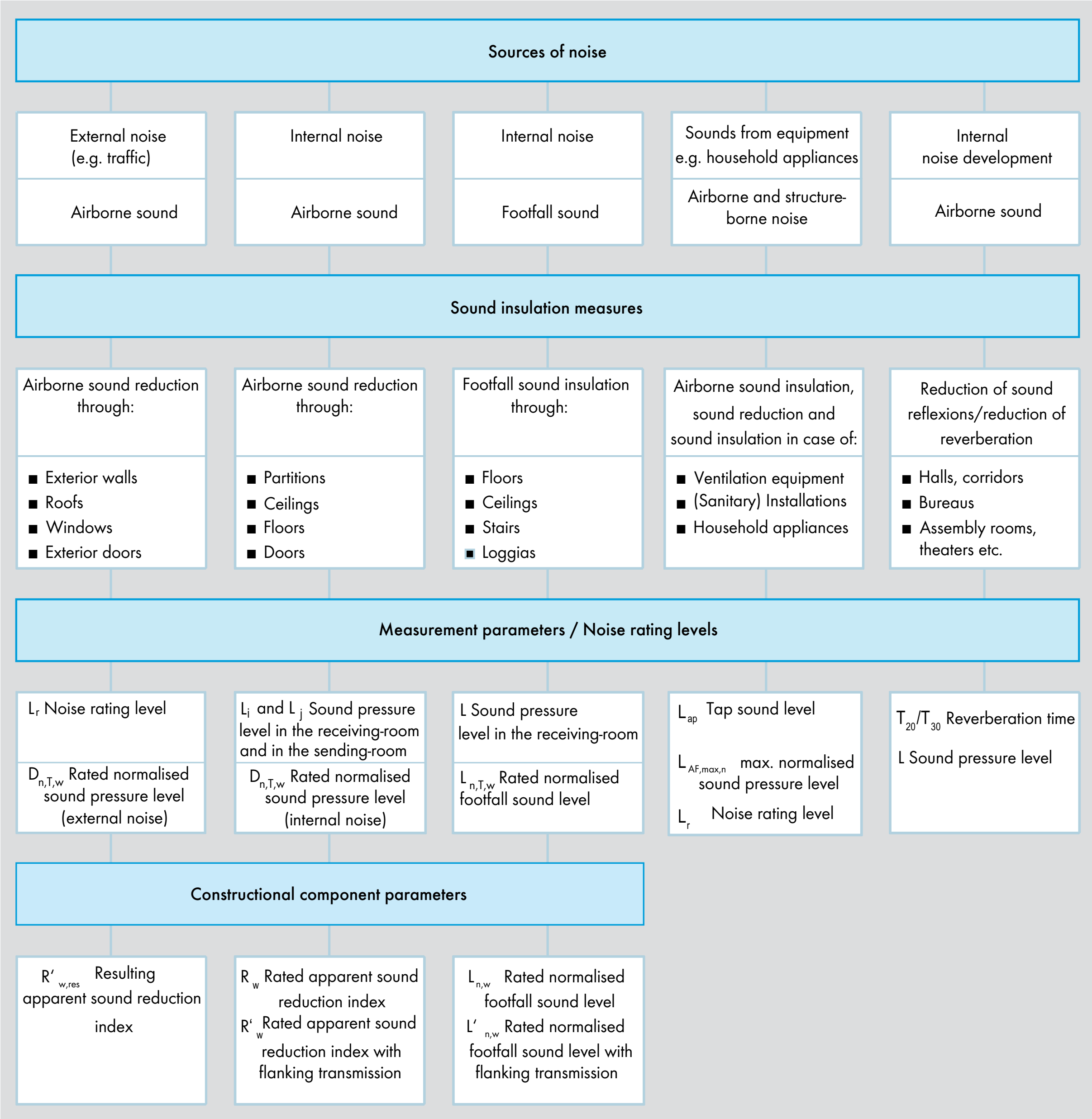


Fig. 6.3: Noise sources, measures, sound insulation characteristics

curve (generally a correction using reference curve A for sound level measurement), the test result is converted to a "weighted apparent sound reduction index R_w".

The weighted apparent sound reduction index R_w describes the airborne sound insulation of a constructional component with sole sound transmission via this component with a single figure value.

In many countries, in accordance with the currently valid standard calculation value $R_{w,R}$ is used. This value is generally arrived at by subtracting a 2 dB margin for walls from the tested value $R_{w,P}$.

Please note:

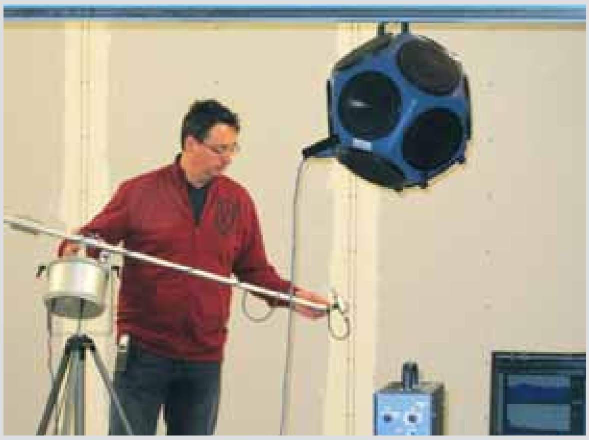
The higher the weighted sound reduction index, the better the airborne sound insulation of the separating components.

Weighted longitudinal sound reduction index $R_{L,w}$ and weighted flanking normalised level difference $D_{n,f,w}$

The insulation of the sound transmission via flanking constructional components is defined in dB by the weighted longitudinal sound reduction index $R_{L,w}$ or the weighted flanking normalized level difference $D_{n,f,w}$.

Both of the terms can be applied according to the standard

a) Test stands



b) Test preparations

c) Test equipment

Fig. 6.4: Building acoustic test stand for measurement of the airborne sound insulation /Knauf Gips KG/

used in the respective country, but consistent use of terms from just one applicable standard is required. Combining terms from different standards is not permissible.

Weighted apparent sound reduction index R'_w and weighted standardized sound level difference $D_{nT,w}$

The weighted sound reduction index R'_w and the weighted standardized sound level difference $D_{nT,w}$ consider that the sound transmission in the building between rooms does not just occur via the separating components but also via the flanking components. Both these parameters are used in dependence on the national guidelines as decisive requirement parameters for airborne sound insulation.

The R'_w value is the sound insulation between two rooms. Here all sound transmission paths, i.e. the direct transmission R_w via separating components as well as the sound transmission $R_{L,w}$ via the flanking paths (generally 4 flanking components) are considered (logarithmic addition).

In most European states, work is on the basis of the European standard instead of the component-related property R'_w of the room-related (reverberation-related) property $D_{nT,w}$ analogue to the specifications of the EN 12354-1 to 3. Using this property in contrast to R'_w, it is assured that the achieved sound protection level for the room concerned considers the acoustic characteristics of the separating components and the flanking components as well as the room geometry (size of the room).

The relationship between the component-related property R'_w and the room-related property $D_{nT,w}$ results from the geometry of the receiving-room acc. to /6.2/:

$R'_w = D_{nT,w} + 10\log(3.1 \cdot S/VE)$ or

for rectangular rooms

$R'_w = D_{nT,w} + 10\log(3.1/l)$

where:

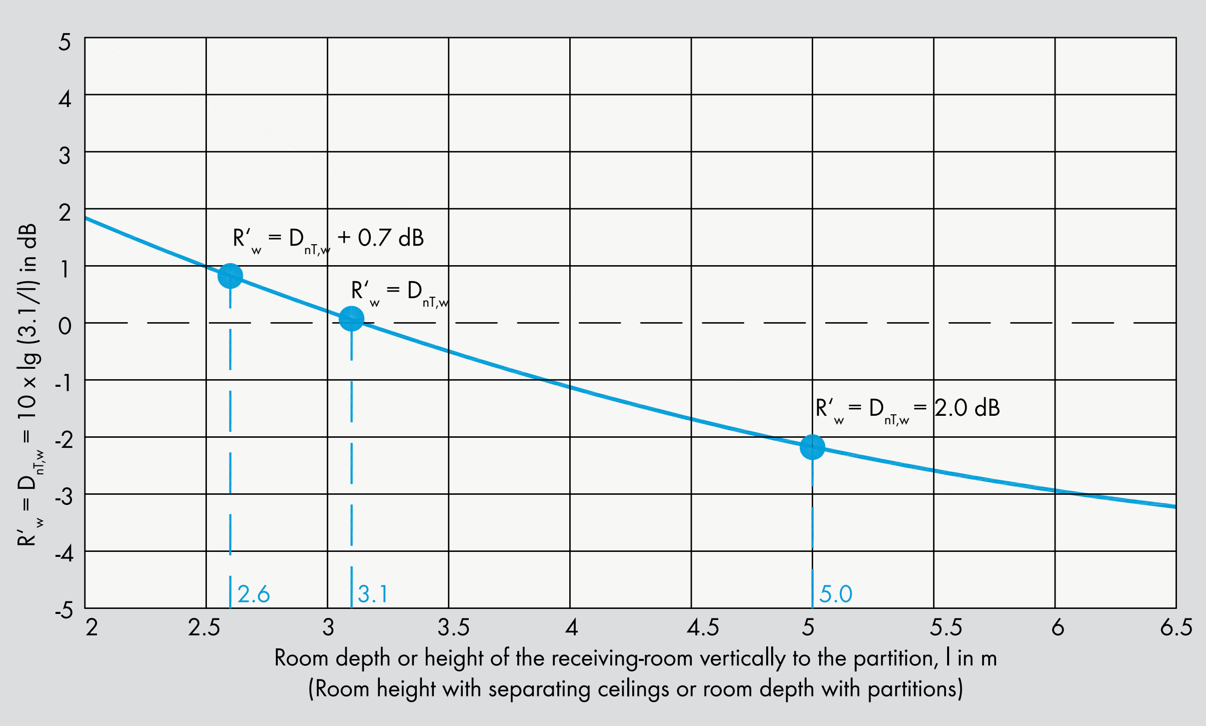


Fig. 6.5: Relationship between constructional component and room-related property of the airborne sound insulation R´ and D_{nT}

- $R´_w$ weighted apparent sound reduction index in dB
- $D_{nT,w}$ weighted standardized sound level difference in dB with a reverberation time of 0.5 s
- l room width in the perpendicular direction to the separating constructional component in m
- S size of the separating surface in m²
- V_E room volume of the receiving-room

Fig. 6.5 illustrates this correlation. In Fig. 6.5, it becomes evident that with a room depth / room height of the receiving room vertically to the separating surface of approx. 3.1 m the R´ and D_{nT} values are identical. If the room depth / room height is greater than 3.1 m, the values of the sound level difference, that is the noise protection, are better than the sound reduction index of the separating component. At smaller room depths / heights on the other hand, the sound insulation reduces at the same sound reduction index of the separating component.

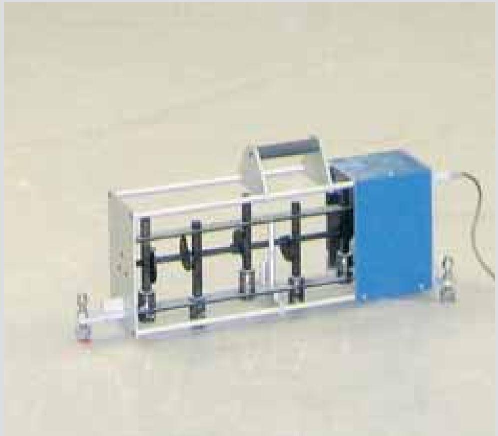
With regard to Fig. 6.5, for example, with the residential rooms with a room depth of 5.0 m the R'_w value of the partition is 2.0 dB less than the $D_{nT,w}$ value. For the floor slabs on the other hand, the R'_w value of the floor slabs with a room height of 2.6 m is 0.7 dB greater than the $D_{nT,w}$ value.

Weighted normalized impact sound level $L'_{n,w}$ and weighted standardized impact sound level $L´_{nT,w}$

The footfall sound pressure level of a component is identified by the weighted normalized impact sound level $L'_{n,w}$ in dB or by the weighted standardized impact sound level $L'_{nT,w}$ in dB.

The weighted normalized impact sound level $L'_{n,w}$ is determined in accordance with ISO 140-7 in a building physics test stand (Fig. 6.6). In contrast to the sound reduction index, the decisive property here is the measured sound level in the receiving-room of the test stand. Measurement occurs across the entire building acoustic range in stages of the third of a bandwidth. The structure-borne excitation of the constructional component occurs with a standardized tapping machine. A correction of the measured values is undertaken taking the sound absorption surface in the receiving-room into consideration with respect to a defined reference surface. Analogue to the airborne sound protection, a correction of the frequency-dependent test values using a reference curve is ultimately performed (generally correction using a reference curve A for sound level measurement),

Test stands for measurement of the impact sound insulation on solid and wood joist ceilings



Impact noise insulation test with a standardized tapping machine

Fig. 6.6: Building acoustic test stand of Knauf Gips KG for measurement of the impact noise insulation using a standardized tapping machine /Knauf Gips KG/

corresponding to the human hearing sensitivity.

The weighted normalized impact sound level $L'_{n,w}$ describes the footfall sound pressure level of a component with normal building flanking paths with a single figure value.

Please note:

The lower the normalized impact sound level, the better the impact sound insulation of the constructional component.

In most European states, work is on the basis of the European standard instead of the component-related property $L'_{n,w}$ of the room-related (reverberation-related) property $L'_{nT,w}$ analogue to the specifications of the EN 12354-1 to 3. With this property in the same manner analogue to the airborne sound insulation in contrast to the $L'_{n,w}$, it must be considered that the achieved impact sound protection level for the room concerned considers the acoustic characteristics of the separating components and the flanking components as well as the room geometry (size of the room).

The relationship between the component-related property $L'_{n,w}$ and the room-related property $L'_{nT,w}$ results from the geometry of the receiving-room acc. to /6.2/:

$L'_{n,w} = L'_{nT,w} + 10 \log (V_E) - 15$

where the following is the case:

- $L'_{n,w}$ weighted normalized impact sound level in the field in dB
- $L'_{nT,w}$ standardized impact sound pressure level in the field in dB with a reference reverberation time of 0.5 s
- V_E room volume of the receiving-room

Fig. 6.7 /6.2/ illustrates this correlation.

From Fig. 6.7, is becomes clearly evident that in the event of a volume of the receiving-room of 32 m^3, $L'_{n,w}$ and $L'_{nT,w}$

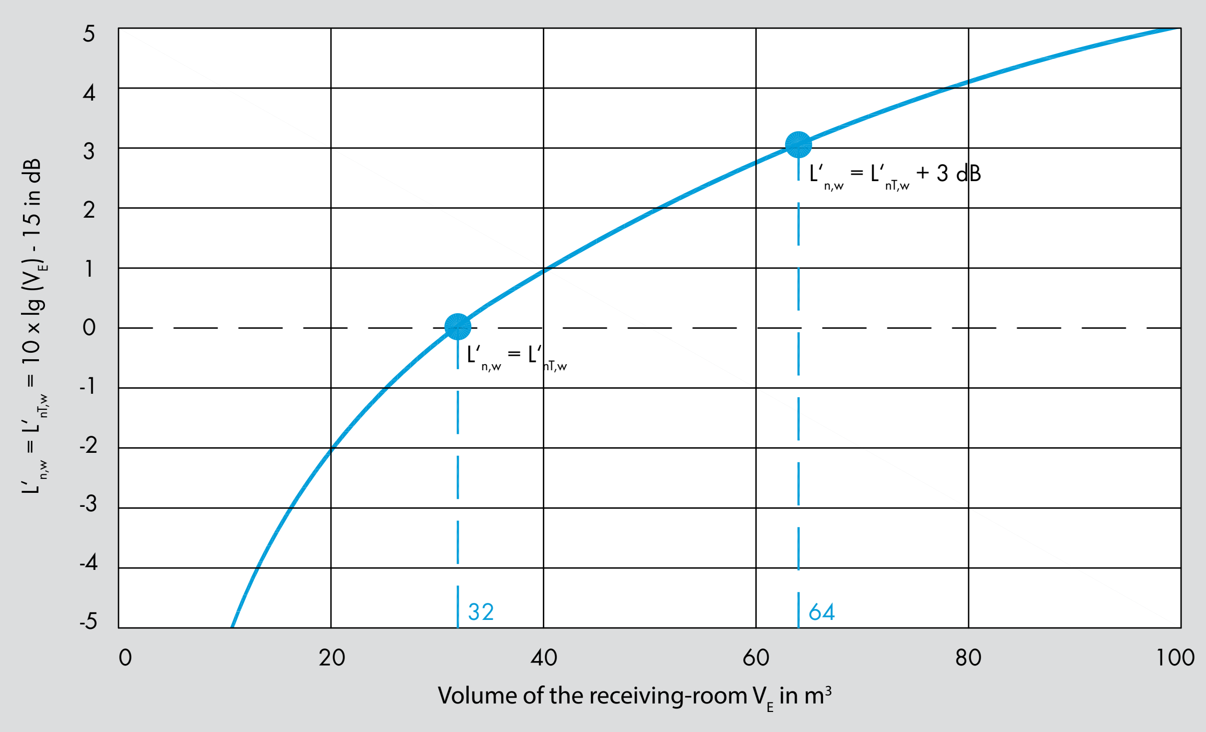


Fig. 6.7: Relationship between constructional component and room-related property of the impact noise insulation $L'_{n,w}$ and $L'_{nT,w}$

are approximately similar. With receiving-rooms becoming larger, the weighted standard footfall sound pressure level $L'_{nT,w}$ is lower than the weighted normalized impact sound pressure level $L'_{n,w}$, i.e. the sound insulation is better than the usual rating expected in Germany according to the weighted normalized impact sound level $L'_{n,w}$. Thus, e.g., the weighted normalized impact sound pressure level L'_{nw} is higher than the standard footfall sound pressure level $L'_{nT,w}$ by about 3 dB with a doubling of the volume of the receiving room to 64 m^3 (view Fig. 6.7).

Equivalent weighted normalized impact sound pressure level $L_{n,w.eq}$, reduction of impact sound improvement index ΔL_w

The equivalent weighted normalized impact sound pressure level $L_{n,w.eq}$ identifies the impact sound insulation performance of solid ceilings without floor slab cover (without floating screed, etc).

The impact noise reduction (impact sound improvement index) ΔL_w is the single value for identification of the improvement in impact sound by a floor slab cover (e.g. floating screed, soft, flexible floor coverings etc.).

The weighted normalized impact sound pressure level $L'_{n,w}$ of the ready-to-use floor slab is in accordance with the following formula:

$L'_{n,w} = L_{n,w.eq} - \Delta L_w$ (dB)

Spectrum adaptation terms

With the spectrum adaptation terms C and C_{tr}, the sound insulation of different constructions in the range 100 – 3150 Hz (if required also with an extended frequency range of 50 to 5000 Hz) paying increased attention to specific noise types (differing noise spectra) is assessed and incorporated for special individual cases in the rating of the sound insulation of the constructional components. The single values for description of the sound insulation quality of components is specified as follows under consideration of the spectrum adaptation terms:

- R_w (C, C_{tr}) in dB
- $L_{n,w}$ (C_i) in dB

In airborne noise, the value C, for example, considers in particular the specific noise spectrum of domestic noise, the value C_{tr}, for example, considers the greater low-frequency share of inner-city street noise. In the impact sound range, the adaptation term C_i in particular, considers low-frequency interference.

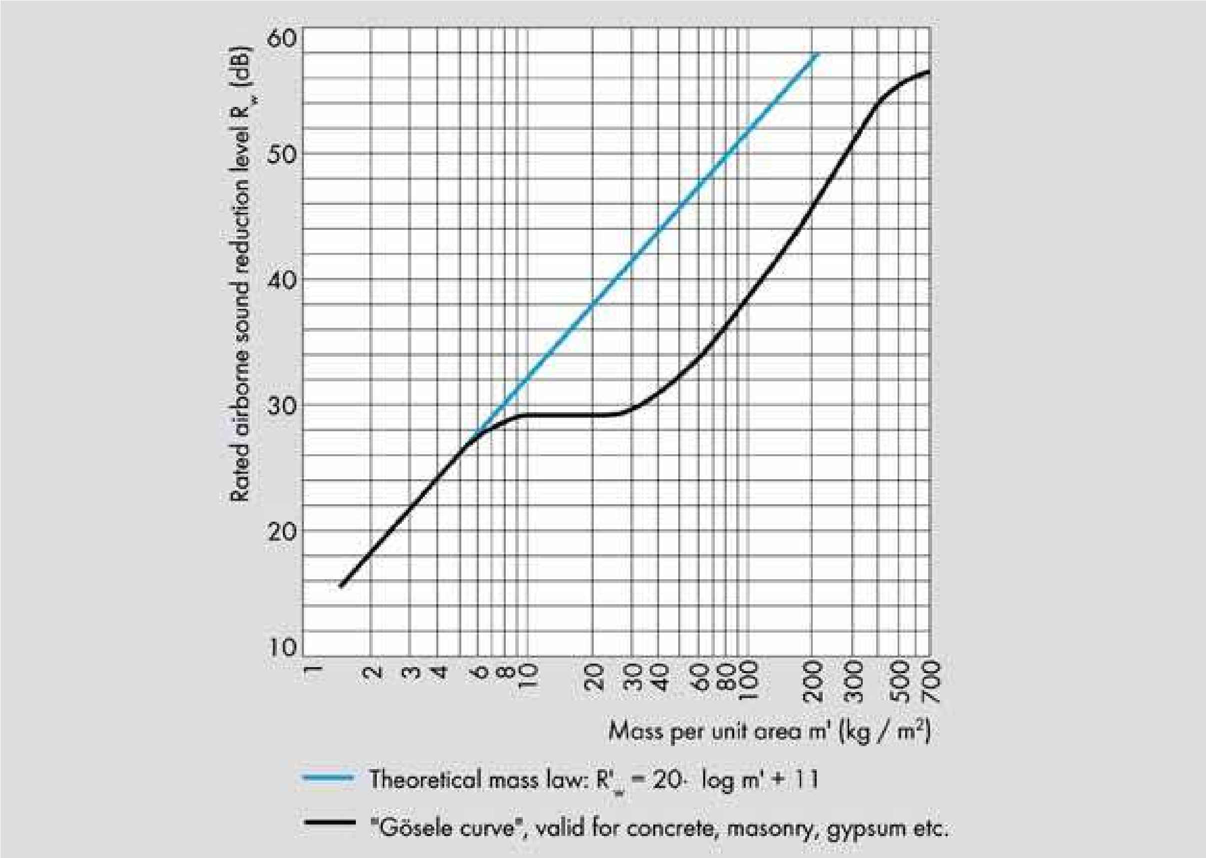


Fig. 6.8: Weighted sound reduction index for single-leaf components in dependence on the mass

6.2.3 Acoustic performance of components – direct sound insulation of separating components

In terms of building acoustics, a differentiation is made between single leaf and multi-leaf (usually two-leaf in practice) constructional components.

Single-leaf components

Single-leaf constructional components are generally constructional components in solid components (e.g. constructional components made of masonry, reinforced steel, solid basic ceilings).

The sound reduction index is dependent on the mass per unit area and the flexural stiffness of the component.

Single-leaf components generally have better airborne sound insulation performance with increasing weight. Usually, the airborne sound insulation performance also increases with the frequency. Only in the critical frequency f_g range of the constructional component (resonance when the wavelengths of the airborne noise and the lengths of the free flexural waves of the constructional component correspond) does the airborne sound insulation degrade. The impairment tendency is visible in Fig. 6.8. This figure indicates using the example of a single-leaf solid construction, the sound protection dip in the mid-mass range compared to the theoretical curve from the mass law of sound insulation. In the lower mass range, and thus accordingly thin, the constructional components are “building acoustically flexurally ductile” and in the upper mass range, and thus accordingly thick, the constructional components are “building acoustically flexurally rigid” and follow the mass law for sound insulation.

Please note:

Good sound insulation with single-leaf constructional components can only be achieved by higher masses per unit area.

Double-leaf components

Construction systems

High area weights to achieve higher levels of sound insulation can be avoided if the constructions are double-leaf. Both leafs are seperated or connected by

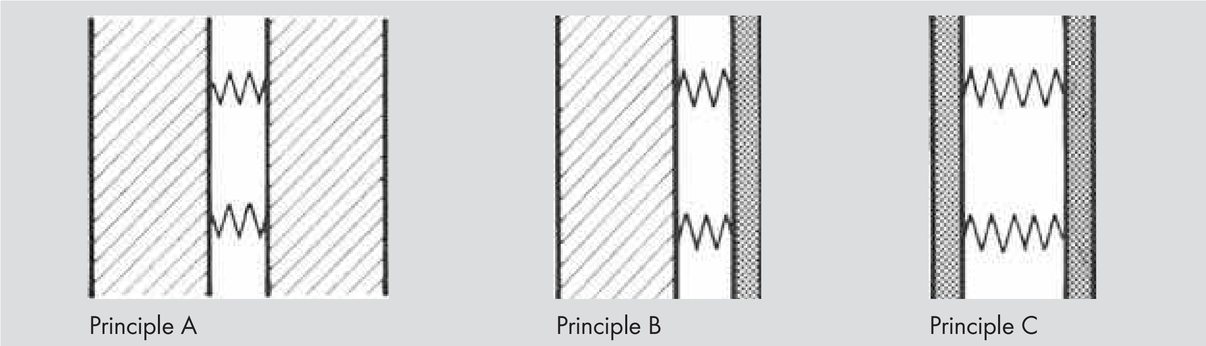


Fig. 6.9: Construction principles of two-leaf constructional component

an attenuating air-layer or a flexible insulation layer. The constructionally necessary connections are flexible in design and should transfer as little sound energy as possible. The construction corresponds to a spring-mass-system principle from a constructional viewpoint.

Here a differentiation is made for the three construction principles according to Fig. 6.10:

- Construction principle A
  Coupling of 2 heavy leafs, generally flexurally stiff shells with an intermediate springy and attenuating layer
  Application:
  Terraced and semi-detached house partitions
- Construction principle B
  Coupling of a heavy leaf, generally flexurally stiff leaf with a light flexurally ductile leaf
  Application:
  Load bearing and non-load bearing partitions with furring particularly in the area of renovation (sound insulation and thermal insulation), solid ceilings with flexurally ductile suspended ceiling / ceiling linings and/or floating screed
- Construction principle C
  Coupling of two flexurally ductile leafs where intermediate components (with good spring properties) must be fitted for construction reasons for stability and connection of the leafs
  Application:
  Drywall partitions

For two-leaf components, the sound insulation of the properties of both individual leafs (= "masses") depends on the connection of both leafs (= "spring") and the insulation material in the cavity. In this case, contrary to

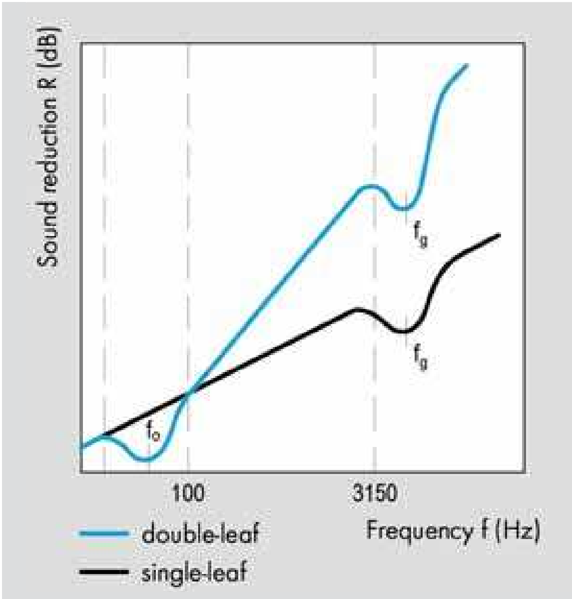


Fig. 6.10: Qualitative progression of sound insulation of one and two-leaf constructions with the same mass per unit area

single-leaf constructional components, there is a diverse range of factors influencing the sound insulation of the constructional components.

A two-leaf construction constitutes a vibration system that itself has a resonance frequency f_0 . If a comparison is drawn between the frequency-dependent progression in sound insulation of two leaf constructional components with single-leaf constructions (Fig. 6.9), it becomes evident that with two-leaf constructions, the increase in the sound insulation with increasing frequency is twice as high than with single-leaf constructions. Degradations occur in the areas of the critical frequency f_g (analogue to single-leaf constructional components) and the resonance frequency f_0 of the two-leaf system.

Until the optimum sound insulation is achieved, two-leaf

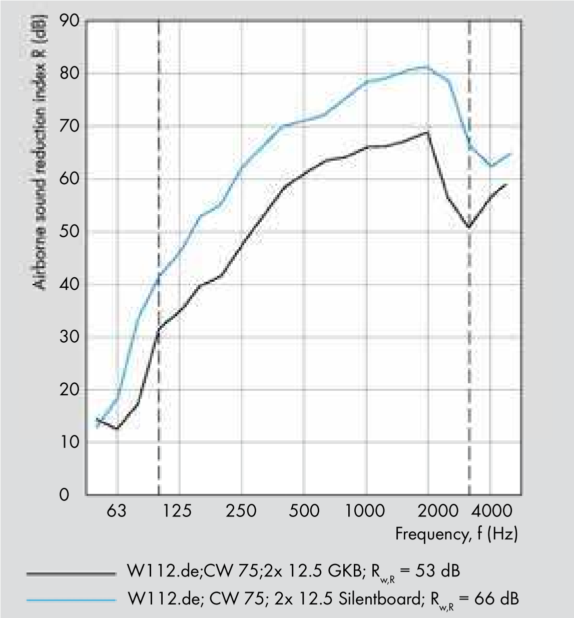


Fig. 6.11: Stud partition in comparison: Cladding made of Silentboard and gypsum board GKB /Knauf Gips KG/

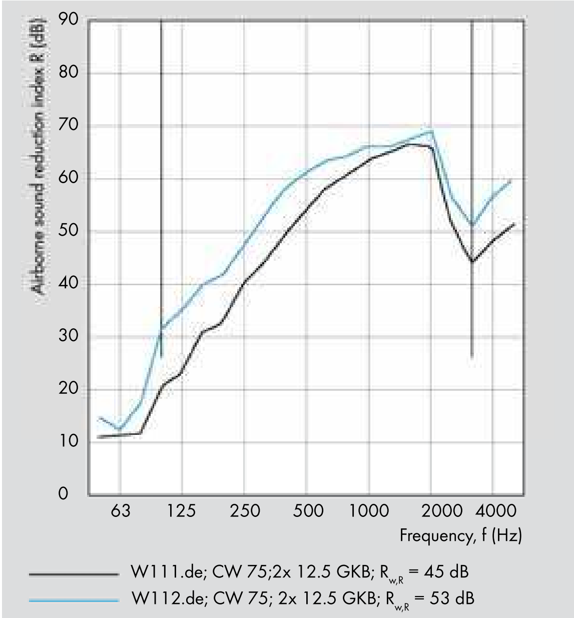


Fig. 6.12: Stud partition in comparison: One and two-leaf claddings /Knauf Gips KG/

constructional components shall be manufactured, so that the resonance frequency, and thus the natural resonance of the system, is below the interesting frequency range of 100 Hz. The resonance frequency becomes lower with an increasing distance between leafs or depending on how much lower the dynamic stiffness of resilient insulation layers is with composite units.

Analogue effects are achieved if the weights per unit area of flexurally ductile leafs are increased. The critical frequency f_g of the flexurally ductile boards should be as high as possible.

In order to prevent so-called "standing waves" in the cavity between the boards (negative resonance effects), attenuation with sound absorbing materials should be implemented.

Please note:

Very high levels of sound insulation can be achieved when employing two-leaf constructional components with significantly lower masses per unit area in comparison to single-leaf solid constructional components.

Typical drywall constructions are implemented according to construction principles B and C.

Metal stud partitions

Metal stud partitions with gypsum boards (construction principle C) can be implemented by an optimum spring-mass-system by the constructional optimization of the metal studs (spring properties) and gypsum boards (flexurally ductile board mass). Very high levels of sound insulation are possible at low construction weights per unit area.

The sound insulation of stud partitions is mainly influenced /6.3/ by:

- Decoupling of the leafs

  Decoupling of the leafs is a decisive property for a well functioning spring-mass-system.

  The lower the acoustic coupling, the higher the sound insulation of the system.

  Dual stud partitions with mutually decoupled (non-connected) stud partitions offer the best and most reliable results in comparison to single stud partitions. Using single stud partitions, to achieve the max. possible sound insulation the stud profiles must be resilient in their design to minimise sound transmission via the studs. In particular with the use of MW profiles, a profile with "spring tabs" (Fig. 6.11) even with single stud partitions achieves a significant reduction in the stiffness in the lateral direction of the cladding and thus achieves a very reliable acoustic decoupling of the leafs.
- Board mass and structure

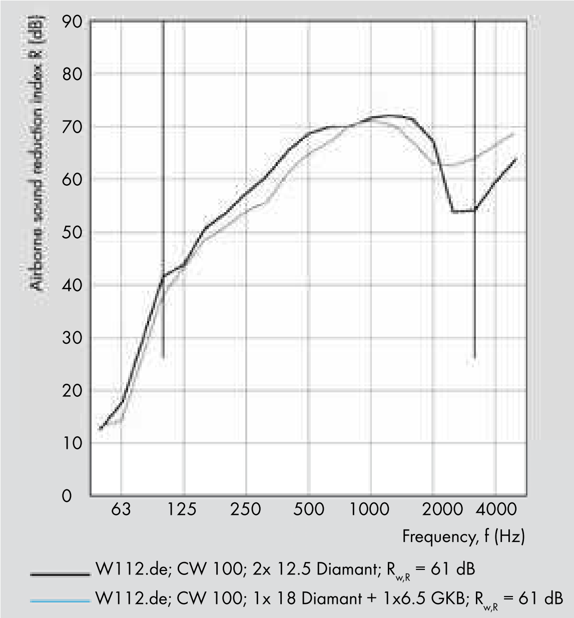


Fig. 6.13: Stud partition in comparison: Mixed board thicknesses per cladding side /Knauf Gips KG/

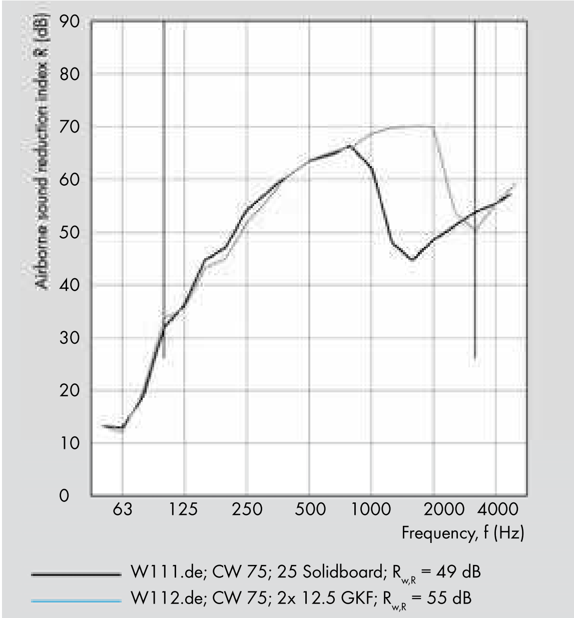


Fig. 6.14: Stud partition in comparison: One and two-leaf claddings with the same cladding thickness /Knauf Gips KG/

Stud partitions achieve optimum sound insulation if they are employed as cladding for building acoustic flexurally ductile boards, to avoid anti-resonance and thus avoid dips in the sound insulation at the interesting frequency range. Gypsum boards at a thickness of ≤ 20 mm fulfil these requirements well. The board weights today with 12.5 mm thick gypsum boards are

- Gypsum Wallboards ca. 8.5 kg/m²
- Gypsum Fire-Resistant Board ca. 10.5 kg/m²
- Diamant Hard Gypsum Board ca. 12.8 kg/m²
- Sound shield Silentboard ca. 17.5 kg/m²

The sound insulating properties of gypsum boards improve with increasing density / board weights. Particularly good results can be achieved in the Knauf system with special sound insulating boards of types "Diamant" and "Silentboard" /6.4/. These board types feature an optimized board core with regard to their sound insulating features, whereby for both the boards mentioned, this property is combined with a high weight per unit area.

With these boards, very good sound insulation values can be achieved even on single stud partitions with normal (relatively stiff) metal stud partitions.

The best possible sound insulation features are achieved by the use of "Silentboard" (Fig. 6.11).

Multi-leaf cladding significantly increases the sound insulation of the stud partitions as opposed to single-leaf cladding (shift of the resonance frequency into the non-critical range Fig. 6.12). Further positive effects on the sound insulation have:

- The combination of different board thicknesses with multi-leaf cladding (reduction of the coincidence dip - Fig. 6.13)
- At the same cladding thickness, select multi-leaf cladding instead of single leaf cladding (2 x 12.5 mm instead of a 25 mm board) - Fig. 6.14 (critical frequency dip shift to a non-critical range)
- Cavity filling

Filling of the wall cavity with open-pore insulation material has a significant influence.

A demand made of attenuating materials is that the length-specific flow resistance is ≥ 5 kPa s/m². This condition is fulfilled by mineral wool with a density ≥ 15 kg/m².

The applicable principle:

The higher the fill level of the cavity, the greater the improvement in the sound insulation of the stud partition as compared to an undampened partition.

In order to fully utilize the sound insulation features

Tab. 6.6: *Weighted improvement of the direct sound insulation via furring in dependence on the resonance frequencies acc. to EN 12354*

Column	**1**	**2**
Row	**Resonance frequency f_0 of the furring in Hz**	**ΔR_w in dB**
1	≤ 80	35 - R_w/2 a) b)
2	100	32 - R_w/2 a) b)
3	125	30 - R_w/2 a) b)
4	160	28 - R_w/2 a) b)
5	200	-1
6	250	-3
7	315	-5
8	400	-7
9	500	-9
10	630 to 1600	-10
11	> 1600	-5

a) *For resonance frequencies under 200 Hz, the minimum value of $\Delta R_w = 0$ dB*

b) *For flexurally ductile furring before solid constructional components with a resonance frequency $30 \leq f_0 \leq 160$ Hz, the improvement in the index ΔR_w can be calculated with the following formula:*

$$\Delta R_w = (74.4 - 20 \lg f_0 - 0.5 R_w) \geq 0$$

R_w: *identifies the weighted sound reduction index of the basic constructional component (wall or ceiling) in dB*

f_0: *resonance frequency*

$$f_0 = 160\sqrt{\frac{0.111}{d}\left(\frac{1}{m'_1} + \frac{1}{m'_2}\right)}$$

where m'_1 *is the mass per unit area of the basic constructional component in kg/m²*

m'_2 *is the mass per unit area of the furring in kg/m².*

d: the cavity depth in m

of the stud partitions, a cavity filling of at least 80 to 100 % should be the objective. When higher density mineral wool with the same degree of filling is used, there is a tendency to expect a higher degree of sound insulation than compared to a "lighter" wool.

- Spacing of the leafs

  The axial spacing of gypsum board leafs, that is the web height of the stud and connection profiles, is not just a structural function feature, it is also a technical sound insulation property. This spacing is responsible for the resonance frequency point that should be below 100 Hz for high-performance stud partitions.

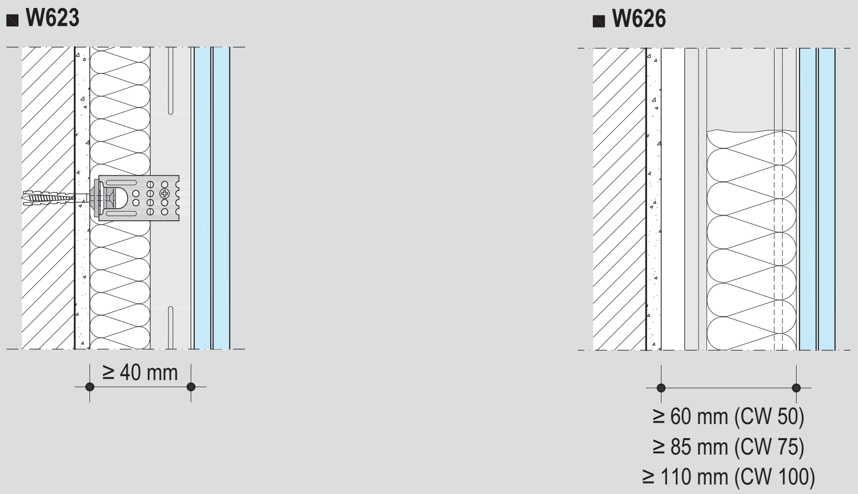
  The applicable principle:

  The greater the spacing between leafs, the lower the resonance frequency and the higher the sound reduction index of the stud partition.

  Deduced from this fact and supported by comprehensive measurements, there is a solution available that even meets the high demands of R'_w of approx. 70 – 75 dB when the corresponding additional site conditions allow it.

The construction palette and the technical and building physical data of the stud partition constructions for interior fittings with the weighted sound reduction index $R_{w,R}$ are summarized in Tab. 3.1.

With façade constructions as metal stud partition constructions in accordance with section. 3.2.7, the same construction principles apply for sound insulation as for stud partitions.



- Detached metal stud framework consisting of CW profiles or metal grid directly connected using damping universal brackets with CD profiles
- Gypsum board cladding
- Cavity depth ≥ 40 mm
- Filling of the cavity (≥ 80 % filling) with open pore insulation material with length-specific flow resistance of r ≥ 5 kPa·s/m² (e.g. common glass wool with approx. 15 kg/m³ density)

Fig. 6.15: Knauf furring, preferred variants

Furring and solid walls

Particularly effective for improving the sound insulation of single-leaf solid walls and similar walls is flexurally ductile furring.

This involves both the level of improvement for direct transmission (ΔR) as well as for flanking transmission (ΔR_L or $\Delta D_{nf,w}$) when used before flanking walls.

The furring together with the solid wall forms a spring-mass-system according to construction principle B. The level of improvement is dependent on the construction design of the furring. Optimum results are achieved when complying to the following fundamentals

- Max. building acoustic decoupling of furring from the solid wall (detached or resilient coupling)
- Cladding with flexurally ductile board
- Tuning of the cavity depth to low resonance frequencies
- Cavity attenuation by open-pore insulation materials

These conditions are ideally implemented using “detached furring” Knauf systems with metal stud framework and “directly attached furring” with metal stud framework with elastic point coupling to the solid wall.

- Sound insulation improvement by detached and directly anchored furring

The improvement index ΔR of furring (even with wooden substructure) can be calculated acc. to EN 12354. In Tab. 6.6, according to EN, new scientific knowledge has been incorporated. For this reason, it is the basis for further versions.

From Tab. 6.6, it is clear that the resonance frequencies in the system “flexurally ductile furring + rigid solid component” are the decisive factors for the expected sound reduction index of the overall construction. The level of the resonance frequency is mainly determined by the distance of the flexurally ductile leaf from the solid constructional component and by the mass of the furring. The mass of the solid constructional component has a minor influence on the resonance frequency.

The following applies:

The greater the clearance (attenuated air gap) and / or the mass of the furring, the lower the resonance frequency and the higher the sound reduction improvement of the solid constructional component.

For Knauf furrings with the preferred solutions acc. to Fig. 6.10, the resonance frequencies in conjunction with the customary solid building walls are in the range of 20 to 80 Hz dependent on the frame (wall clearance) and the board type, thickness and number of board layers (mass of the leaf), so that the maximum sound reduction index

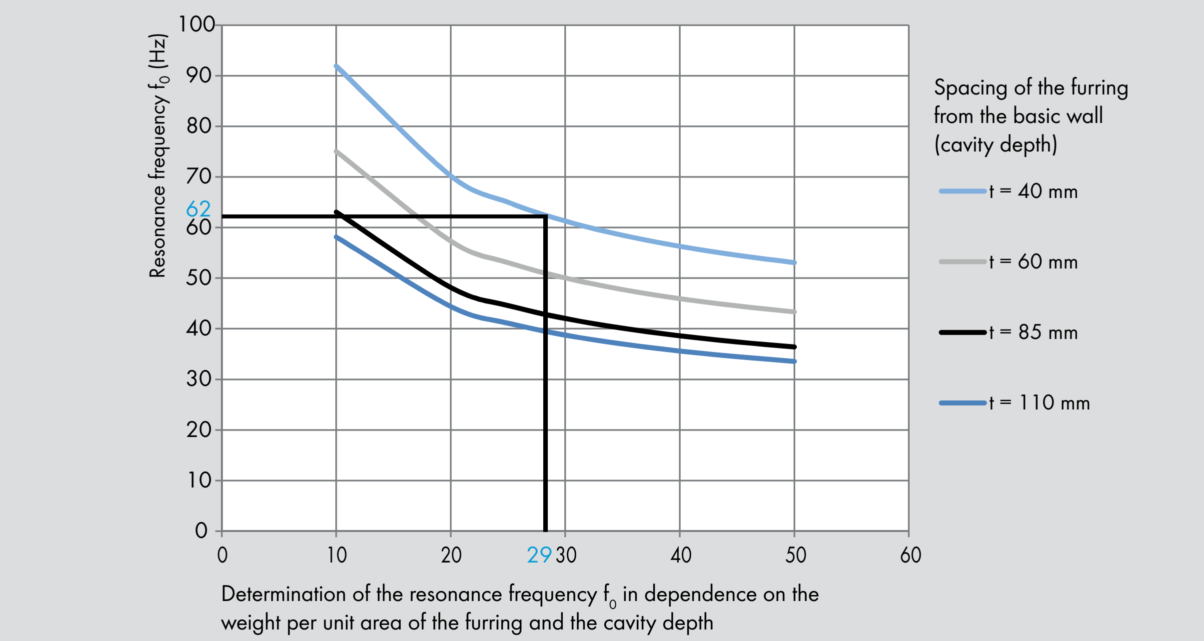


Fig. 6.16: Determination of the resonance frequency

improvement ΔR_w for the solid walls can be achieved.

Simple determination of ΔR_w can be undertaken with sufficient accuracy according to the diagrams of Figs. 6.16 and 6.17.

From Fig. 6.16, the resonance frequencies of typical Knauf furrings with common frames and the resulting distance to the solid construction component of

- 40 mm (CD 60/27 connected at points using universal brackets)
- 60 mm (independent CW 50)
- 85 mm (independent CW 75)
- 110 mm (independent CW 100)

and preferred cladding with mass/m² of 10-50 kg/m², e.g.

- 10 kg/m² (12.5 mm GKF)
- 20 kg/m² (12.5+12.5 mm GKF)
- 24 kg/m² (12.5+12.5 mm Diamant)
- 29 kg/m² (12.5 Silentboard +12.5 mm Diamant)
- 34 kg/m² (12.5+12.5 mm Silentboard)

can be determined with sufficient accuracy for basic variants with a mass ≥ 50 kg/m².

From Fig. 6.17, the determination of the resonance frequency of the improvement index ΔR_w can be read off (linear interpolation between resonance frequency curves permissible with sufficient accuracy), and hence, by addition "R_w basic wall + ΔR_w furrings", the resulting sound insulation $R_{w,ges}$ is determined (see example).

Example for calculation of f_0, ΔR_w and $R_{w,ges}$:

- Basic wall: Mass per unit area m_1 = 100 kg/m²
- Furrings: CD 60/27 substructure, d = 40 mm; 30 mm mineral wool, cladding with 12.5 mm Silentboard + 12.5 mm Diamant as a cover layer, m_2 = 29 kg/m²

Thus, the

- Resonance frequency from Fig. 6.17 for 40 mm furring spacing, mass per unit area of the furring 29 kg/m² :
- Intersection results in resonance frequency $f_0 \approx 62$ Hz
- Improvement ΔR_w from Fig. 6.17 for $f_0 \approx 51$ Hz and mass per unit area of the basic wall of 100 kg/m²: $\Delta R_w \approx 10$ dB
- R_w basic wall for 100 kg/m² from Fig. 6.17: $R_w \approx 38$ dB
- $R_{w,ges} = R_w$ basic wall + ΔR_w furrings
- $R_{w,ges}$ = 38 + 19 = 57 dB (one sided furrings)
- Sound insulation improvement by directly applied furring made of composite board

The improvement index ΔR can also be determined by directly applied furring made of composite board in acc. to Tab. 6.6. In this case, the resonance frequency f_0 should be calculated as follows:

$$f_0 = 160\sqrt{s'\left(\frac{1}{m'_1} + \frac{1}{m'_2}\right)}$$

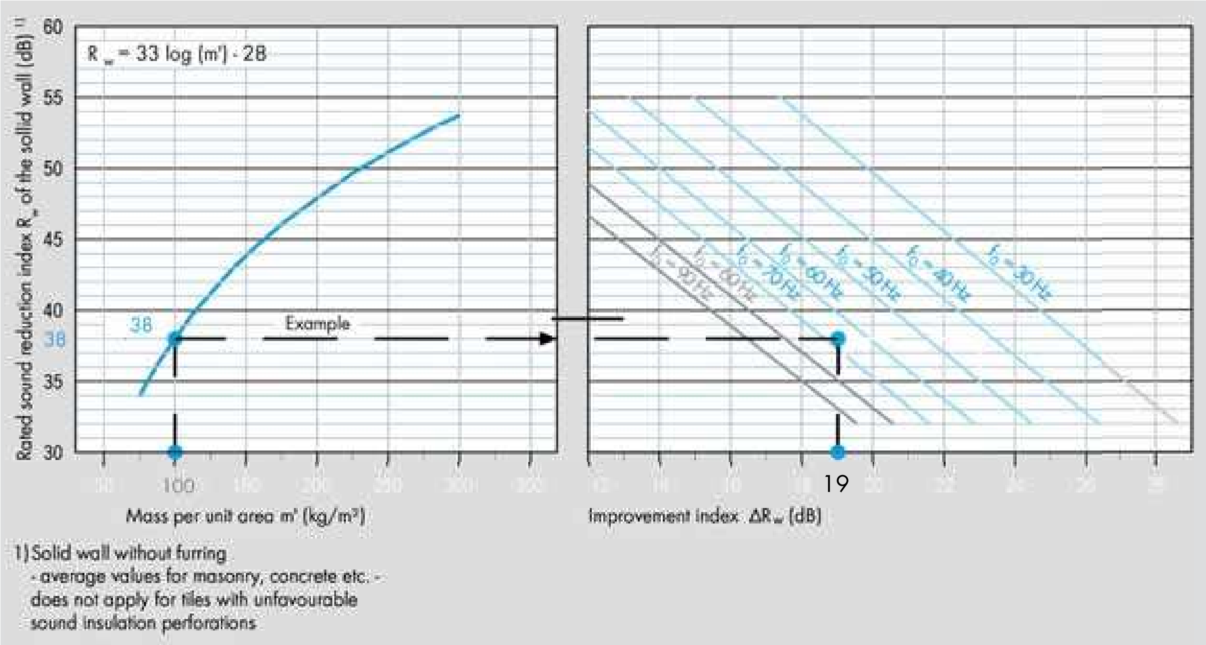


Fig. 6.17: Determination of the improvement index ΔR_w of furrings, Knauf system in dependence on the mass of the basic wall and the resonance frequency

where

s′ is the dynamic stiffness s′ of the insulation layer acc. to EN 29052-1 in MN/m³

m'_1 is the mass per unit area of the basic constructional component in kg/m²

m'_2 is the mass per unit area of the furring in kg/m².

Using empirical data from practical application, the calculated results can only be achieved with a large degree of uncertainty.

Solid floors/ceilings with floating screed / suspended ceiling

For constructional reasons, solid slabs with floating screed, and if applicable, with light flexurally ductile suspended ceilings are the typical two-leaf (or multi-leaf) constructions as highlighted according to construction principle B. As these slabs require, in addition to airborne sound insulation, a sufficient impact noise insulation, the "light leaf" on the top of the slab, the floor, etc. have the task of preventing the direct sound transmission via the supported ceiling construction by decoupling, in as far as possible, the noise input to the top of the slab from the sound emission on the bottom of the slab. This effect is reinforced by the additional suspended ceiling.

The improvement measures for the airborne noise insulation ΔR_w can also be determined acc. to Tab. 6.6.

6.2.4 Acoustic performance of components – longitudinal sound insulation of flanking components

The longitudinal sound transmission is dependent on the construction of the separating constructional component (solid construction, lightweight construction, etc.) and the connection of the separating to the flanking constructional components (contact point attenuation).

According to EN 12354-1, for the combination lightweight component as a separating constructional component and flanking component as well as the combination lightweight component as a separating constructional component and solid constructional component as a flanking constructional component, only the transmission paths Ff (Fig. 6.18) need to be considered.

With solid flanking walls, the longitudinal sound insulation is mainly dependent on the mass per unit area (direct sound insulation index) of these walls. If flanking constructional components are implemented as a lightweight construction, the flanking sound transmission is dependent in particular on the connection design of the separating constructional components to the flanking constructional component.

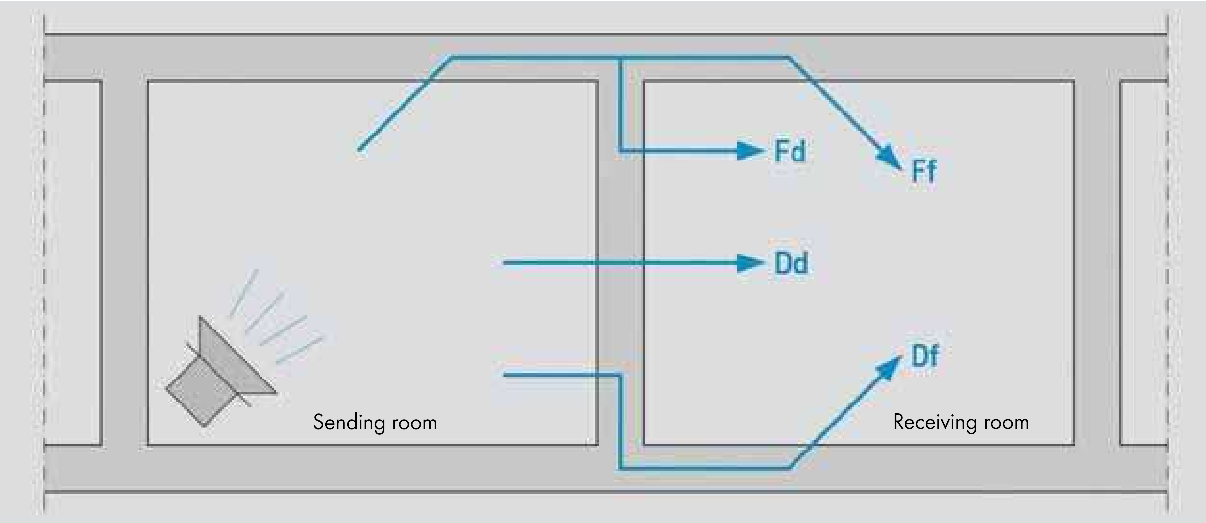


Fig. 6.18: Sound transmission paths between two rooms /Knauf Gips KG/

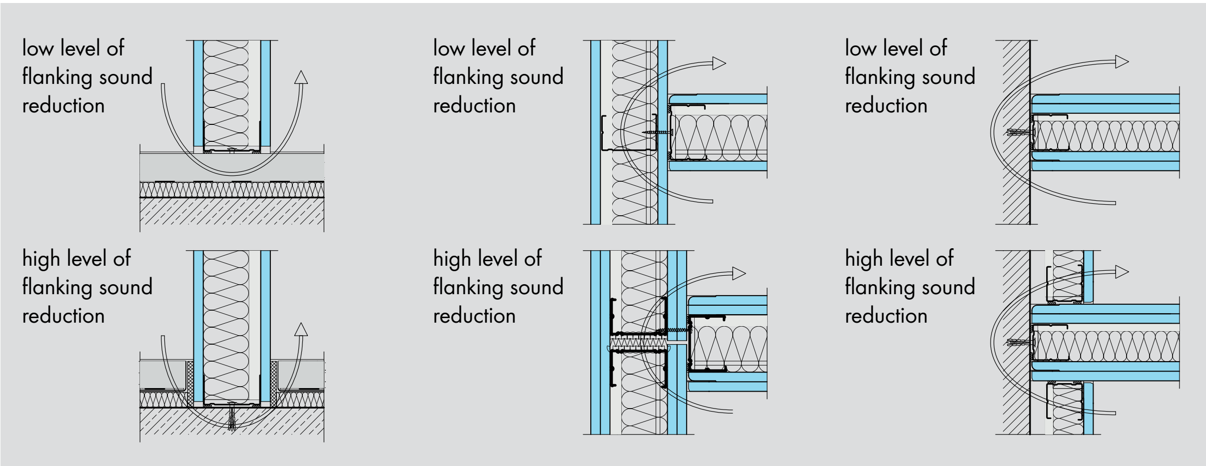


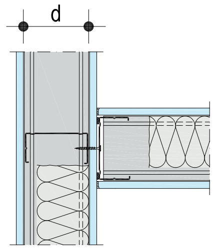
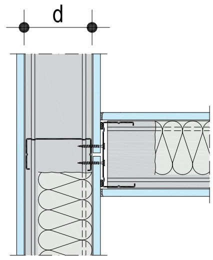
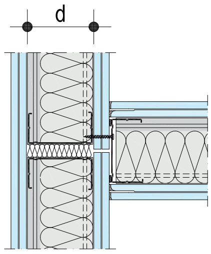
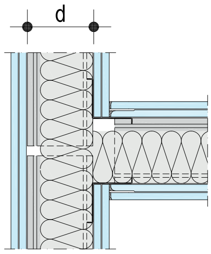
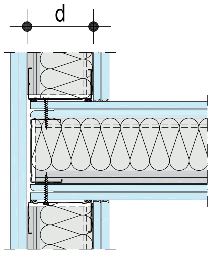
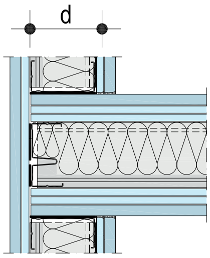
Fig. 6.19: Measures for reduction of the sound longitudinal transmission with flanking constructional components /Knauf Gips KG/

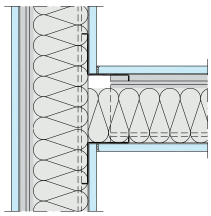
Basically, two paths exist with lightweight flanking components to transmit the sound, irrespective of whether it is with ceilings, floors or walls:

- Transmission via the boards (e.g. lining, cladding)
- Transmission via the cavity
- The measures for reducing the sound longitudinal transmission are effective for both these transmission paths.
- In order to minimize the transmission of sound waves in a cavity, it is attenuated with fibre-based insulation materials or at the very least in the connection area of the separating constructional component (absorber bulkhead).
- A higher cladding mass has a positive effect, so that the flanking sound transmission via double cladding is less than with a single cladding layer.
- Most effective is the separation of the flanking leaf in the connection area to the separating component, i.e. no continuous cladding exists between the two adjacent rooms. Ideally the separating component "slides into" the flanking component and completely separates it. With these types of constructions, the longitudinal sound reduction indexes are so high that a sound longitudinal transmission via the flanking component practically no longer occurs (Fig. 6.19).

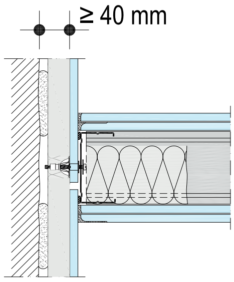
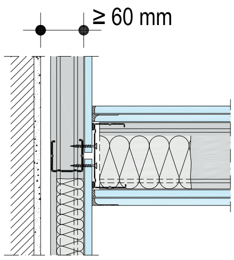
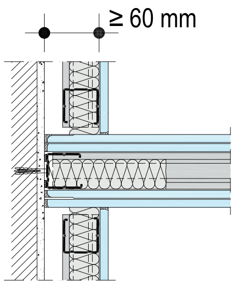
In lightweight construction, the weighted flanking level difference $D_{n,f,w}$ is applied for calculation purposes. Longitudinal sound reduction indeces for lightweight constructions are summarized, for instance, in Tab. 6.7 and 6.9 acc. to E DIN 4109-33.

Tab. 6.7: *Weighted flanking normalized level difference $D_{n,f,w}$ of metal stud partitions acc. to E DIN 4109-33*

Construction example		**Cladding of the inner side of flanking wall**	**Flanking normalized level difference $D_{n,f,w}$**	
		min. thickness mm	**d = 50 mm dB**	**d = 100 mm dB**
Continuous Continuous cladding of the flanking wall without joint		Single-layer ≥ 12.5 board GKB	53	55
		Double-layer ≥ 2x12.5 board GKB	56	59
With joint Room-side cladding of the flanking wall with joint (≥ 3 mm)		Single-layer ≥ 12.5 board GKB	57	59
		Double-layer ≥ 2x12.5 board GKB	60	61
With joint Room-side cladding of the flanking wall with joint (≥ 3 mm)		Double-layer ≥ 2x12.5 Diamant	-	73
With joint Room-side cladding of the flanking wall with joint (≥ 5 mm)		Double-layer ≥ 2x12.5 Silentboard	-	74
Interrupted Room-side cladding interrupted, continuous outer cladding		Double-layer ≥ 2x12.5 Diamant	-	75
Integrated Room-side cladding interrupted, continuous outer cladding		Double-layer ≥ 2x12.5 Diamant	-	75
		Double-layer ≥ 2x12.5 Silentboard	-	76
Integrated Room-side cladding interrupted, continuous outer cladding		Double-layer 1x12.5 Silentboard + 1x18 Diamant	-	80

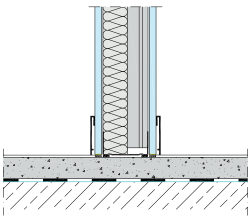
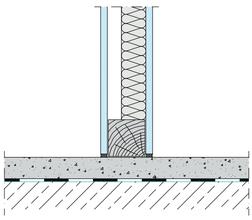
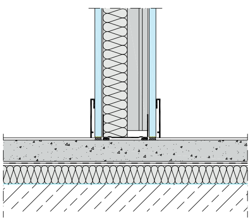
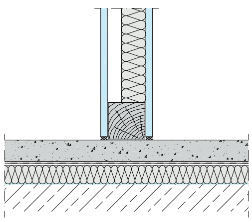
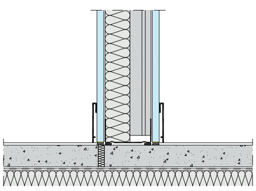
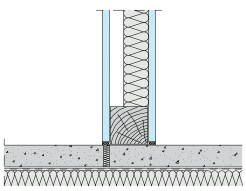
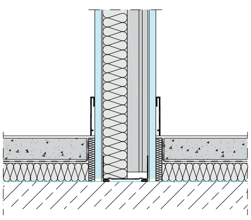
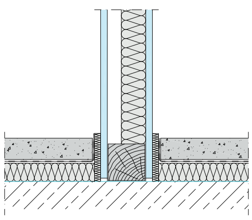
Construction example		Cladding of the inner side of flanking wall	Flanking normalized level difference $D_{n,f,w}$	
		min. thickness mm	d = 50 mm dB	d = 100 mm dB
		100	1	65 (-2;-7)

Tab. 6.8: *Weighted flanking normalized level difference $D_{n,f,,w}$ of rigid walls with furring for horizontal sound transmission acc. to E DIN 4109-33*

Construction example		Mass per unit area of the solid wall	Flanking normalized level difference $D_{n,f,w}$
		kg/m²	dB with 12.5 Knauf wallboard GKB
Continuous furring with composite boards MW		100	55
		200	59
		250	59
		300	60
		400	60
Independent, continuous furring with linear joint		≥ 100	63
Independent, interrupted furring		≥ 100	≥ 70

Tab. 6.9: Longitudinal sound reduction index for flanking solid ceilings with and without screed /Knauf Gips KG/

Solid ceilings without screed or with bonded screed	
Mass per unit area including bonded screed kg/m²	**Rated longitudinal sound reduction index $R_{L,w,R}$ dB**
100	41
200	51
300	56
350	58
400	60
500	63

Solid ceilings with screed on a separating layer / floating screed					
Construction examples Mass per unit area of the solid ceiling ≥ 300 kg/m²			**Rated longitudinal sound reduction index $R_{L,w,R}$ in dB**		
			Gypsum, cement, anhydrite or magnesite screed	**Mastic asphalt screed**	**Pre-fab floor screed**
Continuous screed on separating layer			44 to 48	50 to 52	-
Continuous screed on mineral wool / fibre insulation layer			40	46	-
Continuous screed with separation joint on mineral wool / fibre insulation layer			57	-	-
Screed constructionally separated by partition connection			72	-	72

Tab. 6.10: Longitudinal sound reduction index for flanking solid walls /Knauf Gips KG/

Flanking solid walls	
Mass per unit area kg/m²	**Rated longitudinal sound reduction index $R_{L,w,R}$ in dB**
100	43
200	53
300	58
350	60
400	62
500	65

6.2.5 Calculation of resulting airborne sound insulation from separating components and flanking

The resulting airborne sound insulation is calculated acc. to European Standard EN 12354-1.

In accordance with the procedure as set out in EN 12354-1, for the resulting airborne noise transmission between two rooms the direct sound transmission via the separating constructional components and the sound transmission via the flanking paths are considered. Their individual quantities are added to get the resulting sound transmission energy value.

Further flanking paths, such as the transmission via ducts, ventilation systems, leaks caused by component installation are not considered in the calculation process.

For lightweight, flexurally ductile two-leaf partitions, e.g. partition walls with metal or wood partition walls, an approximation proof for frame designed buildings is used in practice in Germany on the basis of the DIN 4109, amendment 1: 1989 as well as the E DIN 4109-02:2013-11 and E DIN 4109-33:2013-12. With this procedure, the expected resulting building sound reduction index has to be calculated with the following formula by logarithmic addition of the sound insulation value (direct pass) of the separating component (partition) and the longitudinal sound reduction values of the flanking components (generally for floor, ceilings and two walls).

$$R'_{w,R} = -10\log\left(10^{\frac{-R'_{w,R}}{10}} + \sum_{i=1}^{n} 10^{\frac{-R'_{L,w,R,i}}{10}}\right) \text{ dB}$$

A simplified calculation for common room sizes that delivers a sufficiently accurate and safe result is easily possible using a nomographic procedure (Fig. 6.20) by applying the magnitude weighted standardized sound level difference $D_{nT,w}$.

The planning task simply consists of selecting the level of sound insulation of the individual constructional components, so that a corresponding result is achieved that at least corresponds to the required level of sound insulation. Sound insulation overdimensioning of the individual constructional components in a chain of sound transmitting constructional components does not make any sense and only makes the building more expensive.

Please note:

The sound insulation between rooms is only as good as its "weakest airborne noise".

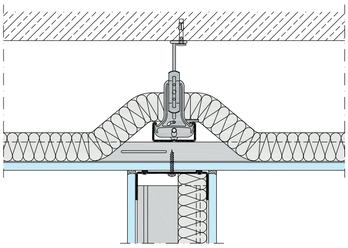
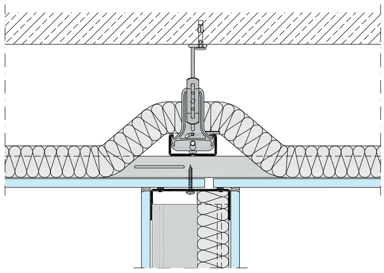
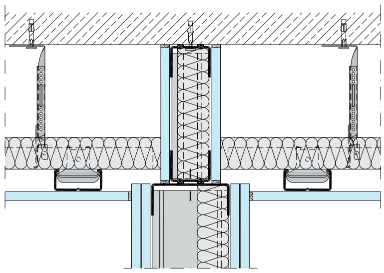
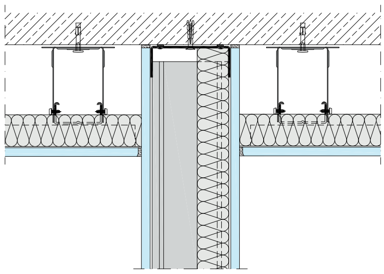
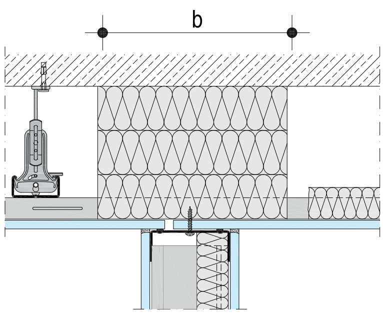
6.2.6 Acoustic performance of components – footfall sound insulation

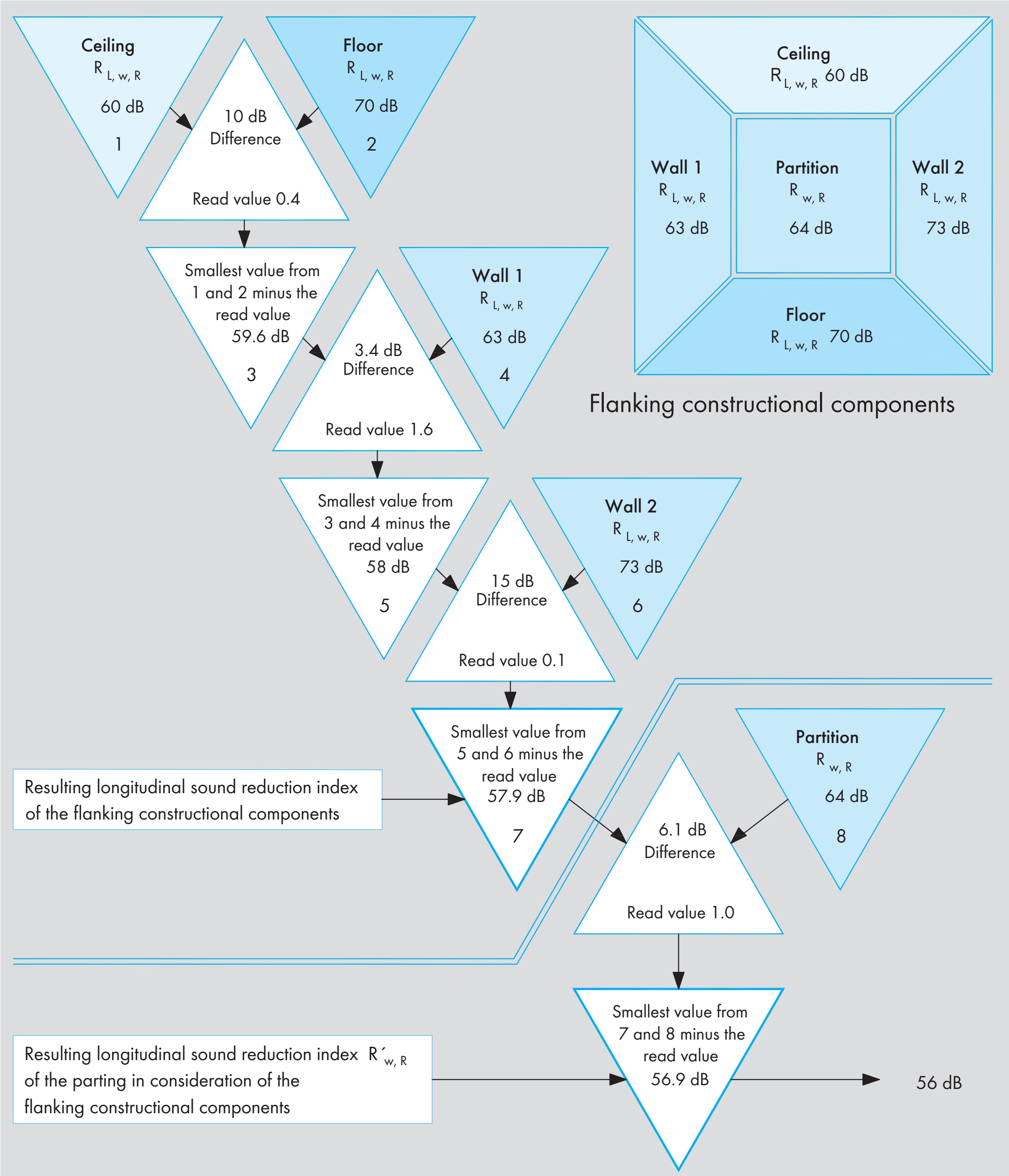
The impact sound insulation quality of a ceiling is determined by the basic ceiling (mass of the basic ceiling and possible combination with a flexurally ductile suspended ceiling) using the value $L_{n,w,eq}$ (equivalent weighted normalized impact sound level), the floor slab cover, e.g. floating mineral-based screeds / floating pre-fab floor screed by the value ΔL_w (impact noise reduction) as well as correction factor K due to flanking transmission.

Impact sound reduction

The impact sound reduction ΔL_w of pre-fab screed constructions, e.g., by Knauf on solid slabs is shown in Tab. 6.12 /6.5/.

Tab. 6.11: Longitudinal sound reduction index for flanking solid ceilings with suspended ceilings /Knauf Gips KG/

Construction examples Suspended ceilings with unperforated surface Suspension height 400 mm		**Cladding with Knauf board GKB**	**Rated longitudinal sound reduction index $R_{L,w,R}$ in dB**		
		min. thickness in mm	**Without mineral wool layer**	**With full mineral wool layer cover**	
				≥ 40 mm	**≥ 80 mm**
Partition connection to suspended ceiling, continuous cover layer		Single layer ≥ 12.5 mm	48	49	50
		Double layer ≥ 2 x 12.5 mm	55	56	56
Partition connection to suspended ceiling, separate cover layer		Single layer ≥ 12.5 mm	50	54	56
		Double layer ≥ 2 x 12.5 mm	57	59	59
Sealing off of the plenum by a board seal		Single layer ≥ 12.5 mm		67	
Partition connection to solid ceiling (The cladding drawn up to the solid ceiling works as sealing off of the plenum)		Single layer ≥ 12.5 mm		67	
Partition connection to suspended ceiling, separate cover layer with absorber bulkhead *) ≥ 400 mm		Single layer ≥ 12.5 mm		62	
*) *Absorber bulkhead from mineral wool acc. to DIN EN 13162, longitudinal flow resistance: r ≥ 8 kPa·s/m²*		***At a suspension height of more than 400 mm, the values have to be reduced by 1 dB.***			



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
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

Read values

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
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

Read values

Fig. 6.20: Approximation calculation of the weighted sound reduction index $R'_{w,R}$ of multi-leaf light partitions using the nomograph process /Knauf Gips KG/

Note:
If the separating surface is 10 m², the room height 2.8 m and the edge lengths 4.5 m, $D_{n,f,w}$ can be set = $R_{Ff,w}$. Should the dimensions of the edge lengths and room heights be significantly less than these values, the flanking sound reduction index from the weighted standardized sound level difference must be calculated using the following equation. Should the dimensions be exceeded significantly, the resulting sound reduction index is underestimated and is thus on the safe side.

Calculation formula:

$$R'_w = -10\log\left[10^{\left(\frac{-R_{Dd,w}}{10}\right)} + \sum_{F=f=1}^{n} 10^{\left(\frac{R_{Ff,w}}{10}\right)}\right]$$

$D_{n,f,w}$: Weighted normalized level difference
l_{lab} : Reference edge length

- For façades and interior walls with horizontal transmission it is 2.8 m
- For ceilings, subceilings and flooring structures with horizontal transmission as well as façades and interior walls with vertical transmission it is 4.5 m

l_f : Common coupling length of the separating and flanking constructional component for the respective building situation in m
S_S : Surface of the separating constructional component in m²
A_0 : Reference absorption surface 10 m²

$$R_{Ff,w} = D_{n,f,w} + 10\log\left(\frac{l_{lab}}{l_f}\right) + 10\log\left(\frac{S_S}{A_0}\right)$$

$R_{Dd,w}$: Direct sound reduction index
$R_{Ff,w}$: Flanking sound reduction index

Equivalent weighted normalized impact sound pressure level $L_{n,w,eq}$

With homogeneous ceiling constructions according to EN 12354-1, the equivalent weighted normalized impact sound pressure level for mass per unit area m` in the range between 100 kg/m² and 720 kg/m² is determined according to the following relationship:

$L_{n,w,eq} = 164 - 35 \lg [m / (1 \text{ kg/m}^2)]$

The determined values for $L_{n,w,eq}$ apply for the direct impact sound transmission in a room directly below.

Correction factor K of solid ceilings

The correction values K with solid ceilings without a suspended ceiling are arithmetically effective if the average resulting area weight of flanking solid walls is equal to or less than the area weight of the ceiling (Tab. 6.13). In addition to the sound insulation effective suspended ceiling under the solid ceiling with a screed cover, they are considered according to E DIN 4109 compliant to Tab. 6.14.

Suspended ceilings are only sound protection rated suspended ceilings that feature improvement potential of airborne noise insulation $\Delta R_w \geq 10$ dB.

If the flanking constructional components are provided with furrings, the deductions due to a reduced flanking transmission path are not necessary.

6.2.7 Calculation of the resulting footfall sound insulation

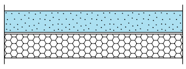
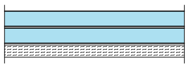
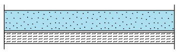
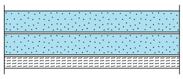
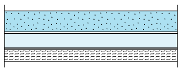
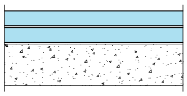
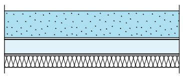
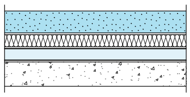
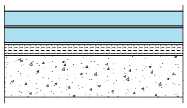
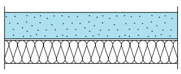
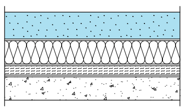
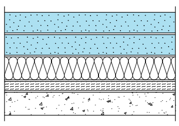
The impact sound insulation (weighted normalized impact sound level $L`_{n,w}$) is calculated acc. to EN 12354-2 using the formula

$L`_{n,w} = L_{n,w,eq} - \Delta L_W - K$ (dB).

6.2.8 Sound insulation regulations and requirements

The building authority requirements for sound insulation are generally enshrined in the national standards and guidelines. Dependent on the usage of the building (residential buildings, educational institutes, hospitals, hotels, etc), generally minimum sound insulation requirements between adjacent apartments and working rooms as well as external buildings are set. Furthermore, the regulations assign limits for noise from domestic systems (e.g. sanitary fittings).

Tab. 6.12: Impact sound reduction of pre-fab screed constructions, Knauf system on solid slabs /Knauf Gips KG/

Floor construction	**Base layer + Construction below the base layer**	**Total thickness**	**Footfall sound reduction Solid ceiling (footfall sound improvement)**	
			Calculation value	**Test value**
	● Brio 18 / Brio 23 ● 20 mm EPS	38/43	$\Delta L_{w,R}$ (dB) 16	$\Delta L_{w,P}$ (dB) 18
	● TUB 2x 12.5 ● 10 mm wood fibre or 10 mm mineral wool or 20 mm EPS or 7 mm PE foil Ethafoam	35 35 45 32	16	18
	● Brio 18 / Brio 23 ● 10 mm wood fibre or 10 mm mineral wool	28/33	17	19
	● Brio 18 + Brio 18 [2)] ● 10 mm wood fibre or 10 mm mineral wool	46	18	20
	● Brio 18 + TUB 12.5 [2)] ● 10 mm wood fibre or 10 mm mineral wool	40.5	19	21
	● TUB 2x 12.5 ● 35 mm Knauf Dry Bulk Leveller PA	60	20	22
	● Brio 23 + TUB 12.5 [2)] ● 10 mm wood fibre or 10 mm mineral wool	45.5	21	23
	● Brio 18 / Brio 23 ● 10 mm mineral wool [3)] or 10 mm wood fibre ● 20 mm Knauf Dry Bulk Leveller PA	48/53 (without cover plate)	22	24
	● TUB 2x 12.5 ● 8 mm wood fibre ● 35 mm Knauf Dry Bulk Leveller PA	68	22	24
	● Brio 23 ● 20 mm mineral wool, $s \leq 40$ MN/m³ [1)]	43	25	27
	● Brio 23 ● 20 mm mineral wool, $s \leq 40$ MN/m³ [1)] ● 8 mm wood fibre ● 20 mm Knauf Dry Bulk Leveller PA	71	28	30
	● Brio 18 + Brio 18 ● 20 mm mineral wool, $s \leq 40$ MN/m³ [1)] ● 8 mm wood fibre ● 20 mm Knauf Dry Bulk Leveller PA	84	31	33
Used for measurements were:	● Wood fibre WF: Density 240 kg/m³; dynamic stiffness 40 MN/m³ ● EPS: EPS DEO acc. to DIN 4108-10 (corresponds to the former PS 20) ● Knauf Dry Bulk Leveller PA: Density approx. 500 kg/m³ ● Mineral wool MW: Density 180 kg/m³, for residential buildings etc. area load 2 kN/m², single load 1 kN. Only use boards, which have been ● declared as suited for gypsum based pre-fab floor screed by the mineral wool manufacturer. General max. compressibility: 1 mm 1) Divergent from above max. compressibility: 2 mm			

Floor construction	Base layer + Construction below the base layer	Total thickness	Footfall sound reduction Solid ceiling (footfall sound improvement)	
			Calculation value	Test value
Remarks for the table:	2) Tested in unbonded state 3) A board covering (≥ 9.5 mm Knauf board) required ● The values apply for composite units and for building site combinations ● For the flooring in italics, Δ L, has been determined. The values of the supplementary flooring are based on experience (equivalent mineral wool / wood fibre; measured value for Brio 18 - equated to Brio 23).			

Tab. 6.13: Correction factor K for flanking transmission with solid ceilings without suspended ceilings according to EN 12354-2 and E DIN 4109-2

Mass m'_s per unit area of the separating slab kg/m²	Average mass per unit area $m'_{f,m}$ of the homogeneous, solid flanking constructional components, which do not have furring constructions kg/m²							
	100	150	200	250	300	350	400	<450
100	1	0	0	0	0	0	0	0
150	1	1	0	0	0	0	0	0
200	2	1	1	0	0	0	0	0
250	2	1	1	1	0	0	0	0
300	3	2	1	1	1	0	0	0
350	3	2	1	1	1	1	0	0
400	4	2	2	1	1	1	1	0
450	4	3	2	2	1	1	1	1
500	4	3	2	2	1	1	1	1
600	5	4	3	2	2	1	1	1
700	5	4	3	3	2	2	1	1
800	6	4	4	3	2	2	2	1
900	6	5	4	3	3	2	2	2

Note: m'_s is the mass per unit area of the separating slab without a screed or suspended ceiling.

Signified:

$R_{w,R}$: calculation value of the weighted apparent sound reduction index of the separating component without longitudinal sound transmission in dB via flanking constructional components

$R'_{L,w,R,i}$ or $D_{n,f,w,i}$: calculation value of the building weighted apparent sound reduction index of the i-th flanking constructional component on the building in dB

n: number of flanking constructional components (generally n = 4)

Tab. 6.14: *Correction factor K_T for flanking transmission with solid ceilings without suspended ceilings according to EN 12354-2 and E DIN 4109-2*

Mass m'_s per unit area of the separating slab kg/m²	**Average mass per unit area $m'_{f,m}$ of the homogeneous, solid flanking constructional components, which do not have furring constructions kg/m²**								
	100	150	200	250	300	350	400	450	500
100	-3	-6	-9	-9	-9	-9	-9	-9	-9
150	-3	-5	-7	-8	-9	-9	-9	-9	-9
200	-2	-4	-6	-7	-8	-8	-9	-9	-9
250	-1	-3	-5	-6	-7	-7	-8	-8	-8
300	0	-2	-4	-5	-6	-7	-7	-8	-8
350	0	-2	-3	-4	-5	-6	-6	-7	-7
400	1	-1	-2	-3	-5	-5	-6	-6	-6
450	1	0	-2	-3	-4	-5	-5	-6	-6
500	2	0	-1	-2	-3	-4	-5	-5	-5
600	3	1	0	-1	-2	-3	-4	-5	-5
700	4	2	1	0	-2	-3	-3	-4	-4
800	5	3	2	0	-1	-2	-2	-3	-3
900	6	4	2	1	0	-1	-2	-3	-3
Note: m'_s is the mass per unit area of the separating slab without a screed or suspended ceiling.									

6.3. Thermal and moisture protection

Thermal protection of buildings includes measures on the envelope of the building and the domestic systems, to on the one hand ensure a cosy interior climate and on the other hand reduce the energy consumption CO_2 and other emissions of pollutants (environmental protection). The climate-related moisture protection, which on the one hand is connected directly with the thermal protection, serves the protection of the building from damage due to the action of moisture. Insufficient thermal insulation leads to the formation of condensation in the construction and on the surfaces of the constructional components. Furthermore, damp building and thermal insulation materials have a reduced insulating effect and impede the energy efficiency of the building.

A differentiation is made between winter thermal protection (heating) or summer thermal protection (air-conditioning, cooling) that can be of varying relevance depending on the geographical location in different countries. For the energy balance, that is the sum of the energy gains and losses for each type of energy flow, there are four types of energy flows considered in accordance with Tab. 6.16 /6.4/.

External building components or separating elements to unheated / non climatized rooms in lightweight construction design can with the corresponding design have a large influence on the transmission heat gains or losses. Fig. 6.21 uses the example of a single-family house in Germany to show the percentage shares of energy requirement, divided according to transmission heat losses through the components on the building envelope

Tab. 6.15: *Energy gains and losses in a building*

High outdoor temperature Building is air conditioned		**Low outdoor temperature Building is heated**	
Effect	**Impact**	**Effect**	**Impact**
Transmission heat gains	negative (room air is heated)	Transmission heat losses	negative (room air is cooled)
Ventilation heat gains	negative (cooled air dissipates to exterior)	Ventilation heat losses	negative (heated room air dissipates to exterior)
Radiated heat gains through windows and opaque constructional components	negative (room air is heated)	Radiated heat gains through windows and opaque, constructional components	positive (room air receives additional heating)
Internal thermal sources (persons, electrical devices, illumination)	negative (room air is heated)	Internal thermal sources (persons, electrical devices, illumination)	positive (room air receives additional heating)

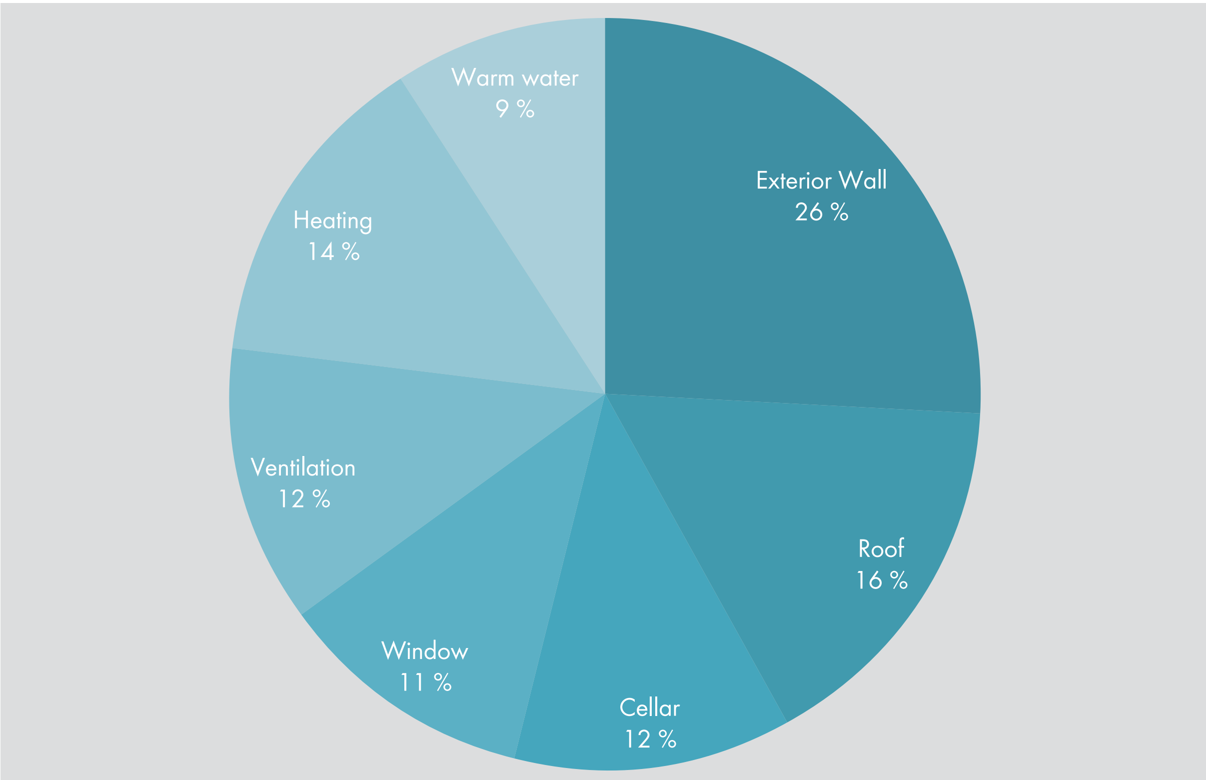


Fig. 6.21: Energy loss of a single-family dwelling /Knauf Gips KG/

(exterior wall, roof, cellar and windows), the ventilation heat losses as well as the energy losses due to heating of the heating medium and heating of the required domestic warm water. From this overview, it becomes clear that a higher level of building thermal protection of the building envelope with the overall view of the residential building in energy terms is the most important energy conservation measure for reducing the heating energy requirement.

6.3.1 Important thermal and moisture protection terms and parameters

Heat insulation

Thermal insulation

The thermal insulation of a building should when possible incorporate a continuous insulation of the building shell from the environment. All heated rooms are enclosed by

Tab. 6.16: Thermal conductivity of lightweight building materials, Knauf system /Knauf Gips KG Iphofen/

Building material	Thermal conductivity in W/(m·K)	Thickness in mm	Thermal resistance in $m^2 \cdot K/W$
Gypsum board	0.25	12.5	0.05
		15	0.06
		20	0.08
		25	0.10
Gypsum fibre board	0.30	12.5	0.04
		15	0.05
Gypsum fibre board High density	0.38	18	0.05
		23	0.06
Cement board Outdoor	0.32	12.5	0.04
Cement board Indoor	0.36	12.5	0.03
Cement board Floor	0.79	22	0.03
Insulation material	0.04	40	1.00
		60	1.50
		80	2.00
	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

the insulated building envelope, and heated rooms are insulated against unheated rooms.

The thermal insulation with solid constructional components is generally applied as interior or exterior insulation.

From a building physics point of view, the arrangement of the insulating layers on the exterior (exterior insulation) is the preferred solution. Exterior insulation, designed as weather protection has good features and is error-tolerant, generally reduces the moisture level in the masonry, minimizes the problem with thermal bridges and facilitates the use of the solid constructional components for heat storage. Internal insulation is complicated and complex from a building physics point of view and planning and application errors can lead to subsequent problems with moisture damage to the building components.

For new buildings, it is usually not a problem to integrate the exterior insulation in the planning phase. This is not the case during renovation of existing buildings, as the application of exterior insulation may not be possible for various reasons, e.g. such as historic protection of the façade. In this case, interior insulation is a real alternative to exterior insulation and is frequently the simpler and faster method and often the only possibility available.

The insulation is generally integrated into the level of the component frame between the interior and exterior cladding with lightweight construction.

Thermal conductivity

The thermal insulation of a building material is

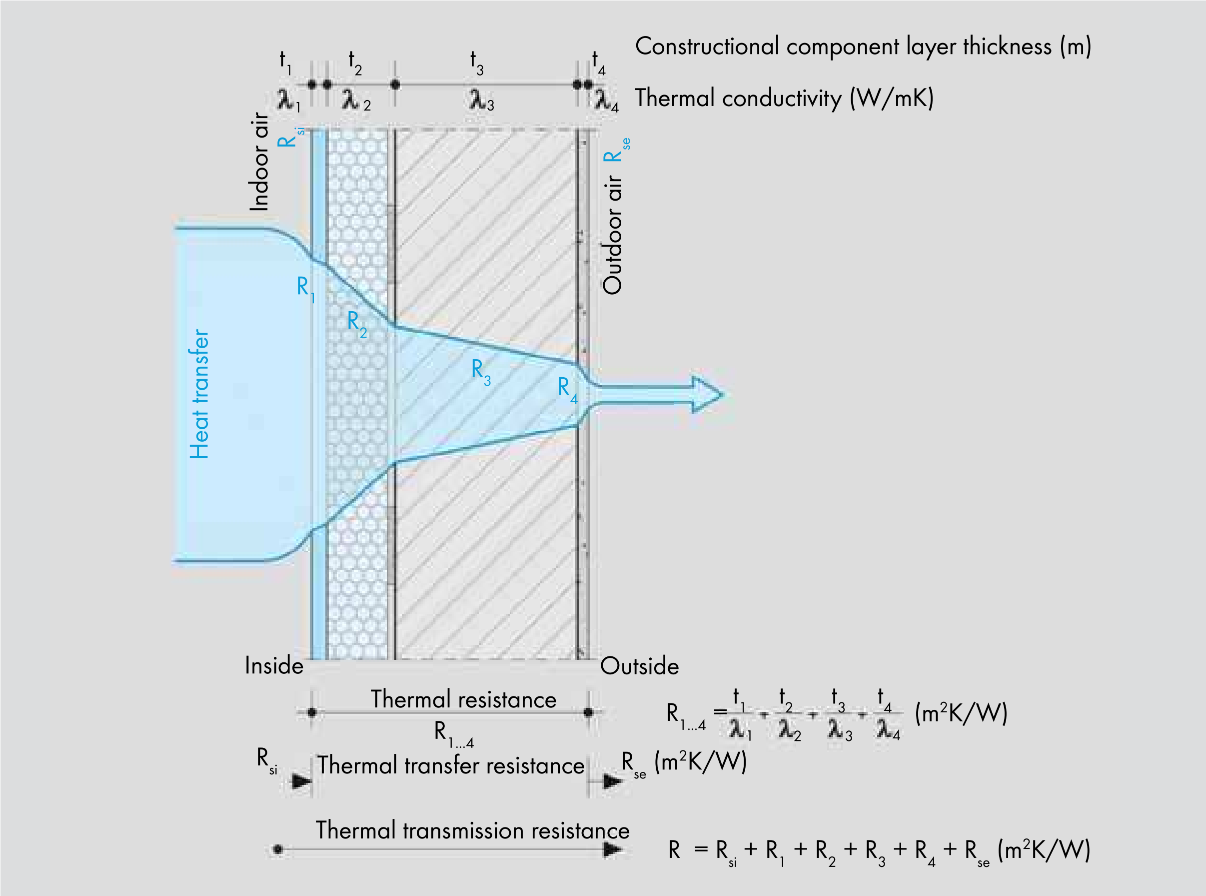


Fig. 6.22: Representation of the technical heat properties using the temperature progression in a constructional component /Knauf Gips KG/

characterized by its thermal conductivity λ in W/(mK). The smaller the thermal conductivity, the better the thermal insulation.

The thermal conductivity λ of typical materials used in lightweight building can be found in a summary in Tab. 6.17.

For the thermal conductivity of the "classic" insulation materials (e.g. mineral wool MW, polystyrene EPS / XPS, polyurethane rigid foam PUR, phenolic resin foam PF, foam glass CG, wood wool WW, wood fibres WF), the standards EN 13162 to EN 13171 and the EN 12524 generally apply.

Thermal resistance R', thermal transfer resistance, thermal transmission resistance and heat transfer coefficient

The heat transmission of a component occurs in three phases. The following calculation variables are used for calculation:

- Thermal transfer from the room air to the component surface, designated as the thermal transfer resistance R_{si}
- Heat transfer through the component defined by the thermal resistance $R_{1..n}$
- Thermal transfer R_{si} from the exterior building component surface to the external air, designated with the thermal transfer resistance R_{se}

Using these variables, the thermal transmission resistance R and the thermal transmission coefficients U are calculated. Fig. 6.22 shows the relationship between these variables.

The thermal transmission resistance R is the thermal protection quality factor of a building component and is specified in m²K/W.

As a property for the heat relevant performance of a constructional component, however, the thermal transmission coefficient U (W/(m²K)) is used for better comprehension. This value is the reciprocal value for the

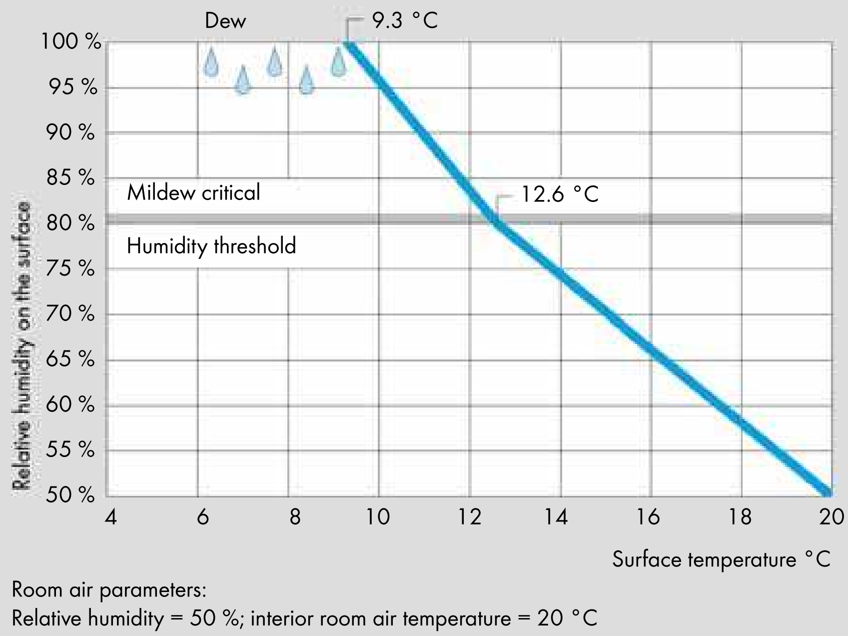


Fig. 6.23: Mould critical humidity threshold according to DIN 4109-2 for heated interiors /Knauf Gips KG/

thermal transmission resistance (U = 1/R). It designates the interesting factor with thermal insulation measures, chiefly the heat losses with respect to 1 m² component surface at a temperature difference of 1 K between the interior and exterior constructional component.

This results in:

The lower the U value of a constructional component, the lower the heat losses via the constructional component.

The calculation of the U value is internationally standardized according to ISO 6946. For stud partition walls on the façade, the U values are determined in accordance with ISO 10211 and EN 13947. The influences on the metal studs in the entire U value of the constructional component are considered during the calculation.

Room side component surface temperatures

In order to avoid an accumulation of water and the formation of mould and mildew on the room side surface of exterior building components, a minimum surface temperature must be assured. The standards require that a proof with specific temperature and air humidity framework conditions (heated rooms with an interior temperature of 20°C in conjunction with 50 % relative room humidity to the exterior air or to the unheated attic – 5°C or to the unheated cellar 10°C) that the surface temperature Θ_{si} in the heated room may not be below 12.6°C (Fig. 6.23).

Thermal bridges

Thermal bridges are locally limited thermal weak points in the thermal transfer shell of the structure. In comparison to component surfaces not affected, there is an increased heat flow here.

According to Fig. 6.24, thermal bridges can be /6.6/:

- Geometrically related thermal bridges such as building corners
- Material related thermal bridges such as building areas with poor thermal insulation materials
- Convective thermal bridges due to leaks

Thermal bridges can be reduced by forgoing the use of strongly divided components (e.g. support brackets) as well as the provision of continuous insulating levels without weak points as well as airtight construction with lightweight construction particularly in the connection areas to other components.

In order to avoid thermal bridges in the exterior wall constructions, general compliance to the rules as set down in ISO 10211 is required. Metallic components must be decoupled by suitable decoupling strips with low thermal conductivity from the connection components.

Moisture protection

The following designs relate to moisture protection measures to prevent the formation of condensation on the constructional component surface or on the building element.

Moisture protection against condensation on the room side surface

By the provision of sufficient thermal protection and avoidance of thermal bridges, it can be ensured that on the whole interior room wall a temperature exceeding the room air dew point temperature is set and avoid the formation of condensation on the interior wall (refer also to "Surface temperature").

Moisture protection against condensation in the component cross-section through water vapour diffusion

In winter, a water vapour partial pressure gradient from the interior to the exterior generally exists. This involves the vapour transport through self-contained component layers due to the existing water vapour pressure gradient between the component surfaces. Condensation is formed where there is a sufficient existing temperature gradient in the air of the component design when the vapour saturation point is reached.

An impermissibly high level of condensation in construction due to water vapour diffusion can be prevented by a correct diffusion-relevant component design, e.g. by coordinated arrangement and rating of vapour retarders.

A more precise rating of the moisture content of constructional components is possible using a hygrothermal simulation calculation (transient calculation of the coupled heat and moisture flows in the exterior area) software. Using these calculation methods, constructional components can be optimized taking into consideration real climatic framework conditions and including all moisture transport modes such as diffusion, surface diffusion, capillary transport, etc.

Moisture protection against condensate water in the component cross-section through water vapour convection

If the interior layers of the constructional components are not sealed airtight, lift and/or wind suction will cause the passage of heated interior air through the joints. This air will introduce humidity (water vapour) into the construction. The ingress of moisture due to convection can be multiples greater than that via diffusion as in Fig. 6.25 comparing water transport due to diffusion on 1 m² of a correctly dimensioned roof surface to ingress due to convection by a 1 m long gap of 1 mm width.

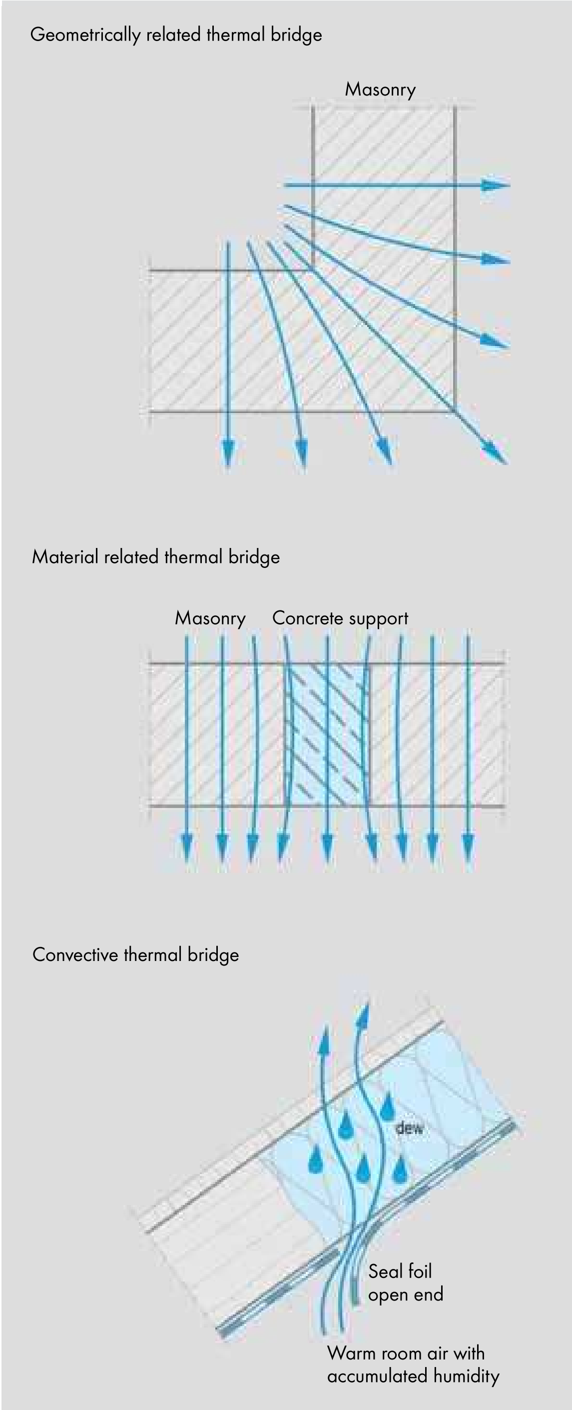


Fig. 6.24: Types of thermal bridges /Knauf Gips KG/

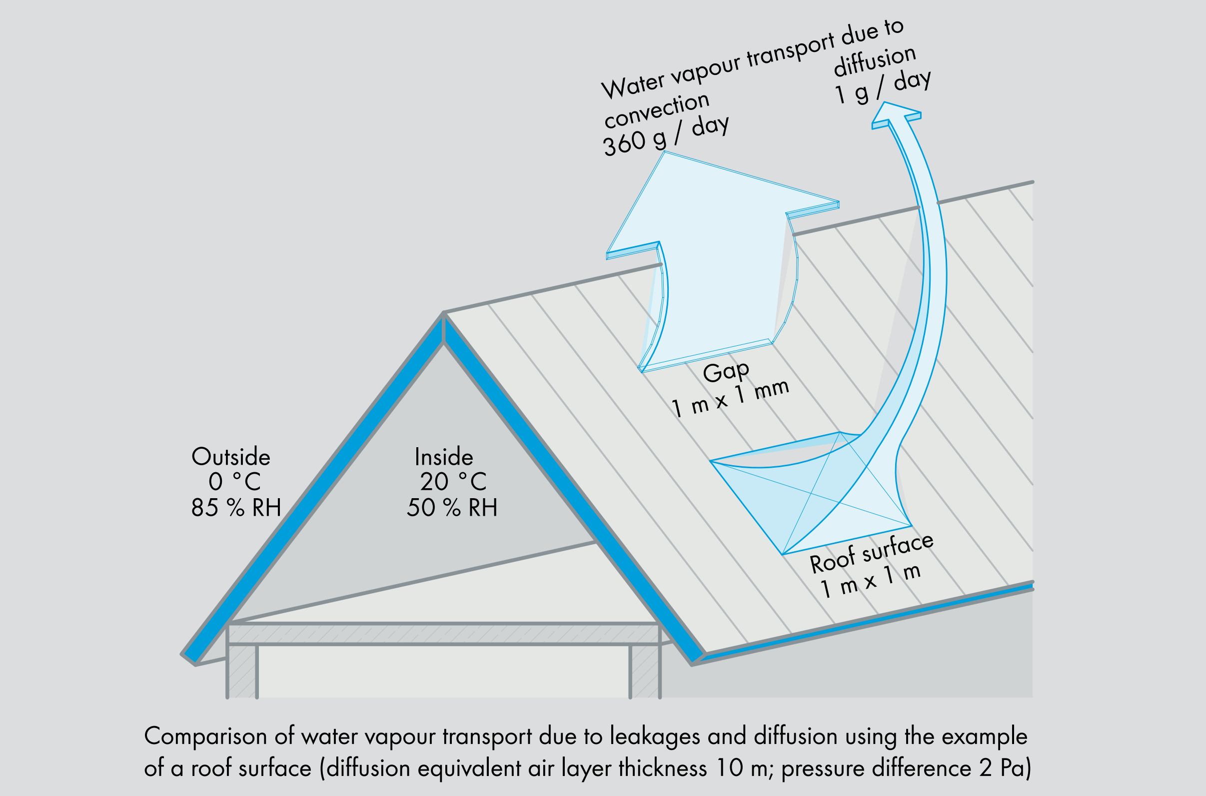


Fig. 6.25: Water vapour transport comparison of diffusion to convection /Knauf Gips KG/

Airtightness

To avoid convection flows (air flow through leaks, joints), the constructions must be configured so that they are airtight.

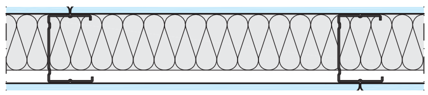
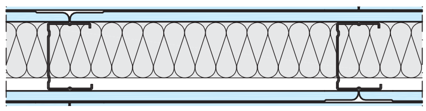
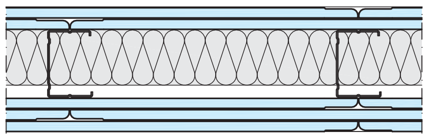
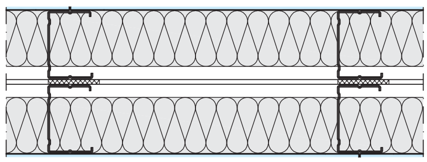
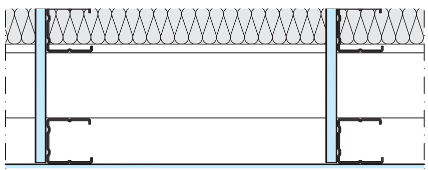
In addition to the necessity to avoid condensation damage as already set down, an airtight building envelope is also essential for the following points:

- Avoidance of energy losses
  Leakages in winter allow warm interior air and the heat energy stored in it to be lost to the exterior. Cold air that flows in to compensate must be heated up. The share of these ventilation heat losses can be up to 50 % with highly insulated buildings, and thus a high level of airtightness is just as important as sufficient thermal insulation. Ventilation systems can only perform their intended function with a sealed building shell.
- Creation of a cozy and healthy room climate
  Air flows through leaks in the building envelope from the exterior to the interior can lead to unpleasant draughts.

Proof of the airtightness is undertaken with the so-called Blower-Door test (Fig. 6.26).

Fig. 6.26: Blower door measurement for determination of the air exchange rate n_{50} /Knauf Gips KG/

Tab. 6.17: Thermal transmission coefficients U of interior walls in metal stud design, Knauf system /Knauf Gips KG/

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	12.5	40	0.66
	100	75		60	0.50
	125	100		80	0.40
W112, metal stud partition, single studs, double-layer cladding					
	100	50	2 x 12.5	40	0.61
	125	75		60	0.47
	150	100		80	0.38
W113, metal stud partition, single studs, triple-layer cladding					
	125	50	3 x 12.5	40	0.57
	150	75		60	0.44
	175	100		80	0.36
W115, metal stud partition, double studs, double-layer cladding					
	155	105	2 x 12.5	2 x 40	0.37
	205	155		2 x 60	0.27
	255	205		2 x 80	0.21
W116, installation wall, double studs, double-layer cladding					
	≥ 220	≥ 170	2 x 12.5	40+60	0.34

1) Gypsum boards in accordance with EN 520 with a thermal conductivity of 0.25 W/(m·K) in accordance to EN 12524

2) Insulation with thermal conductivity 0.04 W/(m·K)

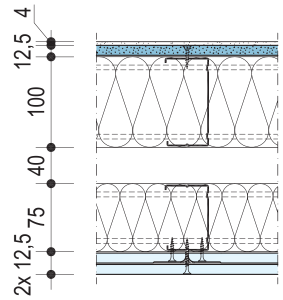
The air sealing level shall be arranged on the interior in conjunction with drywall constructions before the thermal insulation without any defects or skips. It is generally implemented with foil that simultaneously acts as a vapour retarder. Foil edges are generally bonded using suitable adhesives (adhesive beads) or adhesive tapes. The connections on the surrounding components are generally carefully applied with ageing-resistant sealing adhesives or tapes.

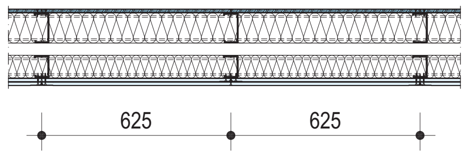
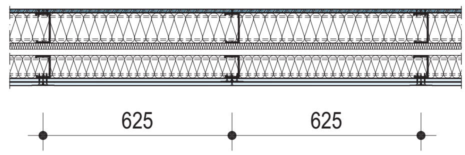
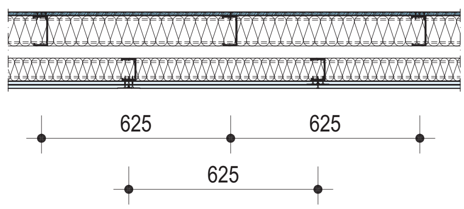
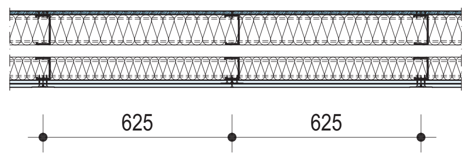
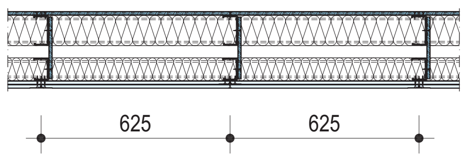
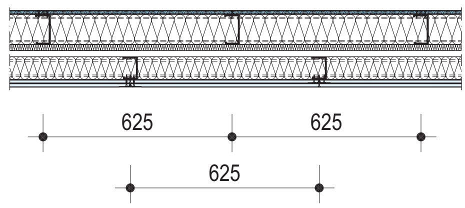
Gypsum boards and cement boards are airtight in terms of the standard. The board joints must be filled crack-free and airtight. For connections to surrounding enveloping components, a flexible connection seal made of foil strips is recommended.

6.3.2 Heat transfer coefficient of selected wall constructions

The thermal transmission coefficient U for interior walls and façade constructions in metal stud design with gypsum boards or in the combination gypsum and cement boards for outer wall application should be taken for example from Tab. 6.17 and 6.18. In both

Tab. 6.18: Thermal transmission coefficients U of exterior walls in metal stud design, Knauf system Aquapanel /Knauf Aquapanel GmbH/

	U value with profile	U value without profile	ψ value	Details
	W/m²K	W/m²K	W/mK	
01 Standard construction	0.28	0.18	0.0635	
02 Construction with alternating profiles	0.25	0.18	0.0415	
03 Construction with linked profiles	0.29	0.18	0.0667	
04 Construction with insulating layer	0.22	0.16	0.0396	
05 Construction with slotted profiles	0.22	0.18	0.0259	
06 Combination of a construction from alternating profiles 02 and insulating layer 04	0.20	0.16	0.0280	

	
01 Standard construction	04 Construction with insulating layer
	
02 Construction with alternating profiles	05 Construction with slotted profiles
	
03 Construction with linked profiles	06 Combination of a construction from alternating profiles 02 and insulating layer 04

application cases, the U value is mainly determined by the thermal transmission value and the thickness of the insulating layer.

6.3.3 Standard-based and statutory requirements for thermal and moisture protection

Requirements of the thermal protection on the one hand relate to a basic level of thermal performance to assure the harmlessness of constructional components and on the other hand as statutory "instruments" for saving energy and reduction of CO_2 emissions to protect the environment.

Energy conservation as an environmental protection measure is set down on a European level on the basis of the European directive on the energy performance of buildings (2002/91/EC) and the European buildling regulation based on it (2010/31/EU) /6.7/. The member states of the EU are obliged to implement national energy directives in their respective countries.

7 Sustainability and integral design

Maria Founti

"Sustainable development meets the needs of the present without compromising the ability of future generations to meet their own needs" /7.1/
The construction and use of buildings in the EU account for about half of EU extracted materials /7.2/, about half of the energy consumption /7.3/ and about a third of water consumption /7.4/. Furthermore, the construction sector generates about one third of all EU waste /7.5/. Environmental pressures arise at different stages of a building's life cycle including the manufacturing of construction products, building construction, use, renovation and the management of a building.
This chapter aims to provide an overview of the requirements for the sustainability assessment of structures. Environmental assessment methods are briefly reviewed. Emphasis is placed on the integral assessment of environmental sustainability, resilience and seismic safety of lightweight gypsum drywall construction. The example of a prototype anti-seismic demonstration building is given together with its overall environmental assessment.

7.1 Sustainability fundamentals: Drywall construction

7.1.1 Sustanability concepts

The overall aim of sustainable construction is to promote a more efficient use of resources consumed by buildings (new and renovated commercial, residential and public) and to reduce their overall environmental impacts throughout the building's life cycle. The sustainability of a structure can be approached via a "multi-performance based approach" (Tab 7.1), which should include optimization of durability and of the mechanical, economic and environmental performance during the whole life-cycle /7.6/.

Environmental, economic, health and community benefits can be associated with sustainable construction. Environmental benefits include reduced energy and water consumption, reduced waste disposal and improved air and water quality. Economic benefits address reduced building operational costs, lower maintenance costs and increased revenue (sale price or rent). Health and community benefits include enhanced occupant comfort and health as well as reduced liabilities.

A sustainable design method should include:

- Concept definition; design for required performance, service life, life cycle impacts
- Definition of building performance and life cycle requirements using quantitative methodologies
- Assessment of conformity with the pre-defined requirements

Sustainability indicators are a useful means that allow comparative assessment of alternative concepts and systems. They should be widely accepted, should be impartial – not favouring any specific solutions, should reflect the inherent characteristics of the sustainable design methodology but also take into consideration user needs and requirements.

An example of a sustainability indicator is the "minimum duration of a building's life cycle". Proposed values for a sustainable building, taking into account the design stage with adequate emphasis placed on adaptability throughout the design stage of a building and the quality of construction on site, are of the order of Tab. 7.2 /7.7/.

Tab. 7.1: Requirements for a "multi-performance based approach" for sustainable construction

Requirements for a "multi-performance based approach" for sustainable construction	
Environmental	Utilization of construction materials with low embodied energy and which promote low energy consumption during the service life High recycling rates of structural components Reduced energy and water consumption Reduced waste disposal
Social	Hygiene, health, indoor comfort conditions Mechanical resistance and stability Safety in case of fire Safety in use Protection against noise Structural resistance Robustness and resilience
Economic	Raw material cost Production costs Reduced construction costs Reduced operational costs Reduced maintenance costs Increased revenue

Tab. 7.2: Minimum duration of a building's life cycle

Design stage	**Duration**
Structure	100 – 200 years
Building fabric	50 – 100 years
Services	20 – 30 years
Furniture & fittings	10 – 20 years

Consumption of resources and related environmental impacts throughout a building's life cycle can be reduced by:

- Promoting a better design that weighs resource use against the needs and functionality of the building and considers scenarios for deconstruction
- Better project planning, which ensures a greater use of resource and energy efficient products
- Promoting more resource efficient manufacturing of construction products by, for example, using recycled materials, reusing existing materials and using waste as a fuel
- Promoting more resource efficient construction and renovation by, for example, reducing construction waste and recycling/re-using materials and products, so that less is sent to landfill

Gypsum drywall construction is essentially sustainable, since it is based on an eternally recyclable material (gypsum) and simultaneously:

- Offers good seismic and fire safety
- Offers shorter construction time
- Reduces the time that scaffolding needs to remain in place by 2 to 3 weeks compared to solid construction; this corresponds to relative cost savings of 18 % – 24 %. The shorter construction time and improved safety of gypsum drywall construction reduces costs for insurance. Shortening the construction period allows renting the building faster and, thus, an earlier return on investment is possible
- Offers easy conversion and change of type of use, substitution and introduction of new systems
- Gypsum boards can be selectively deconstructed at the end of their life cycle and can be recycled

Tab. 7.3 Social advantages of gypsum drywall construction

Social advantages of gypsum drywall construction	
Quality of life, health and safety of the citizens	Safety, comfort, durability and low life costs are major keywords of anti-seismic gypsum drywalls. Gypsum drywall steel-frame anti-seismic construction can provide housing solutions in high risk regions (due to earthquakes, violent attacks, climatic disasters, strong building vibrations, large scale fires etc.), where there is need for "fast" solutions (buildings must be constructed in a very short time), "safe" solutions (post-disaster events affecting population psychology) as well as offering a permanent solution reducing costs. In addition, modern concepts of the home and its uses are directly linked to innovative advanced technologies, such as those addressed by this book. The forecasted increase in citizen mobility and changes in nature of employment will necessarily lead to additional changes in housing habits and requirements.
Contribution to working conditions, employment, training, education	Modular housing offers a huge advantage with regard to working conditions for the workers and production efficiency: Improved safety at work (potentially less accidents at work site, since activities are moved to the factory); reduction of noise-waste-work risk at sites and better aesthetic view of construction sites (due to shift of activities); independence from weather conditions.

7.1.2 Advantages of lightweight steel construction

Advantages of lightweight steel constructions compared to solid construction with regard to seismic performance are:

- Low dead load (= lower earthquake loads)
- Effective coupling of soft and rigid structures to reduce resonance effects
- In contrast to solid constructions, lightweight gypsum drywall constructions support earthquake safety
- Ductile deformation behaviour prior to collapse; infill masonry walls exhibit brittle and comparatively rigid deformation patterns that cause significant load transfer with dangerous, brittle and unannounced collapse that can even lead to total building collapse
- Preservation of enclosing function even after severe structural damage
- Noise and low frequency ambient vibration insulation
- Gypsum drywall materials and systems are a major advantage in re-modelling and renovation
- Flexible for re-decorations. Buildings are easier to adjust to the requirements
- Less weight leads to less energy consumption and saves resources
- Lightness has a positive impact on the stability and therefore the quality of the construction

Gypsum drywall construction is global; independent of climatic and geological restrictions. A sustainable gypsum drywall building has low consumption of natural resources (FGD and/or recycled gypsum are included in gypsum board production), can ensure seismic and fire safety and can control the release of harmful emissions. A variety of External Thermal Insulation Composite Systems (ETICS) and insulation systems can be used in drywall construction that are able to create the desired building performance accounting for user needs and requirements stated by the owner.

The good environmental performance and the competitive advantages of gypsum drywall construction as well as their social advantages (see Tab. 7.3) are beneficial for architects, designers, manufacturers of construction products, builders, developers and investors. Environmental assessment of gypsum drywall buildings can provide detailed information on product and building

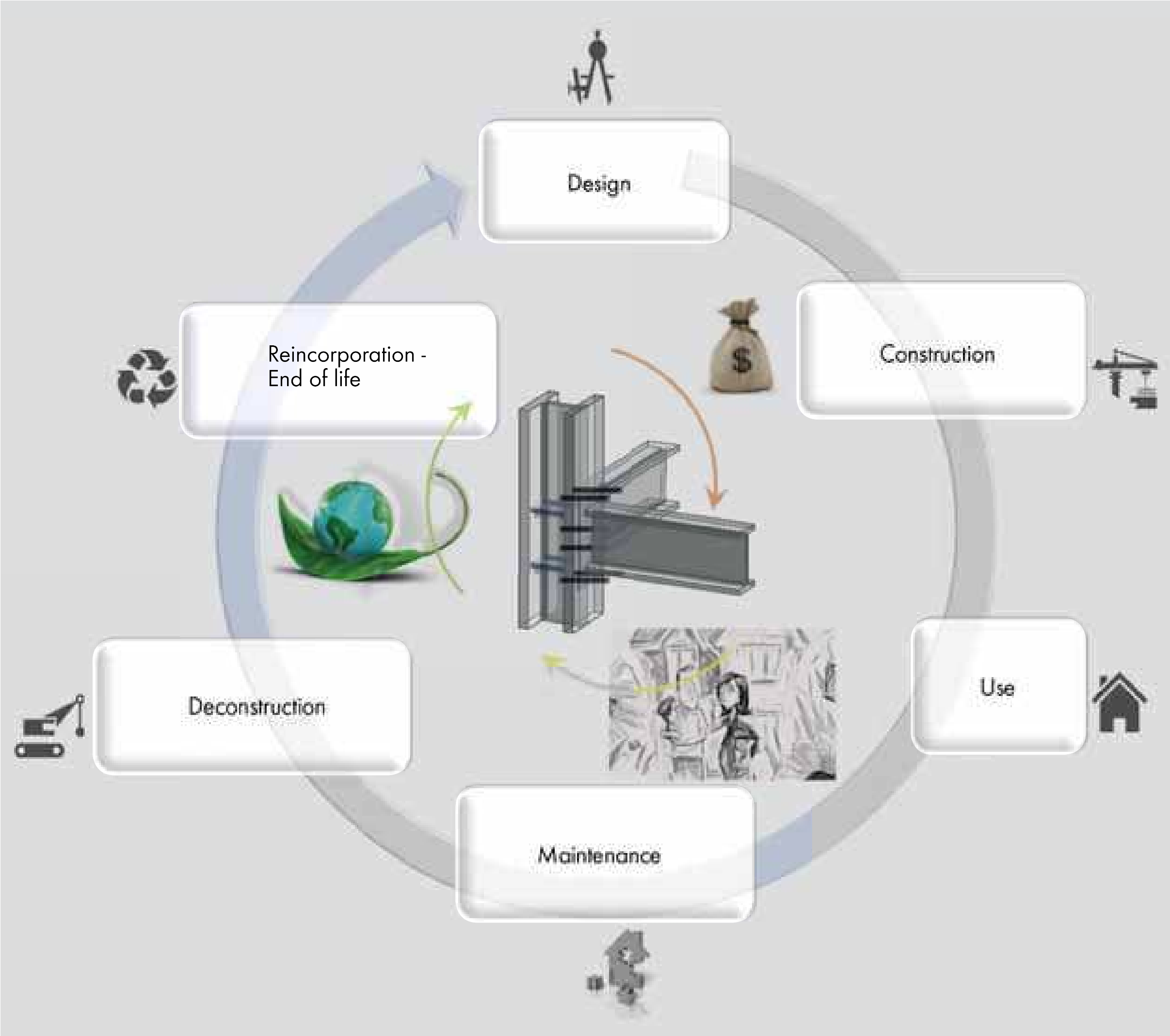


Fig 7.1: Life cycle stages of a building /http://www.aia.org, *Landolfo, R./*

level that may lead to reduced costs when incorporating sustainability aspects. Developers will be able to compare performance of projects more easily. Investors, property owners and insurers will be able to improve the allocation of capital and to integrate environmental risk into their decisions.

7.1.3 Environmental assessment methods

Many countries throughout the world have established specific requirements for "green buildings", and governments have adopted relevant specific regulations and directives aiming to promote sustainability of building construction and throughout its life cycle.

There are several levels of assessment of a "green" building. Components (such as façades, roofing, structural elements) can be described by an Environmental Product Declaration (EPD). Energy efficiency is either regulated or labeled. Building certifications evaluate the complete building (e.g. BREEAM®, LEED®) and might integrate social and economic aspects.

A full building Life Cycle Assessment (LCA) evaluates the impacts of the complete life cycle of a building. LCA and sustainability indicators can help quantify the performance and life cycle requirements of any type of building. They can act as guidelines towards design for a pre-specified building performance and life cycle and they can support decisions between alternative options accounting for environmental and economic parameters.

Compliance to different environmental standards and regulations is required, depending on the type of study. For example:

- Generic LCA studies: ISO 14040 – 44

- Generic environmental declaration: ISO 14025
- Environmental declaration for construction products world-wide: ISO 21930
- Environmental declaration for construction products in Europe: EN 15804

Building 'Life cycle'

The life cycle stages of a building (Fig. 7.1) /7.8/ are:

- Materials manufacturing: Extraction of raw materials from earth, transportation of materials to the manufacturing locations, manufacturing of finished or intermediate materials, building product fabrication, packaging and distribution of building products. Especially for gypsum board production, the material manufacturing incorporates closed-loop manufacturing processes, use of recycled products (such as flue gas desulphurization gypsum – FGD, recycled paper liner, etc.), recovery of manufacturing waste as well as of construction and demolition waste, reduction of water and energy usage
- Design and construction: All activities relating to the actual building project construction
- Use and maintenance: Building operation including energy consumption, water usage, environmental waste generation, repair and replacement of building assemblies and systems, and transport and equipment use for repair and replacement
- Deconstruction / Dismantling / Reincorporation and End of life: Includes energy consumed and waste produced due to building demolition and disposal of materials to landfills and transport of waste materials. Recycling and reuse activities related to demolition waste can also be included and generally have a "negative (beneficial) impact"

In Europe, there are several European Union directives and regulations, such as the Energy Performance of Buildings Directive /7.9/, the Energy Efficiency Directive /7.10/, the Construction Products Regulation /7.11/, the EU Emissions Trading System /7.12/, the Industrial Emissions Directive /7.13/, the Waste Framework Directive /7.14/ and the Landfill Directive /7.15/ that focus on different resources and parts of the life cycle and, for the time being, they are not designed to provide an overall life cycle approach.

Indicative similar U.S.A regulations include the High Performance and Sustainable Buildings Guidance /7.16/, the Energy Independence and Security Act of 2007 /7.17/ and the Executive Order 13514 /7.18/. The United States Green Building Council (USGBC) and the National Association of Home Builders (NAHB) actively support product ratings, assessment tools and practical guidelines to ensure practical achievement of sustainability.

Environmental assessment tools

The majority of existing building environmental assessment tools (Tab. 7.4) evaluates environmental performance of buildings relative to explicitly declared or implicit benchmarks /7.19/. Typically, the environmental performance is described with the help of indicators, which try to express both the environmental impacts as well as other performance aspects, such as indoor conditions /7.20/. Widely used certification systems based on a criteria system are BREEAM® (UK), LEED® (USA) and the GBTool® (Canada). Bees (USA), envest2 (UK) and ATHENA (Canada) are based on the LCA methodology.

The approaches and associated methodologies gradually widen their scope from pure environmental assessment to overall assessment of sustainability aspects of buildings taking into account design, technical, economic, environmental and social aspects.

Life cycle assessment

Life cycle assessment (LCA, also known as life cycle analysis, eco-balance, and cradle-to-grave analysis) /7.21/ is a technique to assess environmental impacts associated with a product, process or activity by identifying energy and materials used and wastes released to the environment, and to evaluate and implement opportunities to affect environmental improvement. LCA assesses environmental impacts associated with all the stages of a product's life from-cradle-to-grave, i.e., from raw material extraction through material processing, manufacture, distribution, use, repair and maintenance, and disposal or recycling. In Europe, the EN ISO14040-44: 2006 defines

Tab. 7.4: Environmental assessment tools

Environmental assessment tools	Institution	Country	Web page
SBTool	International Initiative for Sustainable Building	International	http://www.iisbe.org/sbmethod
BREEAM	BRE	United Kingdom	http:/www.breeam.org
Envest2	BRE	United kingdom	http://envest2.bre.co.uk/detailsLCA.jsp
DGNB	German Sustainable Building Council	Germany	http://www.dgnb.de/_de/
Ecoprofile	Norwegian Building Research Institute	Norway	http://www.sintef.no/home/
HQE	Association pour la Haute Qualité Environmentale des bâtiments	France	http:/www.assohqe.org/hqe
Nordic Swan	Nordic Council of Ministers	Nordic countries	http://www.svanen.se/
MINERGIE	Minergie Building Agency	Sweden	http://www.minergie.ch/
PromisE	Green Building Council Finland	Finland	http://www.promiseweb.net/
Protocolo ITACA	Instituto per L´Innovazione e Trasparenta degli Appalti e la Compatibilitá Ambientale	Italy	http://www.itaca.org/
Verde	GBC España	Spain	http://www.gbce.es/herramientas/informacion-general
LIDERA	Departamento de Engenharia Civil e Arquitectura do Instituto Superior Técnico	Portugal	http://www.lidera.info
LEED	US Green Building Council	USA	http://www.usgbc.org/LEED/
Bees	NIST	USA	http://ws680.nist.gov/bees/
ATHENA	The ATHENA Institute	Canada	http://www.athenasmi.org/what-we-do/lca-data-software/
Green Globes	Building Owners and Managers Association of Canada (BOMA)	Canada	http://www.greenglobes.com
NABERS	NSW (New South Wales Government)	Australia	http://www.nabers.com.au
Green Star	Australia Green Building Council	Australia	http://www.gbca.org.au/
CASBEE	Japan Green Building Council	Japan	http://www.lbec.or.jp/CASBEE/english/index.htm
EEWH	Taiwan Green Building Council	Taiwan	http://www.taiwangbc.org.tw/en/
Green Mark	Singapore Building and Construction Authority (BCA)	Singapore	http://www.bca.gov.sg/GreenMark/green_mark_buildings.html
HK BEAM	HK BEAM Society	Hong Kong	http://www.beamsociety.org.hk
SBAT	Council for Scientific and Industrial Research (CSIR)	South Africa	http://www.csir.co.za/

Life Cycle Assessment as the "Compilation and evaluation of the inputs, outputs and the potential environmental impacts of a product (and/or service) system throughout its life cycle."

The LCA procedures are part of the ISO 14000 environmental management standards: ISO 14040:2006 "LCA – Principles and Framework" without requirements and 14044:2006 reviewed in 2010 – "LCA-Requirements and guidelines" with all requirements (ISO 14044 replaced earlier versions of ISO 14041, ISO14042 and ISO 14043.) Greenhouse Gases (GHG) product life cycle assessments can also comply with standards such as PAS 2050 and the GHG Protocol Life Cycle Accounting and Reporting Standard. /7.21, 7.23, 7.24/

The four characteristic stages of an LCA are:

- Goal and Scope definition (G&S)
- Life Cycle Inventory (LCI)
- Life Cycle Impact Assessment (LCIA)
- Life Cycle Interpretation (LCIN).

There are two main types of LCA. Attributional LCAs seek to establish the burdens associated with the production and use of a product or with a specific service or process at a point in time (typically the recent past). Consequentially, LCAs seek to identify the environmental consequences of a decision or a proposed change in a system under study (oriented to the future), which means that market and economic implications of a decision may have to be taken into account. Social LCA is under development /7.22/ as a different approach to life cycle thinking intended to assess social implications or potential impacts. Social LCA should be considered as an approach that is complementary to environmental LCA.

The LCA contributes towards avoiding a narrow outlook on environmental concerns by:

- Compiling an inventory of relevant energy and material inputs and environmental releases
- Evaluating the potential impacts associated with identified inputs and releases
- Interpreting the results to help make a more informed decision /7.23/

The use of LCA for buildings requires a set of guiding principles, which consider the unique character of each building design, complexity in defining systems, and related decisions. At the design stage, LCA addresses the selection among different design options and it helps to identify the life cycle stages associated with maintenance, repair and rehabilitation of components

Life cycle cost

Life cycle cost (LCC) refers to the total cost of ownership over the life of building /7.24/, also commonly referred to as "cradle to grave" or "womb to tomb" costs. ISO 15686-5:2008 (Buildings and constructed assets – Service-life planning – Part 5: Life-cycle costing) gives guidelines for performing life cycle cost (LCC) analyses of buildings and constructed assets and their parts.

The main objective of LCC is to minimize the sum of the life cycle costs, in current values, thus benefiting both owner and end users. LCC aims at the optimization of the design granting better results in extended life, performance and sustainability, avoiding over design and excessive waste.

Current approaches estimate only the direct costs for construction and maintenance. LCC extends the analysis over the whole life of the project, showing the real value of the investment.

Costs considered include the financial cost which is relatively simple to calculate and also the environmental and social costs which are more difficult to quantify and assign numerical values. Typical areas of expenditure which are included in calculating the whole life costs include planning, design, construction and acquisition, operation, preventive maintenance, repair (and sometimes deconstruction/demolition), depreciation and cost of finance of a building. Failure costs accounting for inspection, e.g. as a result of severe damage, can be included in the maintenance and repair costs.

Life cycle performance

Life Cycle Performance (LCP) targets the evaluation of the structural performance during the building's life cycle. LCP can be analyzed in accordance with ISO 13823:2008. The ISO specifies general principles and recommends procedures for the verification of the durability of structures subject to known or foreseeable environmental actions, including mechanical actions, causing material

Tab. 7.5: LCI and LCA results of the Eurogypsum study (2009)

LCI Assessment results per m² of gypsum board	
CO_2 (kg/m²)	1.8
CO (kg/m²)	0.00063
NOX (kg/m²)	0.00278
SO2 (kg/m²)	0.00289
N2O (kg/m²)	0.00007
CH4 (kg/m²)	0.00403
NMVOC Non-methane volatile organic compound (kg/m²)	0.00062
PM Particulate matter (kg/m²)	0.00023
Primary energy (MJ/m²)	34
Primary energy renewable (MJ/m²)	1.9
Water (kg/m²)	11.77
LCA results per m² of gypsum board	
ADP Abiotic depletion potential (kg Sb-eq./m²)	0.01483
FAETP Freshwater Aquatic Ecotoxicity (kg DCB-eq./m²)	0.00347
MAETP Marine Aquatic Ecotoxicity (kg DCB-eq./m²)	55.5
EP Eutrophication potential (kg PO4-eq./m²)	0.00080
HTP Human Toxicity Potential (kg DCB-eq./m²)	0.06203
ODP Ozone depletion Potential (kg R11-eq./m²)	1.6E-07
POCP Photochemical Ozone Creation Potential (kg ethene-eq./m²)	0.00044
TETP Terrestrial Ecotoxicity (kg DCB-eq./m²)	0.00335
GWP Global warming potential (kg CO_2-eq./m²)	2.14
GWP Global warming potential (kg CO_2-eq./m²)	0.0050

degradation leading to failure of performance. LCP evaluates the period of time during which a structure or any component is able to achieve the structural performance requirements defined at the design stage with an adequate degree of reliability.

ISO 13823:2008 specifies general principles and recommends procedures for the verification of the durability of structures subject to known or foreseeable environmental actions, including mechanical actions, causing material degradation leading to failure of performance. It is necessary to ensure reliability of performance throughout the design service life of the structure. Fatigue failure due to cyclic stress is not within the scope of ISO 13823:2008. LCP offers comparison of durability requirements (actual capacity considering deterioration as a result of a failure event) at the same design level that is currently used for ordinary mechanical design (e.g. limit state method, probability based design).

Gypsum board LCA – Gypsum drywall recycling Eurogypsum LCA study

In 2009, Eurogypsum (European federation of national associations of gypsum products manufacturers) (www.eurogypsum.org) carried out an LCA for gypsum boards, in compliance with ISO 14040. The environmental product declaration of gypsum board was based on data from the ELCD (European Reference Life Cycle Database 3.0) /7.25/ and was performed by PE international. The functional unit was 1 m² of gypsum board, and the values of the different impacts were calculated using the software GABI 4. The study used a cradle-to-gate approach, including the impacts related to raw material extraction, transport of raw materials and production. The end-of-life recycling stage was not included. The Life Cycle Inventory (LCI) data and the Life Cycle Assessment (LCA) results are shown in Tab. 7.5.

Tab. 7.6: Indicative WRAP impact assessment results. All figures assume low transport, 50 km or less between point of collection and point of use /WRAP/

Impact Category	Unit	Baseline	15 % recycled content	25 % recycled content
Global Warming (GWP100)	kg CO_2 eq.	11.90	11.68	11.45
Human Toxicity	kg 1,4-DB eq.	2.43	2.40	2.33
Eutrophication	kg PO_4^{3-} eq.	0.0043	0.0043	0.0041

Tab. 7.7: Relative savings between baseline gypsum board and 25 % recycled content gypsum board (WRAP)

Impact Category	Unit	Savings per sheet of Type A gypsum board	Savings as a % of total impacts for the baseline system
Global Warming (GWP100)	kg CO_2 eq.	0.769	6 %
Human Toxicity	kg 1,4-DB eq.	0.09	4 %
Eutrophication	kg PO_4^{3-} eq.	0.00020	5 %

WRAP LCA report

In January 2007, WRAP /7.26/ - a UK government body, which funds and carries out waste mitigation research, commissioned Environmental Resources Management Ltd to carry out a Life Cycle Assessment (LCA) for gypsum board. The aim was to quantify the environmental impact of incorporation of recycled gypsum in gypsum boards. The study encompassed all life cycle stages from raw material production to end-of-life management. It focused on the closed-loop recycling of gypsum from post-consumer sources back into gypsum board production.

Four cases were assessed:

- Baseline - based on the current (2007) mix of gypsum used in Type A gypsum board production (12.5 mm thick; 1200 x 2400 mm; square edge profile)
- 15 % recyclate - based on increased levels of post-consumer recycled gypsum (to a maximum of 15 % total recycled gypsum content)
- 25 % recyclate - based on increased levels of post-consumer recycled gypsum (to a maximum of 25 % total recycled gypsum content)

Tab. 7.6 /7.27/ shows results from the WRAP study for three main life cycle parameters:

- Global warming potential (GWP), which indicates the effect on climate change, expressed in CO_2 equivalent
- Human Toxicity, the effect on individuals in the population, expressed in equivalents of the toxic chemical compound 1,4 dichlorobenzene
- Eutrophication, which describes the effect on rivers and lakes in terms of excessive growth of algae, expressed in phosphate equivalents

The impact profiles generated from these three scenarios suggested that while environmental benefits can be achieved through incorporating post-consumer recycled gypsum into gypsum board, the benefits are relatively small in comparison with the overall system impacts.

The scale of savings is shown in Tab. 7.7. It can be seen that the benefits are all less than 10 % between the current product (Type A gypsum board) and the product with 25 % recycled content.

Gypsum drywall recycling

The end of life stage in an LCA of gypsum drywall systems originating from construction, and/or demolition or deconstruction sites should include either landfilling of construction and demolition waste (C&DW) and/or re-incorporation in the manufacturing process. The WRAP study estimated the potential gains in increasing the percentage of recycled material (stemming either from production or demolition/deconstruction). The position of Eurogypsum on this matter has been expressed as /7.26/:

Tab. 7.8: *Gypsum board embodied energy vs other building materials /www.bath.ac.uk/*

Material	Energy MJ/kg	Carbon kg CO_2/kg	Density kg/m^3
Bricks (facing)	8.2	0.52	1700
Bricks (common)	3.0	0.22	1700
Marble	2.0	0.112	2500
Clay tile	6.5	0.46	1900
Gypsum board	6.75	0.38	800
Gypsum plaster	1.8	0.12	1120
Ceramic tiles	9.0	0.59	2000

"There is also an environmental penalty to pay, as recycling gypsum waste is more energy intensive than using raw gypsum, because of the need to collect and transport the material, sort and purify it and remove the moisture content, which is the most energy intensive part of the manufacturing process."

However, increased recycling rates do reduce the amount of materials going to landfill and close the loop on materials and resource consumption, and as such is an important consideration in itself. This is currently examined within the frame of the Life+ European research project called GtoG: Gypsum to Gypsum, coordinated by Eurogypsum /7.20/. The main objective of the GtoG project is to change the way gypsum based waste is treated. Despite the fact that a closed loop is possible, the reality is different. The GtoG project aims at transforming the European gypsum demolition waste market to achieve higher recycling rates of gypsum waste, thereby helping to achieve a resource efficient economy. Closed loop recycling for gypsum products will only happen if /7.20/:

- Dismantling practices are applied systematically (as a standard) instead of demolishing buildings
- Sorting of waste is preferably done at source, avoiding mixed waste and contamination
- Recycled gypsum meets stringent specifications in order to be re-incorporated into the manufacturing process

Operational and embodied energy – gypsum board embedded energy

The "operational energy" is the total energy used by a building over a typical meteorological year. It necessarily constitutes one of the stages in any LCA. The energy consumption from heating and lighting in the operation phase of a building is covered by various EU regulations /7.29, 7.30, 7.31, 7.32/.

Studies show that between 5-10 % of the total energy consumption across the EU is related to the production of construction products /7.33/.

A product's embodied energy involves a series of complicated processes that aggravate environmental pollution, cause depletion of natural resources and the degradation of the Earth. The term "embodied energy" in the construction sector includes the energy needed:

- To extract minerals and raw materials from the earth
- To transport the material to the industrial plant
- To produce the final product
- To assemble, transport, install in the building
- To disassemble and put back into the environment.

An LCA that includes the above four is the primary means of computing the embodied energy of a material used in a building /7.34/. The entire life cycle of a building must be considered if the environmental impacts are to be tackled effectively. Typical figures of embodied energy for covering materials, based on a 'Cradle-to-Gate' analysis, are shown in Tab. 7.8 .

A material's embodied energy is often reflected in its price. Composite materials involving carbon fibres or ceramic compounds have a relatively high embodied energy. However, when they are appropriately used, significant amounts of energy can be saved during the product's use phase, due to their advanced physical properties, e.g., strength, stiffness, heat or wear resistance.

Buildings that are designed and constructed to reduce life cycle environmental impacts deliver direct economic benefits, such as lower operational and maintenance costs /7.28, 7.33, 7.34/ slower depreciation and a higher asset value /7.35, 7.36/. In addition, there are also positive social impacts like improved health and productivity. Currently, most certified buildings are high-end commercial and public buildings (such as prestigious hotels and offices) because of the additional administrative and certification costs, which should rather be seen in the context of the longer-term benefits /7.37/.

Sustainability indicators

Indicators are increasingly recognized as a useful tool for policy making and public communication in conveying information on countries' performance in fields such as environment, economy, society, or technological development.

In Europe, ISO TS 21929 (ISO 2006) defines a framework for sustainability indicators of buildings. The framework is based on the assumption that sustainable construction brings about the required performance with the least unfavourable environmental impact, while encouraging economic, social and cultural improvement at a local, regional and global level.

Environmental indicators address environmental aspects in terms of environmental loadings or impacts assessed on the basis of life cycle inventory or assessment. Environmental loadings are the use of resources and the production of waste, odours, noise and harmful emissions to land, water and air. "Consequential" environmental indicators are needed and used in requirements setting, design and selection of products for a sustainable building. Consequential environmental indicators express environmental impacts in terms of building performance or location either quantitatively or qualitatively.

Economic indicators indicate monetary flows connected to the building life cycle. According to the European ISO TS 21929, the following economic flows are related to the life cycle of a building:

- Investment: Site, design, product manufacturing, construction
- Use: Energy consumption, water consumption, waste management etc.
- Maintenance and repair
- Deconstruction and waste treatment
- Development of the economic value of a building
- Revenue generated by the building and its services

Social indicators of buildings are used to describe how buildings interact with issues of concern related to sustainability at the community level. Community level issues that may be relevant are for example urban sprawl, mixed land use, access to basics, availability of green and open space, attractiveness of city centres, development of brown-fields, availability of housing, social segregation, cultural quality and protection of cultural heritage, safety, noise and air quality. Social aspects can also be addressed on the building level like for example in ISO 2006:

- Quality of buildings as a place to live and work
- Building-related effects on health and safety of users
- Barrier-free use of buildings
- Access to services needed by users of a building
- User satisfaction
- Architectural quality of buildings
- Protection of cultural heritage

7.2 Integral environmental, technical-seismic safety and economic assessment

Low-to-medium rise buildings (up to 3 storeys) are the more frequent typology for housing, requiring particular attention in developing sustainable solutions for construction.

Buildings need to provide for welfare, health and safety of occupants. The occurrence of strong earthquakes in the European Mediterranean region, even in moderate seismic zones (e.g., the 2011 Lorca earthquake in

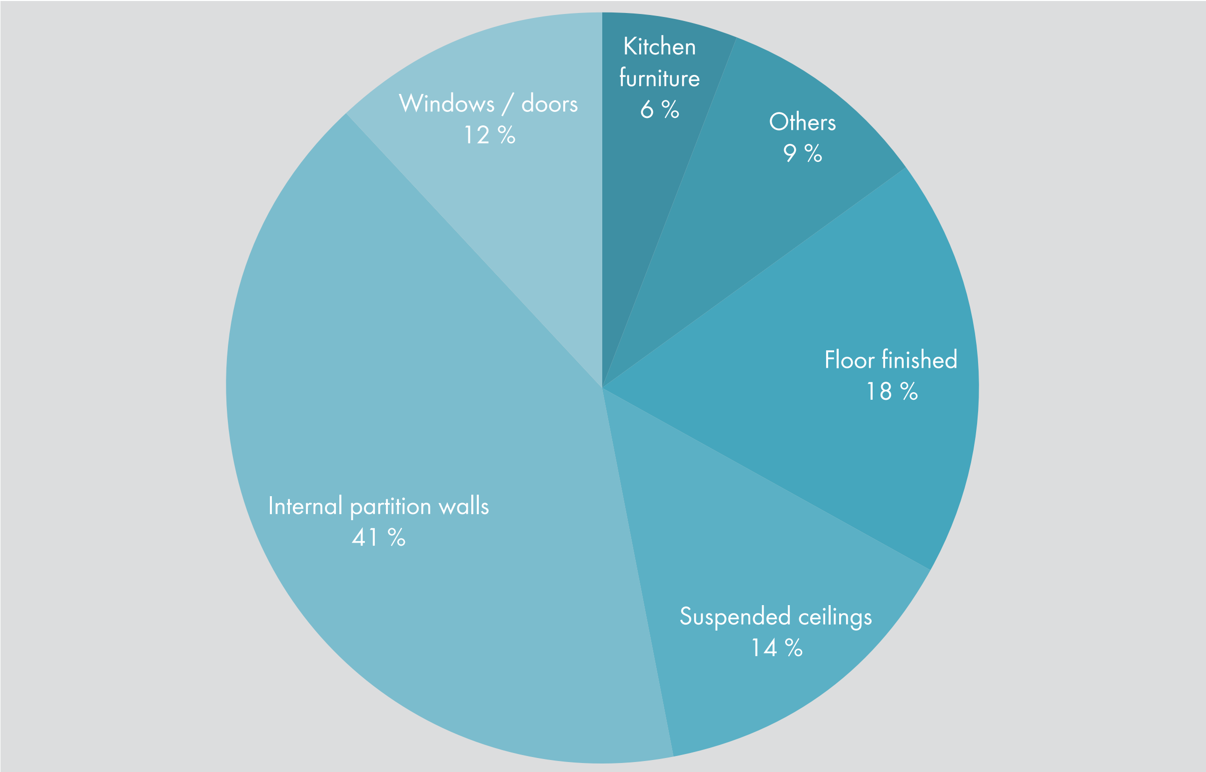


Fig. 7.2: Environmental impacts from materials use on non-load bearing construction of a typical house over 60 years /Rose, A./

Spain), highlighted the consequences of poorly designed earthquake resistant structures regarding: Damage, injured people, deaths, post-earthquake traumata and reconstruction costs. It is known that earthquakes can take place all over the world causing large losses. The seismic action needs then to be adequately considered in the design of buildings, as addressed in recent methodologies and codes for seismic safety assessment of structures /7.38/.

For seismic regions, a serious drawback of current procedures is that they neglect the impact of seismic events on a building's sustainability. For an enclosure of a building to be considered sustainable, it must have durable performance. Most enclosures are designed considering durable performance to withstand water, air and vapour intrusion and aging due to environmental loadings including wind, rain, temperature variation, exposure to sun, etc. Most do not consider seismic sustainability. The sustainability concept is often applied in the fields of construction economy or green buildings as a whole, with less emphasis on the structural typology especially in terms of earthquake resistance. Framed RC structures, given their prevalence, are commonly assumed as reference for sustainable building design. However, the optimization of the building performance in general (economy, safety, durability, etc.) calls for a broad approach to sustainability, which needs to take the structural typology into account /7.38/.

Cost-effective structural solutions can present higher vulnerability to earthquakes. Combining sustainability and earthquake resistance is a serious contemporary challenge. Extreme events should be part of the life cycle cost (LCC) decision-making for sustainable design of the building envelope and other non-structural components of buildings.

According to Rose /7.39/, Mileti /7.40/ has defined sustainability in relation to disasters in part as the ability of a community to recover by utilizing its own resources. Resilience is literally the ability of a material to absorb energy, when it is deformed elastically, and release its energy upon unloading.

A resilient structure is one that shows:

- Reduced failure probabilities
- Reduced consequences from failures, in terms of lives

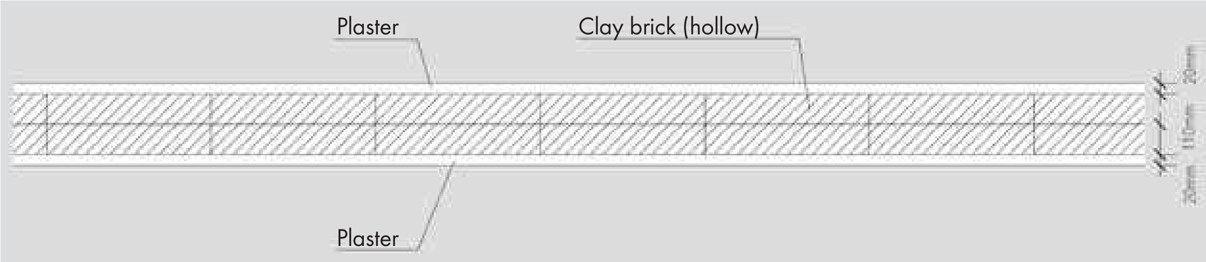


Fig. 7.3: Horizontal cross-section of a conventional heavyweight masonry partition wall /Marques, R., Paulo, B., Lourenço/

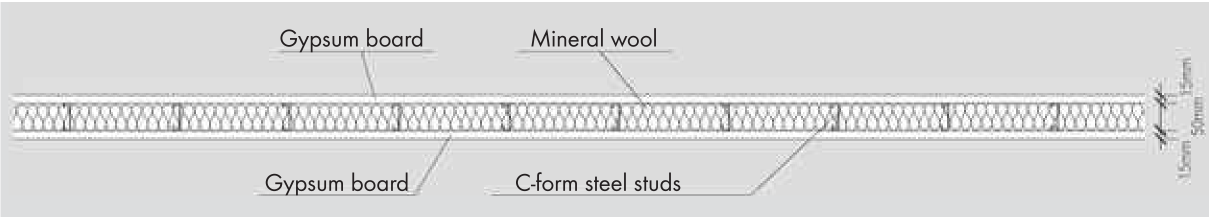


Fig. 7.4: Horizontal cross-section of a lightweight reference gypsum board partition wall /Marques, R., Paulo, B., Lourenço/

lost, damage, and negative economic and social consequences

- Reduced time to recovery (restoration of a specific system or set of systems to their "normal" level of functional performance)

Rose has defined economic resilience /7.39/:

Static economic resilience – The ability of a system to maintain function when shocked. This is the heart of the economic problem, where ordinary scarcity is made even more severe than usual, and it is imperative to use the remaining resources as efficiently as possible at any given point in time during the course of recovery. This holds the prospect of decreasing business interruption.

Dynamic economic resilience – Hastening the speed of recovery from a shock. This refers to the efficient utilization of resources for repair and reconstruction. Static resilience pertains to making the best of the existing capital stock (productive capacity), while this aspect is all about enhancing capacity. As such, it is about dynamics, in that it is time-related. Investment decisions involve diverting resources from consumption today in order to reap future gains from enhanced production.

Another important distinction is between inherent and adaptive resilience /7.39/. The former refers to aspects of resilience already built into the system, such as the availability of inventories, excess capacity, input substitution, contractual arrangements accessing suppliers of goods from outside the affected area (imports), and the workings of the market system in allocating resources to their highest value use on the basis of price signals.

7.2.1 LCA of non-load bearing gypsum drywall partitions

The life cycle environmental impacts of an internal partition wall solution result directly from the attributes of the used materials, such as the embodied energy and thermal properties and from the way the solution is built and maintained. Literature shows that partition walls have the higher contribution to the overall life cycle impacts, when compared to other non-load bearing construction elements, as presented in Fig. 7.2 /7.42/.

A relevant advantage of a lightweight gypsum drywall partition wall system is its lower thickness when compared with the heavyweight conventional system. It can be easily moved and disassembled in order to promote the flexibility in the use of the indoor space without compromising the mandatory and necessary functional requirements for a partition wall. Gypsum drywalls allow maximizing the net floor area of a building. Partition walls contribute to the internal mass of an area, thus influencing the thermal inertia of the building. From

Tab. 7.9: LCA indicators for gypsum drywall partition /Marques, R., Paulo, B., Lourenço/

Dimensions	**Indicators**	**Units**	**Methods**
Environmental	Global warming potential (GWP)	kg CO_2 equiv.	CML 2 baseline 2000 V2.04
	Depletion of the stratospheric ozone layer (ODP)	kg CFC-11 equiv.	CML 2 baseline 2000 V2.04
	Acidification potential (AP)	kg SO_2 equiv.	CML 2 baseline 2000 V2.04
	Eutrophication potential (EP)	kg PO_4 equiv.	CML 2 baseline 2000 V2.04
	Formation potential of tropospheric ozone (POCP)	kg C_2H_4 equiv.	CML 2 baseline 2000 V2.04
	Abiotic depletion potential of fossil resources (ADP_FF)	MJ equiv.	Cumulative energy demand V1.0
Functional	Airborne sound insulation (R'_w)	dBA	Meisser's analytic method
	Flexibility (F)	e	Qualitative method
	Thermal insulation (U)	W/m^2 C	Portuguese thermal code
Economics	Construction cost (CC)	€	Average market value

a thermal analysis perspective, partition walls are, in most cases, totally internal to a zone and therefore are not tested for incident solar radiation or adjacency with other parts of the building. Heat transfer through partition walls is ignored, when the partition is dividing two areas with same thermal comfort demands, i.e. two heated or two non-heated areas. Nevertheless, in some cases, for instance in partitions separating heated from non-heated areas, it is necessary to fulfil a maximum heat transfer co-efficient. In this case, a lightweight partition wall can have a similar or better performance than a heavyweight one, since its thermal insulation could be improved by placing an adequate insulation material in the space separating the two surfaces of the wall.

Sound insulation is an important requirement in the design of gypsum drywall internal walls. Although this requirement is of maximum importance in partition walls separating different dwellings, also the partition wall separating two adjacent rooms must provide a barrier for airborne sound transmission. Previous studies show that gypsum drywall lightweight partition walls can perform better than a conventional heavyweight system.

Mateus et al /7.42/ analyzed - among others - the environmental, functional and economic performances of a hollow brick partition wall (Fig. 7.3) and a gypsum drywall partition wall (Fig. 7.4). The LCA included the environmental impacts until the end of the construction phase and the impacts resulting from end-of-life scenarios. Other life cycle stages like the maintenance were excluded. The considered indicators in this study and the respective units and quantification methods are shown in Tab. 7.9. /7.42/

The environmental indicators resulting from the performed LCA /7.42/ are reported in Tab. 7.10, highlighting the gypsum drywall advantages. The global warming potential is the most significant in terms of absolute values, and the results indicate more than 50 % lower values for the lightweight gypsum drywall system in comparison to the masonry wall.

Tab. 7.10: LCA environmental indicators /Marques, R., Paulo, B., Lourenço/

Building technology	GWP (kg CO_2 eq)	ODP (kg CFC-11 eq)	AP (kg SO_2 eq)	EP (kg PO_4 eq)	POCP (kg C_2H_4 eq)	ADP (kg Sb eq)	ADP_FF (MJ eq)
Masonry partition wall	4.4E+01	3.2E-06	1.12E-01	3.19E-02	5.43E-03	1.90E-01	3.39E+02
Lightweight gypsum board partition wall	1.7E+01	1.58-06	7.06E-02	2.89E-02	5.26E-03	1.33E-01	2.49E+02

7.2.2 Integral design of structural elements - seismic sustainability

As already mentioned, a holistic assessment of the environmental performance of any building should take into account the most essential aspects of environmental impacts, with the implication for stakeholders. Such aspects are /7.43/:

- Total energy use, including building operational energy (based on existing legislation) and embodied energy of products and construction processes
- Material use and the embodied environmental impacts
- Durability of construction products
- Design for deconstruction
- Management of construction as well as demolition waste (C&DW)
- Recycled content in construction materials
- Recyclability and reusability of construction materials and products
- Water used by buildings
- The use intensity of (mostly public) buildings, e.g., flexible functionality for different users during different times of the day
- Indoor comfort

However, most designers do not take the evaluation of seismic performance into any more depth than that required by the governing building code. The intent of building codes is to design for a minimum objective of life safety in a design-level event (a major earthquake). The additional cost and resources to make all buildings survive this level of earthquake undamaged would counter the intent of sustainability by utilizing too much energy and resources for a low probability event. For many years now, the minimum objective of life safety has been clear in the codes for the design of the base building structure itself. However, only recently do codes specifically provide seismic design requirements to mitigate failure of glazing systems and other non-structural façade components that can pose a hazard as they fall from the building. Designers currently have little direction on procedures and lack the data necessary to have a better understanding of seismic performance of enclosures. Understanding probable damages and repairs is necessary for performance-based designs, as well as for informed sustainable design. Performance-based design provides choices for the enclosure based on knowledge of the performance for various levels of earthquake ground motion.

The annual probability of a given damage state being equaled or exceeded is found by integrating the probability of the seismic hazard with the fragility curve over all possible values for a site-specific parameter. The area under this curve is the probability of the damage state being equalled or exceeded. The probability of the given damage times the cost of the damage is the seismic risk, and this risk should be included in the LCC of the enclosure along with the risks of other potential damage states. /7.43/

With this approach, one would make design choices that appropriately reduce the potential for seismic loss (risk) and account for seismic loss estimates in LCC studies. The lower the LCC from seismic risk, the greater the seismic sustainability. The losses could be defined in economic terms, environmental impact, or both. /7.43/

New trends propose the adaptation of a "Multi-performance Time-Dependent Based Approach" that is based on three pillars:

- Enhanced safety and reliability: Quantification can be achieved with life cycle performance assessment (ISO 13823:2008)
- Reduced environmental impacts: A full LCA

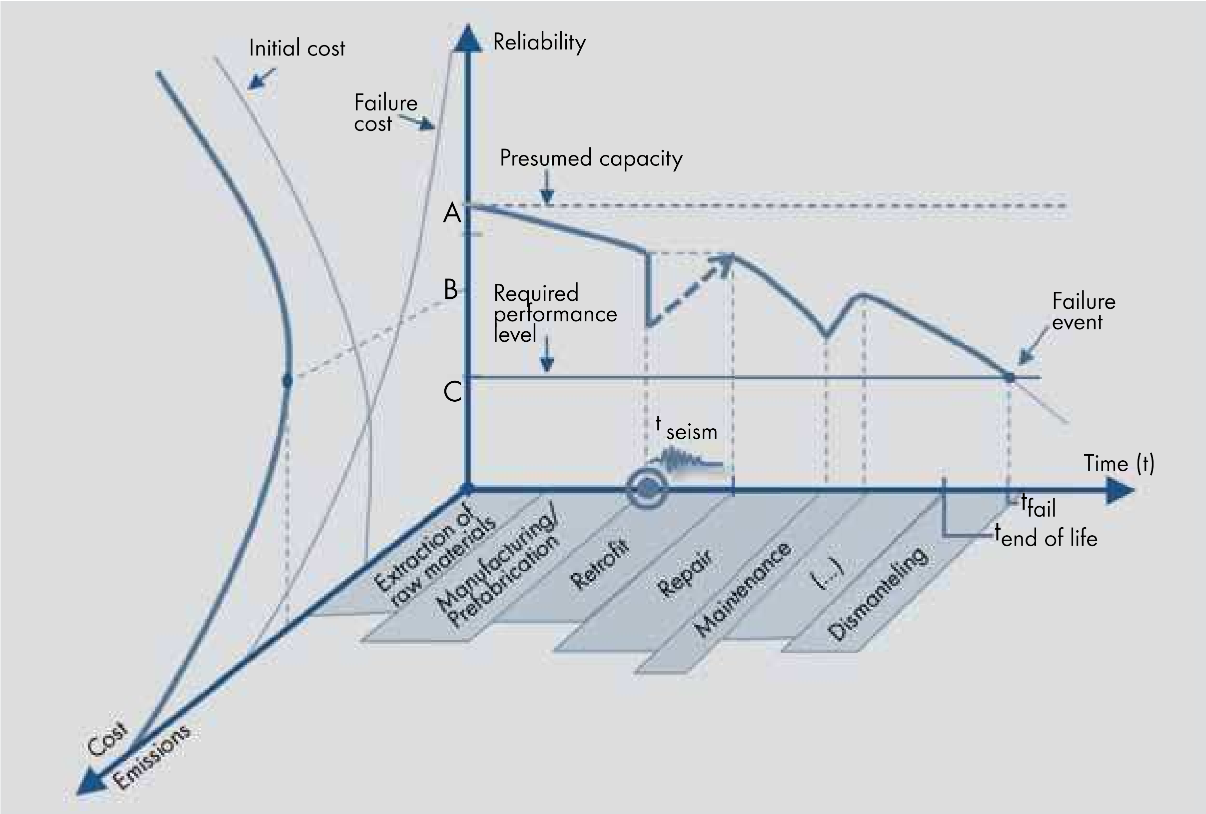


Fig. 7.5: Example of "Multi-performance Time-Dependent Based Approach" /Landolfo, R./

(EN ISO14040-44: 2006), accounting for all life cycle stages of a building (Fig. 7.1) can provide the required quantitative data for assessing the environmental impact

- Optimized life cycle costs: Life cycle costing (ISO 15686-5:2008) can provide analysis over the whole life of the project, showing the real value of the investment

It should be stressed that the above basic requirements should be achieved during the whole life cycle of the construction. The performance requirements should be verified based on quantitative methodologies.

Fig. 7.5 gives an overview of the "Multi-performance Time-Dependent Based Approach". The analysis takes into account the reliability, cost and environmental impact for all the life cycle stages of the building. The consequences of a failure event (seismic event) are included in the calculation of the failure cost and the required performance level to ensure the building's reliability over time.

Overall, lightweight drywall construction offers a very attractive integrated solution in terms of eco-efficiency, structural performance and economic aspects for both structural and non-structural applications, due to the very good environmental performance already presented, the durability and seismic resistance and moderate life cycle costs.

7.2.3 Example: A prototype anti-seismic lightweight steel-gypsum drywall house

The anti-seismic demo building has been constructed by Knauf Gypsopiia ABEE, Greece. It is a two-storey house in Amphilochia, mid-west Greece (Fig. 7.6). The building has a typical residential arrangement plan (ground floor: kitchen, office, a utility room and living room; first floor: master and auxiliary bedroom separated by bathroom). The prototype building has been developed in the frame of three European Union funded projects: I-SSB /7.45/ (The project proved the seismic and fire performance of non-load bearing hybrid steel/drywall structures. Seismic tests up to 1.06 g have been performed with a mock-up building without any damage), MESSIB /7.46/ (the demo house has been equipped with its energy systems

Fig. 7.6: Anti-seismic demo house in Amphilochia, Greece

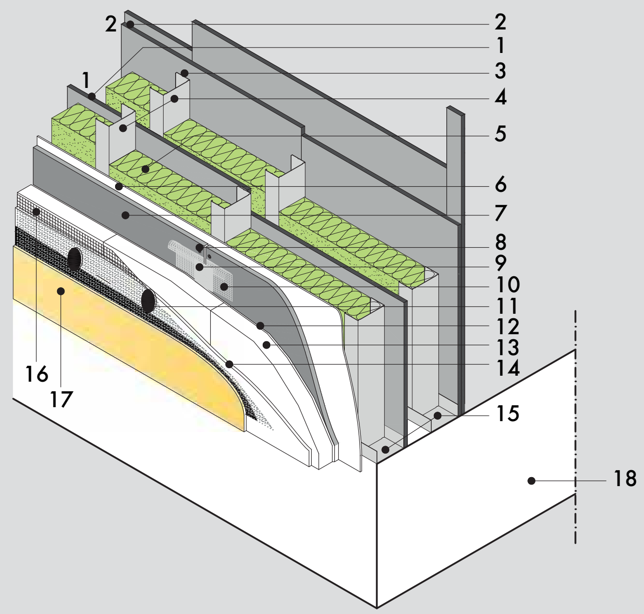
and with Phase Change Material (PCM) integrated into internal walls and partitions. The project proved that implementation of PCMs can significantly improve thermal mass and related performance of lightweight buildings) and FC-DISTRICT /7.47/ (Development of the Building Management System).

The demo house measures 12.33 m width, 9.90 m length and 8.47 m height. The total area, including heated and non-heated spaces is approximately 151.23 m^2, while the heated area is 128.42 m^2. The house has a load-bearing hybrid steel skeleton with CFS members combined with gypsum drywall systems (Internal Walls: W112, Aquapanel; External Walls: Outdoor Aquapanel, Knauf Betocoat, Thermoprosopsis® EPS80/SM700), in accordance with earthquake (EN 1998), fire resistance, thermal and sound insulation requirements. The foundation of the demo house has been constructed as a uniform slab of reinforced concrete. The steel members of the metal frame of the demo house are connected with high tension bolts, and welding is not used. UA Profiles are used for the door and windows openings.

The external walls (Fig. 7.7) of the demo house are multi-layered, consisting of (from exterior towards interior): external wall cladding Thermoprosopsis® comprising a layer of 50 mm EPS 80 insulation, one 12.5 mm cement board (Knauf Aquapanel), a layer of 80 mm Rockwool insulation, one 12.5 mm Knauf Type A gypsum board, a 300 mm cavity (allowing space for the steel frame and plumbing), a layer of 80 mm Rockwool insulation and the final layer of two 15 mm PCM gypsum board joined together.

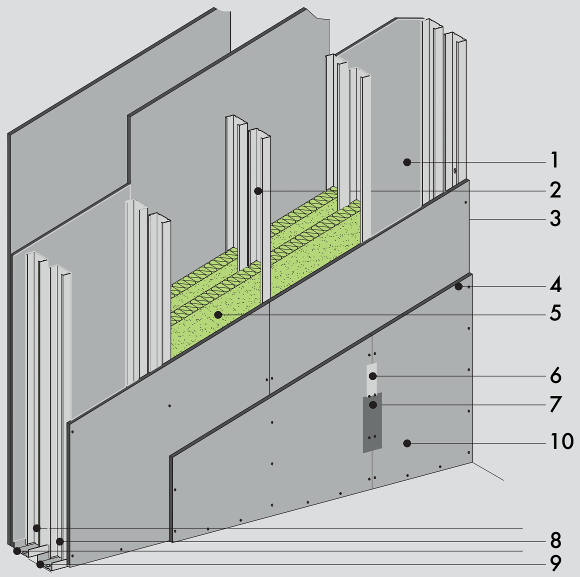
The internal wall consists of two 15 mm PCM boards, a layer of 80 mm Rockwool insulation and two further 15 mm PCM boards (Fig.7.8). On the first floor, there are two layers of Knauf Vidifloor, total thickness 20 mm, followed by the layer of 25 mm polystyrene insulation, needed for the underfloor heating, a 28 mm gypsum fibre panel of Knauf Integral, 250 mm cavity, a layer of 100 mm Rockwool and a layer of 10 mm Knauf Thermoboard.

The demo house's existing HVAC equipment comprises of (Fig. 7.9): Solar panels (for heating, DHW), heat pump (heating, cooling, DHW), buffer tank (hot water storage for heating and DHW). The thermal behaviour



1. Knauf gypsum board Type A 12.5 mm
2. Knauf Smartboard 2 x15 mm
3. Self-tapping screw TN 25 mm
4. CW-100x50x0.6 mm stud profile
5. Knauf insulation 80 mm 50 kg/m^3
6. Tyvek
7. Aquapanel outdoor 1.20x2.80x12.5 mm
8. Cement board screw 4.2x35 mm
9. Knauf Betocoat 15 kg
10. Alcaly resistant joint tape
11. Ntiku Thermoprosopsis®
12. SM700 Natur weiss
13. EPS 80 Thermoprosopsis®
14. PASTOL 20 kg
15. UW-100x40x0.6 mm Runner profile
16. Alcaly resistant reinforcing mesh
17. Conni S 3.0 mm 25 kg
18. Foundation slab

Fig. 7.7: External wall



1. Knauf Smartboard 2 x15 mm
2. CW-100x50x0.6 mm stud profile
3. Self-tapping screw TN 25 mm
4. Self-tapping screw TN 35 mm
5. Knauf insulation 80 mm 50 kg/m^3
6. Uniflott 25 kg bag
7. Paper strip 75 m roll Knauf Kurt
8. UW-100x40x0.6 mm Runner profile
9. Self strip, self adhesive 50 mm
10. Knauf Smartboard 2 x15 mm

Fig. 7.8: Internal wall

of the building has been continuously monitored (http://demohouse.hmcs.mech.ntua.gr/demohouse_site/?lang=en_us) and evaluated since 2011.

The house is equipped with sensors/actuators and a wireless electronic network for early recognition and active monitoring/control of building components. A plethora of measuring devices are installed for the measurement of:

- water temperature; the temperature of the water is measured at all the points of the piping joints, into the water tank (upper, lower level).
- water pressure into the piping; it is critical for the operation of the heat pump
- heating and cooling energy (calorimeters) supplied to the floor and ceiling loops, also at the solar panels circuit for solar energy supplied to the tank
- electric supply and consumption parameters
- water consumption for the DHW

In brief, the experience with regard to the Amphilochia Demo house is:

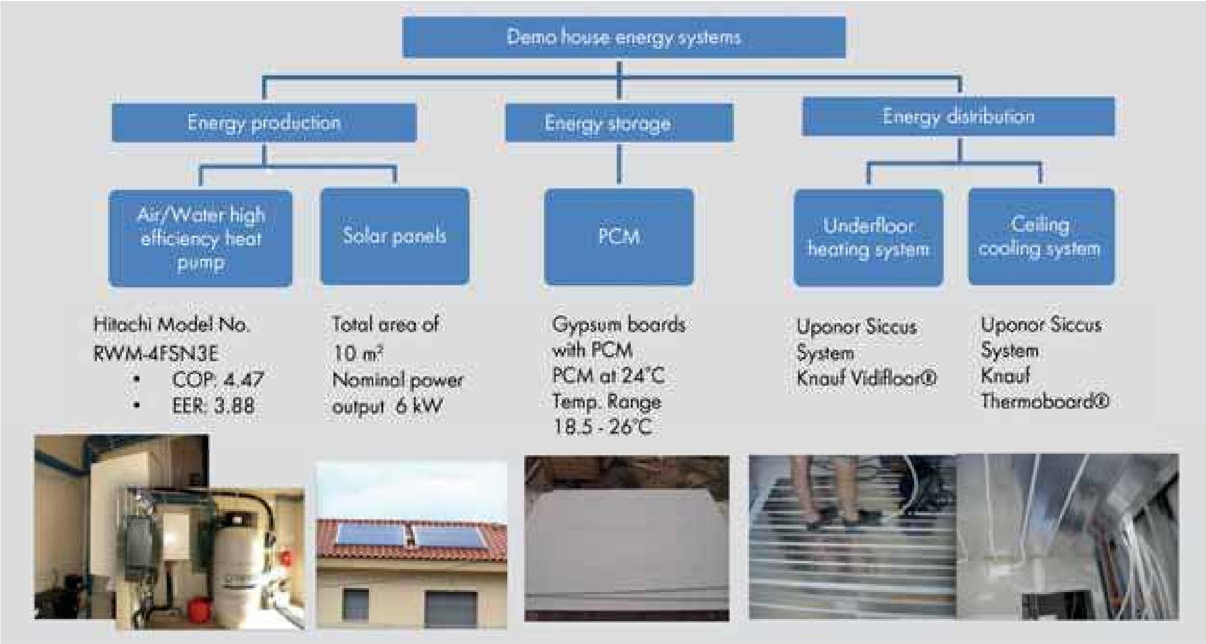


Fig. 7.9: The demo house energy production, storage and distribution systems

- Monitoring (since 2011): Heat flux, temperature and humidity sensors in rooms and inside wall layers, compact weather station (nearby)
- Measurements: Data acquisition: measurements of temperature, heat fluxes and humidity inside the building since 2011, Uponor DEM Controllers operating since 2012, BMS to manage, monitor & control HVAC wirelessly
- Thermal Simulation (TRNSYS): Heating demand: 43.87 kWh/m²a, cooling demand: 17.88 Wh/m²a (PCM), 25.37 kWh/m²a (without PCM); TRNSYS simulation for HVAC, including heat pump and underfloor / ceiling systems; Greek Energy audit Tool simulation: Classification A
- Fire Investigation: FDS / CFD tool for fire protection engineering, fire spread investigation inside the building
- Environmental assessment: Via I-SSB: DGMR environmental index calculation

Energy assessment of the Amphilochia demonstration building

The Amphilochia demo building was built to very high standards compared to the Greek Regulation for the Energy Efficiency of Buildings /7.48/. While KENAK requires 6 cm of insulation (depending on lamda value of materials), the demo building has 16 cm of mineral wool insulation and 5 cm EPS external insulation, adding up to 21 cm of insulation in the external walls. Roof and windows feature also very low heat losses (Tab. 7.11) compared to the reference typical new house. Thus the yearly heat demand is ~43 – 45 kWh/m²a and a cooling demand of 18 – 20 kWh/m²a. The total primary energy demand of the building is 52 % lower than the reference house for the Greek Regulation for the energy efficiency of buildings.

Environmental assessment of the demo house

The environmental assessment has been performed by the Dutch company DGMR (http://dgmr.nl/) in the framework of the I-SSB project using the Greencalc+ (http://www.greencalc.com/) software. The methodology has been adjusted for Greece. The demo building has been assessed against a reference building of the same dimensions constructed out of conventional materials (massive construction, concrete and bricks) with 75 years life expectancy for both buildings. The detailed assessment took into account hidden environmental costs, namely costs that should be made in order to compensate for environmental damage.

Overall, the material environmental costs of the examined demonstration house were 13 % higher than a traditional

Tab. 7.11: Amphilochia demo house building properties

Building element	Highest allowed U value in the climatic zone of Amphilochia (B) (W/m²K)	Demo building U value (W/m²K)
Roof	0.45	0.30
External Walls	0.50	0.16
Floor adjacent to soil	0.90	0.93
Windows	3.00	1.7 – 1.9

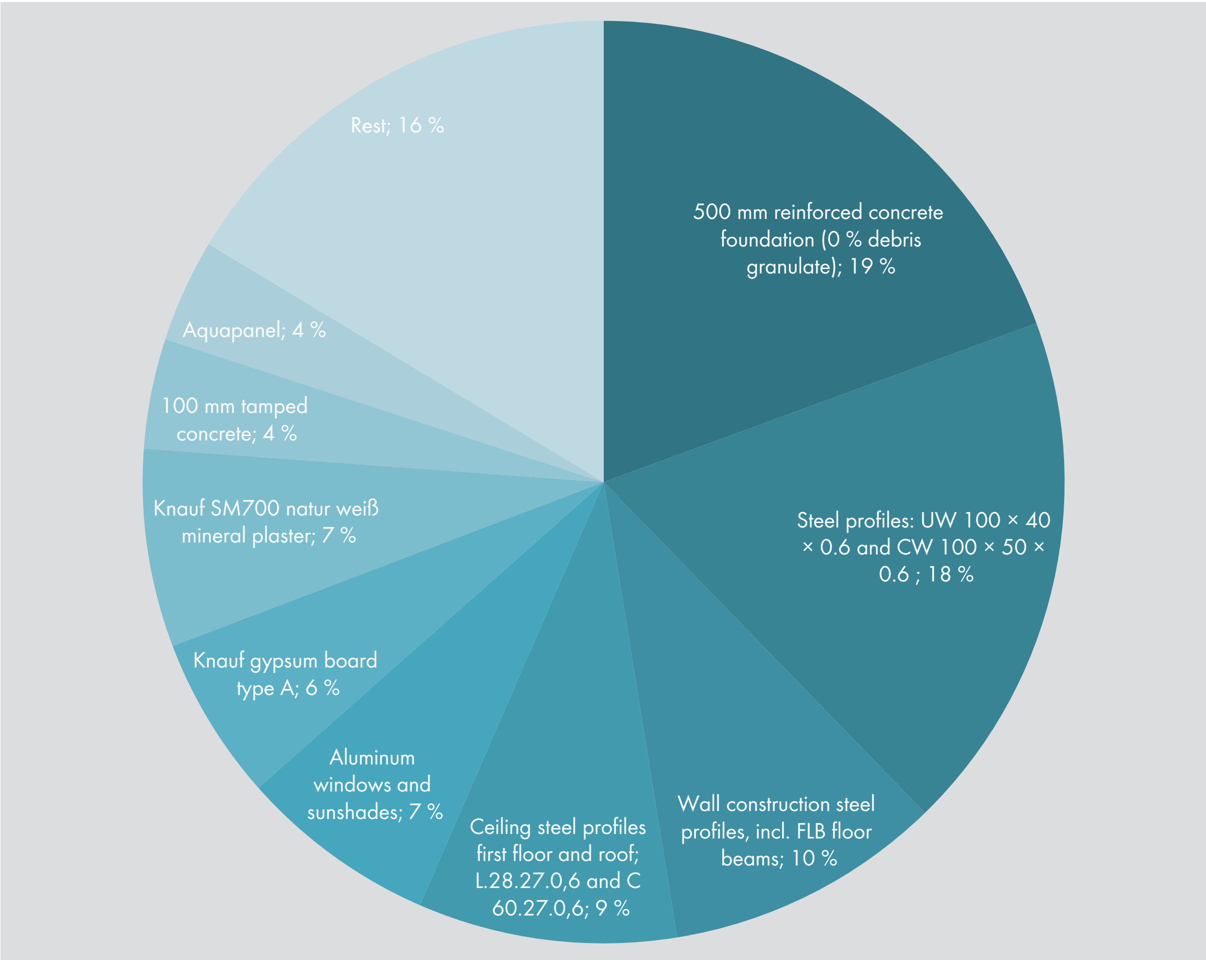


Fig. 7.10: Construction materials with high environmental costs

house due to the large amounts of steel in exterior and interior walls (to ensure seismic safety), aluminium windows and sunshades. It can be seen (Fig. 7.10) that the reinforced concrete foundation accounts for 19 % of the environmental costs, whereas 38 % of the environmental cost of the wall is attributed to steel profiles (frame, wall and ceiling). The reinforced concrete foundation has been considered the same as in the traditional house. The gypsum drywall systems including insulation contribute approx. 17 % to the environmental costs, highlighting the good environmental performance of such systems.

An environmental and energy index has been calculated for four scenarios of energy sources of the building as a percentage of the reference building. The examined scenarios referred to building operation with 1) conventional oil boiler (reference building) 2) natural gas boiler 3) "all electric" with electricity from the grid and 4) combination of renewable energy sources and micro cogeneration systems. Values higher than 100 indicated better performance than the reference building.

The energy index of the demo house (= 404) indicated that the building can have four times better energy performance when renewables and thermal storage are used. With traditional energy concepts (boilers) the energy performance is similar to the traditional house. Namely, the negative impact of the materials used to fulfil the anti-seismic requirements is counterbalanced by the very good energy performance of the building. With innovative energy concepts (such as a micro-cogeneration heat and power system combined with renewables) the demo house becomes “energy neutral”.

The overall environmental index of the demo house with renewables and thermal storage was 159, whereas with the innovative energy concept it reached 205 (further improvement by approx. 29 %). The demo house produces 43 % lower CO_2 emissions than the traditional house, whereas when the innovative energy concept is implemented the overall reduction in CO_2 can be as high as 59 %.

Finally, if the average life of the traditional house has been considered to be 30 years (75 for the demo house), the material index improves from 88 to 154, and the total environmental index from 159 to 210. This is a plausible assumption, since the lightweight, anti-seismic gypsum drywall building can sustain earthquakes and other hazards (e.g. fire).

The study has proven the sustainability of lightweight dry wall construction and has quantified the improvements in environmental performance as a result of anti-seismic design.

8 Case studies

Dennis Holl, Raffaele Landolfo

Following on from the comprehensive explanation of the earthquake-resistant lightweight steel construction, this chapter introduces some representative case studies of projects already implemented or projects being planned and executed in the near future.

Lightweight steel construction is increasingly being used in areas subject to exposure to earthquakes and is used in new buildings, when extending existing buildings as well as for reconstruction in areas that have been subject to destructive earthquakes.

The greatest challenge to be faced is the break from traditional building methods while making way for innovative solutions. The case studies presented are intended as encouragement and should also show that this method of construction is not subject to any limitations in terms of architectural design and occupant comfort. Quite the contrary is true, as the planning creativity is enhanced thanks to the lightweight construction method and flexible application features as well as the benefits of possible pre-fabrication.

Irrespective of the current absence of relevant standards and constraints, the acceptance on the part of building supervisory authorities and planners is increasing. On the one hand, due to the continuously improving technological standards from the industry and the rapid advances in research and development, and on the other hand, due to the time-saving factor offered by the use of pre-fabricated elements, particularly in regard to rapid reconstruction. Existing standardized solutions are employed in part here, which can be quickly put into action when the need arises. In this particular application, time and efficiency are the decisive factors, as it is essential to re-establish the infrastructure and housing space as quickly as possible and to ensure that the financial resources provided by donations and emergency funds are used for their intended purpose.

Depending on the usage, special planning approaches and points of emphasis must be taken into consideration with respect to the earthquake safety. However, in all areas, the primary priority is the protection of life and health.

Residential buildings

Residential buildings tend to place the focus of the demands on protection of property and the cost-effectiveness over a comparatively long service life. In this case, the demands and requirements of the occupants and the investors/owners may differ. The investors/owners are focused on the construction costs as well as the costs for any required repairs. For the occupants, however, the focus is heavily on the need for protection in their own home, in their familiar environment and private property. Clarity with regards to the presence of those affected is useful for the evacuation should it be necessary.

Public buildings

Public buildings place high demands on safety due to the potential for large gatherings of people. Also to be considered is the possible infrastructural significance, necessitating a preservation of the function, particularly in the event of a catastrophe, as is the case for hospitals, administration buildings and security services.

Commercial buildings

Commercial buildings have a comparatively low service life. In addition to the protection of life and health, the focus is on the protection of assets and business activity.

Here it is necessary to differentiate between the modes of operation (e.g. manufacturing, sales, office space, services) and to base the planning on results of a corresponding risk assessment.

Residential building, CasaLow, Crevalcore, Italy

Crevalcore suffered great losses during the magnitude 5.9 Emilia Earthquake in 2012. Designed especially for the fast and realizable recovery of the infrastructure, a project called "CasaLow" was introduced. Node analysis and a special foundation were the basis for a lightweight steel structure according to the Italian building code. The motivation was to combine renewable energy, an innovative envelope as well as comfort and earthquake safety for single family houses at affordable prices.

Drywall systems have been provided by Knauf Italy to deliver a high performance (thermal and acoustic) and provide the mechanical characteristics to resist seismic actions. The selected drywall systems for the ceilings (D112) ensured fast installation during the seismic retrofitting of the interior partitions as a combination of single and double stud partitions with two layers of suitable gypsum boards. The interior loads were reduced and the physical properties enhanced. The façade is a combined construction of cementboard and gypsum board cladding with ETICS on the exterior.

Client:	Private
Builder:	Nuova Rinnova P. Testi
Architectural design:	Studio =2A - Debora Venturi/Luigi Orioli
Type of building:	Detached house/steel construction

Trielxis appartments, Athens, Greece

This project shows attractive appartments in a modern look built in an efficient way. A clean cubature with two C-shapes situated at the encompassing one another in the form of overhanging balconies providing an exterior extension of the living space.

The wall elements may seem rather heavy and solid, however, as the supporting structure of the building consists of a steel construction with concrete slabs on top of the metal sheet and is additionally insulated with the Knauf system Thermoprosopsis Organic, this heavy character is purely visual. The elongated wall elements encompass the overhanging balconies like a frame, and their design provides shade.

The internal room layout was implemented using different types of internal wall partitions mainly with gypsum boards and gypsum fibre boards, whereas for the exterior walls, a mix of gypsum and cement-based board partitions were used.

Accordingly, reducing the dead loads, which are critical in the event of an earthquake, was the overall aim.

Client:	Zacharias Douros
Builder:	Trielxis ATE
Architectural design:	Zacharias Douros
Type of building:	Appartments/steel construction



Residential extension, Pratinou Str., Heraklon/Crete, Greece

In this project, new residential space was created by extending an existing old building by three additional floors. The benefits of drywall constructions can be used to great effect to meet the desired properties and especially with regard to the main challenge with extensions – minimizing the additional loads in order to not overload the existing supporting structure of the building.

In general, the addition of one or more storeys generates more living space without exceeding the building circumference. The restrictions may vary from case to case, however, if the construction plans do not impair the character of the city scape or the individual structural limits of the building concerned, it is an option worth considering. One of the main aims is the creation of lucrative and attractive living spaces in popular areas.

In particular in seismically-active areas, the issue of newly added loads must be closely considered as was the case in this project with drywall ceiling constructions D112 and double layer cladding wall partitions W112.

Client:	Aris Pappas
Builder:	C & M Engineering
Architectural design:	Konstantinos Pappadopoulos
Type of building:	Residential extension/ steel constuction/ reinforced concrete




Detached house, Catanzaro - Via dei Tulipani, Italy

Three-storey residential building with a gross floor area of about 180 m^2. According to the client's needs, the dining room with kitchen and toilet are located on the ground floor. On the first floor, there are living room, study and bathroom, and the sleeping area is on the second floor. The building is equipped with a photovoltaic system that guarantees the total energy needs.

The lateral seismic resisting system was obtained by using a so-called "all-steel" solution. In particular, cold-formed steel stud walls braced with diagonal straps were used to counter the horizontal seismic actions. In this project, the cladding was mainly implemented using gypsum fibre boards and the exterior insulation with stone wool.

Client:	Corapi Giuseppina
Builder:	SUPREMA evoluzioni tecniche
Architectural design:	Arch. Claudio D'Onofrio
Type of building:	Detached house/steel constuction

Construction of a one-family house, Sersale (CZ), Italy

The house is an extension of an existing small rural building. The new construction incorporates the existing building, which is used as a warehouse. The house was built on sloped ground, where a solution with low weight was required for geotechnical reasons.

A two-storey one-family house with the ground floor used as a living area, a utility area, for storage and a garage, and the first floor was used as a sleeping area. The building is equipped with a water heating system with an electrical heat pump for heating, and cooling using cooling convectors to cool every room. The equipment has integrated solar panels to supply hot water.

The lateral seismic resisting system was established by using an all-steel solution. In particular, cold-formed steel stud walls braced with diagonal straps were used to counter the horizontal seismic actions. A steel construction is also the basis for the floor. The façade is a ventilated façade with U = 0.17 W/(m²K) insulated with stone wool.

Client:	Luca Torchia , Simona Pitari
Builder:	CONDINO Engineering
Architectural design:	Ing. Michele CONDINO
Type of building:	Detached house/steel constuction

Deutsche Schule Athens, Athens, Greece

In institutions such as schools, evacuations in case of emergencies are conducted on a regular basis to ensure a safe and quick exit from the building. Escalators may not be used, emergency exits and defined escape routes should be known. Nevertheless, it is always a challenge to evacuate a large number of people from the building simultaneously. Fire protection concepts sometimes suggest that rather than totally evacuating the building, people should be removed from the endangered unit to a safe unit. As the consequences of an earthquake may bring unforeseen damage to the building, this is not the preferred solution.

The single floor steel structure of the German School, which was finished in 2013, is a double-layer exterior wall with insulation and cement board cladding. Today's main tasks involved with schools are good acoustic planning, the indoor climate for a good learning environment and also higher levels of indoor safety and security due to the recent events of individuals running amok. Drywall systems with gypsum hard boards can be chosen to suit to these higher exposures.

Client:	Deutsche Schule Athens
Builder:	Aris Pappas
Architectural design:	
Type of building:	School/steel constuction

BFS School Naples, Naples, Italy

The building is a drywall construction, implemented using CFS members, adopted to realize both standard steel buildings and stick-built constructions based on the result of scientific studies developed at the University of Naples Federico II.

The building received the ACAI 2011 award for its constructive system made of CFS profiles, which, when compared to traditional steel systems, provides a number of indisputable advantages such as: Lightweight, low cost, simplicity of construction and sustainability.

More detailed information is provided in chapter 5.5.

Client:	Defense estates operation international - HQ European division
Builder:	COSAP (Consortium stable public works) – Giugliano in Campania, province of Naples
Architectural design:	
Type of building:	School/steel constuction

Hellenic Motor Museum - Capitol shopping mall, Athens, Greece

A look inside the Hellenic Motor Museum shows an exhibition of splendid oldtimer cars. Museums in general host valuable exhibits, which would be irretrievably lost, should a major earthquake occur. In general, planners nowadays focus their attention more on the building services, as valuable exhibits often require constant room climates which can be achieved by intelligent cooling and heating systems operaing autonomously in parallel to an air conditioning system. The integration of suitable lighting introduces further loads into the building which might be critical in cases of fire or earthquakes. All these technical devices, however, can be perfectly integrated into drywall systems as in the ceiling D112 with a subceiling for an improved acoustic performance (D127) to reduce the echo effects in the wide halls of the museum. Another requirement in museums might be the demand of higher cantilever loads, so that exbihits can be directly attached to the partition walls. High density gypsum fibre boards are integrated as a support for cantilever loads.

Client:	Charagionis Group
Builder:	Charagionis Group
Architectural design:	Charagionis Group
Type of building:	Museum/steel constuction

Knauf Iran's corporate headquarter building, Tehran, Iran

The system for lateral load bearing of structures is a flexural frame. In an excavated area deep under ground, reinforced concrete shear walls implemented as retaining structures form a core element for earthquake resistance. A range of drywall systems were chosen for the multi-storey building completed in March 2015. The acoustic performance in the offices was optimised using suspended gypsum board ceilings with metal subconstruction (D112), dropped ceilings with mineral boards (AMF System C) or a free-spanning ceiling (D131). Free-spanning ceilings are an option, when there is a high density of installations in the ceiling plenum, and there could be difficulties with regular hanger intervals and fixing points to the basic ceiling. Generally, free-spanning ceilings also acoustically decouple the ceiling from the basic ceiling and may be used very efficiently for this purpose.

To reduce the effect of the façade on the supporting structure, a curtain wall in front of a double stud partition with two layers of AQUAPANEL Cement board was chosen.

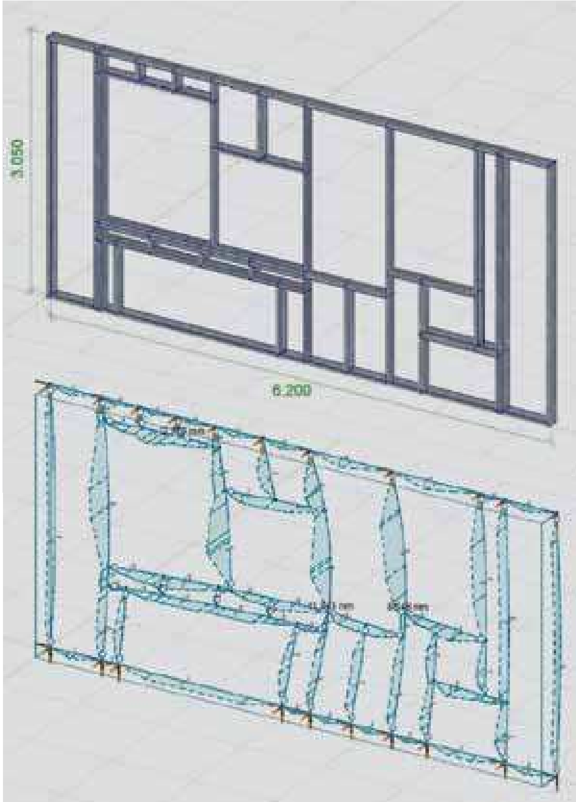
Client:	Knauf Iran company
Builder:	Dar afzin consulting Engineers Company
Architectural design:	Dar afzin consulting Engineers Company
Type of building:	Office building/steel construction

Südpark Basel,
Basel, Switzerland

An extraordinary project "Südpark" was implemented when restructuring the city of Basel. It is a ten-storey building (70 x 80 m) housing appartments, offices and shops as well as a retirement home. A major challenge was to combine the various needs of the different groups of users.

Südpark is, thanks to its unusual facade design, a real eye-catcher in the Gundeldinger quarter located behind the railway station. The seemingly randomly arranged three-dimensional shifted geometries of window combinations are based on a sophisticated programming of the façade, which was developed by Herzog & de Meuron in collaboration with the CAAD professor at the ETH in Zurich. In contrast to the front, the rear facing façade is horizontally structured and almost fully glazed. A real challenge, as besides providing for the structural stability and building physics dimensions of the façade construction, the building was constructed earthquake-safe because of a risk potential of level 3.

Client:	Südpark Basel
Builder:	
Architectural design:	Architects Herzog and de Meuron
Type of building:	Multi-purpose building/ Steel lightweight construction



The ERNE AG Holzbau specifically developed a solution based on a non-combustible lightweight steel construction with "Cocoon-Transformer"-Profiles for the project Südpark Basel. All steps from planning, production and realization came from a single source. For best planning results, the complete façade was set up as a 3D-Model based on the existing 2D ground model, a catalogue of 308 elements served as the planning basis. Profound calculations such as windload simulation were carried out to design the building shell constisting of a modular construction with subconstruction. A possible deformation of the basic construction should have a minimized influence on the façade system. This is solved intelligently by leading the arising forces into the supporting structure. A dual system of point loads is transferred via enforced beams and cantilevers via the frontal side of the prestressed ceiling into the building structure, and the main beam supports the overall structure to compensate for the deformation.

The building time was an additional highlight of this project. The façade construction company ERNE applied their own homogeneous production process, which facilitated the highest possible degree of prefabrication. Accordingly, the facilities for the preproduction were specially adapted to this project and allowed a smooth on schedule fabrication of 2 elements per day at the outset, eventually leading to an output of up 4 or 5 elements per day at the later stages. The entire process optimization also integrated the suppliers into the overall planning process up until the production stage.

The system-based construction facilitated an optimized, efficient and economic production process with consistent high precision and quality with regard to controllable environmental conditions in the delivery to the construction site as ready-to-plug-in-elements just in time.

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