Normenvergleich zwischen Österreich und Amerika im Bezug auf Erdbebensicherheit an einem konkreten Beispiel eines Mauerwerksgebäudes

Comparison of the technical standards between Austria and America of earthquake safety on a concrete example of a masonry building

Diplomarbeit

Master – thesis

Zur Erlangung des akademischen Grades

Diplom-Ingenieur/Diplom-Ingenieurin für technisch-wissenschaftliche Berufe

der FH Campus Wien

Masterstudiengang Nachhaltigkeit in der Bautechnik

Vorgelegt von:
Christine Ottendorfer

Personenkennzeichen
1110325010

Erstbegutachter/in:
Dipl.-Ing. Dr.tech. Franz Zach

Zweitbegutachter/in:
DI Dr.techn. Anton Pech

Abgabetermin
04.07.2013

Erklärung:

Ich erkläre, dass die vorliegende Diplomarbeit von mir selbst verfasst wurde und ich keine anderen als die angeführten Behelfe verwendet bzw. mich auch sonst keiner unerlaubter Hilfe bedient habe.

Ich versichere, dass ich dieses Diplomarbeitsthema bisher weder im In- noch im Ausland (einer Beurteilerin/einem Beurteiler zur Begutachtung) in irgendeiner Form als Prüfungsarbeit vorgelegt habe.

Weiters versichere ich, dass die von mir eingereichten Exemplare (ausgedruckt und elektronisch) identisch sind.

Datum: .... Unterschrift: ....

Kurzfassung

Diese Arbeit beschäftigt sich mit einem Normenvergleich zwischen Österreich und Amerika. Da Europa und Amerika unterschiedliche Kulturen repräsentieren, liegt die Vermutung nahe, dass es auch Unterschiede in der Handhabung technischer Richtlinien gibt. Im Detail soll die Regelung und Vorgehensweise zufolge Erdbeben und den damit erforderlichen Nachweisen und Anforderungen erläutert und verglichen werden. Da dieses Thema ein sehr umfangreiches Gebiet umfasst, soll diese Arbeit speziell auf das Gebiet der Mauerwerksbauten beziehen. Das Mauerwerk wurde aus dem Grund gewählt, da Österreich sehr von der Ziegelbauweise geprägt ist. Seit über hundert Jahren werden Häuser aus Mauerwerk hergestellt. So wurden auch die Gründerzeithäuser in Wien in dieser Bauweise errichtet. Gründerzeithäuser sind ein wichtiger Teil des heutigen Stadtkerns. Da die alte Herstellungsweise jedoch nicht mehr konform mit den heutigen Standards ist, liegen hier deutliche Schwächen zufolge der heutigen Erdbebennachweise vor. Aus diesem Grund soll mit den amerikanischen Normen eine Gegenüberstellung erfolgen, um zu ermitteln, wie die Vorgehensweise in einem stärker beanspruchten Land erfolgt.

Zu diesem Normenvergleich werden für Österreich die ÖNORMEN EN und B und für Amerika wird die Norm ASCE herangezogen. Da ASCE lediglich für das Erdbeben dient werden weitere Normen für die Anforderungen des Mauerwerks erläutert.

Mit einem näheren Einblick in die Werte und Vorgaben wird es ermöglicht, diese zu einen theoretischen gewählten Beispiel normgerecht anzuwenden. Mit diesem praktischen Teil werden beide Kalkulationen zufolge der Normen zu einen konkreten Beispiel durchgeführt und die Ergebnisse miteinander verglichen. Mit der Gegenüberstellung der Ergebnisse sollen die Unterschiede erkenntlich gemacht werden und Basis für eine nähere Erläuterung geben.

Abstract

This thesis deals with a comparison of the technical Standards between Austria and America. As Europe and America are representing different cultures it seems likely that there are also differences in the use of technical guidelines. In detail, the arrangements and course of action according to earthquake and the necessary qualifications and requirements should be discussed and compared. For the reason that this topic covers a width area, this work is specifically related to the field of masonry structures. Masonry is chosen for the reason that Austria is dominated by brick construction. For over a century is masonry used for buildings. So also the Wilhelminian buildings in Vienna were built in this way. Wilhelminian buildings are an important part of today's city center. But the old method of construction is not longer conforming to today's standard and show significant weaknesses for the proof against earthquake that is now required. For this reason, a comparison with the American standards should occur to determine how the course of action is defined in a country which is more at risk.

For the comparison of the technical standards is used for Austria the ÖNORM EN and B, and for America the ASCE standard. Because ASCE is only including requirements for earthquake, are other codes used to clarify the requirements of the masonry.

With a closer look at the parameters and input requirements it is possible to apply the standards to a chosen theoretical example. With this practical part, both standards are applied for the calculation according to this specific example. The results will be compared with each other. With the confrontation of the results, the differences should be demonstrated and form the base for a detailed explanation.

List of Abbreviations

CEN	The European Committee for Standardization
ÖNORM EN	österreichische Norm, Kategorie Europäische Norm
ÖNORM B	österreichische Norm, Kategorie Bauwesen
RC	reliability classes
CC	consequences classes
DSL	design supervision level
IL	inspection level
EERI	Earthquake Engineering Research Institute
ASTM	American Society for Testing and Materials
ANSI	American National Standard Institute
ASCE	American Society of Civil Engineers
TMS	The Masonry Society
ACI	American Concrete Institute
NIST	National Institute of Standards and Technology
SDOF	single degree of freedom oscillators
MCE	Maximum Considered Earthquake
USGS	United States Geological Survey

Keywords

Building Code

Austria

America

Design

Earthquake

Seismic evaluation

ASCE

Unreinforced masonry

Cross wall

Shear strength

Methods of Analysis

Equivalent lateral force

Performance

Index

1. Introduction

For the most part, Austria is not really at risk of earthquakes. There are about 600 earthquakes a year, but average, only 40 are noticeable to people. Nevertheless, earthquakes consistently cause structural damage to buildings, because some Austria's buildings are not resistant enough. [1]

This depends on the out-of-date standard which was used during this time. Through the continual changing of technical standards and the defined minimum requirements for a building, these buildings do not catch the achievements of today. The current research and development of new technologies and possible solutions has led to a better understanding of construction materials and its reaction in special situations. Today, there is far greater knowledge than hundred years ago which allows new constructions to be built much better than before.

But every building tells a historical story and reflects the development of Austrian culture. Therefore every architectural change must be evaluated. It is necessary to balance the costs of repairing structural damage against the cost of constructing a building which would be totally resistant to earthquake damage.

For a better understanding of engineering requirements in respect to earthquakes, it is necessary to know the requirements of the technical standard, and in particular the implications of the choice of building material. For example, Austria has a lot of old, brick-built masonry buildings.

1.1. Question

Austria is generally safe from catastrophic earthquake destruction, and thus most buildings can meet technical standards. While modern earthquake standards could

[1] Zentralanstalt für Meteorologie und Geodynamik: Übersicht. In: http://www.zamg.ac.at/cms/de/geophysik/erdbeben/erdbeben-in-oesterreich/uebersicht_neu (last access: 3.7.2013)

theoretically be met, it is important to recognize that older buildings were not constructed in the same manner as modern buildings and thus there are many technical differences between new and existing buildings.

In contrast to Austria, America has several regions that are more seismically active, which has necessitated the development of technologies and materials to minimize damage or potential damage to life and limb.

As America is generally more at risk to earthquakes, the question naturally arose whether or not the standards in America are different to those in Austria. Therefore it is meaningful to take a closer look at both standards to discover the main differences between Austrian and American earthquake engineering standards. Do both standards reflect the current scientific knowledge?

Is it possible for one standard to improve the other? Do both standards produce the same results with regard to earthquake safety?

1.2. Target

In general, the comparison of the Austrian and American standards should show where the main differences lie. The main problem that arises is if it is even possible to compare two unequal countries with completely different input facts.

By ascertaining the main points of both the Austrian and American standards, it should be possible to recognize where the standards differ and where they are similar, thereby establishing common points of reference, which can then be used to directly compare the standards.

Because of the fact that many buildings in Austria are made of bricks, the main focus of comparison should be masonry structures, in particular the Wilhelminian buildings. Due to their cultural significance, it is important to know how they would react in the case of an earthquake.

2. Wilhelminian Buildings [2]

Many buildings in Vienna are Wilhelminian buildings. They are part of Austria's history and culture. Vienna has a famous historical center, and one of the main focuses of tourists is to see the old buildings of Austria.

Fig. 1: Front of a wilhelminian building

In general, Wilhelminian buildings were built of brick around 1900. The prevalence of brick construction highlights the importance for the inclusion in Austrian standards.

Living in a Wilhelminian building is very popular because they were built with features that are uncommon in modern buildings: high ceilings, beautiful extensive fronts and a lot of charm. It is important to conserve these houses for the future.

The probably most fundamental shortcoming related to this kind of building is earthquake resistance.

[2] Hollinsky, Karlheinz: Kapitel 01 – Bauweisen von Gründerzeithäusern, FH Campus Wien, Skriptum. WS 2011/2012

2.1. Structure of wilhelminian building

In order to recognize how the building will react in an earthquake, it is first necessary to know about the structure and how the pieces work together. Further should be described which problems are appear with the common structure.

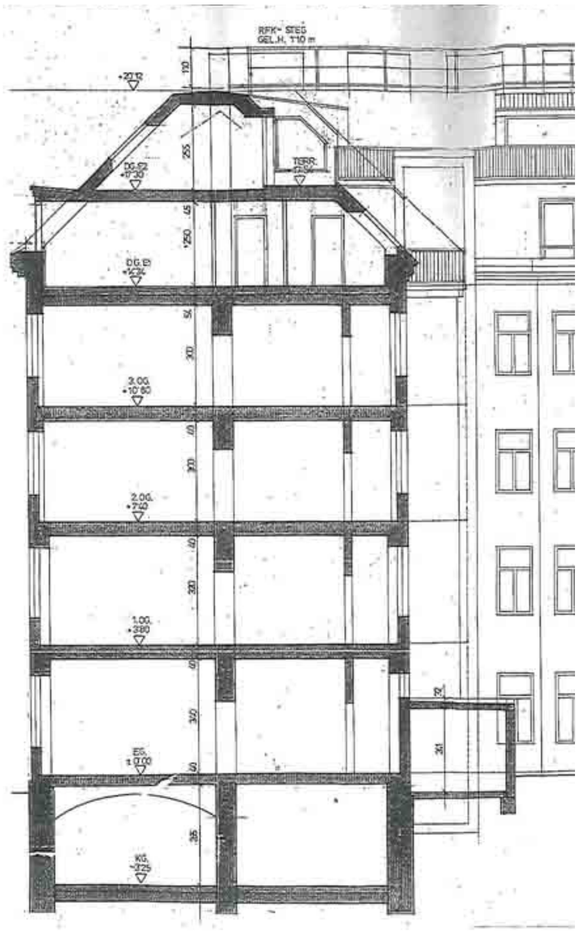


Fig. 2: Profile of a wilhelminian building

2.1.1. Foundation

The foundation of a Wilhelminian building works like a strip footing. It depends on the layers of material in the soil and its carrying capacity.

Sometimes pile foundations are also present, but in most cases the foundation is limited to the walls which extend into the soil at the deepest level. Often problems arise because there is a minimum tie-in of 0,8 – 1,0 meter which means a danger of base failure when the tie-in is less.

2.1.2. Side walls

These walls, of which the foundations are essentially built, are typically found throughout the whole building. The main walls consist of the middle wall and the two walls at the exterior of the building. They are the major components for bleeding out the vertical forces.

The middle wall is the most critical part because, depending on the influence area, it must carry approximately half of the overall load, despite having many weak areas, such as the breakdowns for the doors or areas in which the inspection chambers of the chimney are situated.

Typically the size of these walls varies. The thickest part of the wall is at the bottom and will reduce as the height increases. They can start with a width of 90 cm or more, than they typically change their width with each floor level, reducing by a brick's width (roughly about 15 cm). This style is based on providing more load pressure at the bottom but also it is used as a bearing for the ceiling.

2.1.3. Cross walls

The cross walls run counter to the axis of the street and are very important for stability, especially to bleed off the horizontal forces in case of earthquake. They go through the whole building but normally do not change their width in the upper floors.

The cross walls consist of the two end walls against the neighbors' estate and the smaller walls inside the building. Normally, the rooms inside a Wilhelminian building are smaller than today, so usually after every second window row is a cross wall.

The original buildings have good cross-bracing as a result of the tight arrangement, but mostly a building undergoes changes during its lifetime.

2.1.4. Ceilings

There are many different construction forms for ceilings, but the most commonly used material is wood. Traditionally, there are preferential construction types for certain ceilings.

The main floor is mostly built as an arch, with steel beam and bricks for the bow. For the upper floors, a construction called Tramdecke is commonly used which means wooden beams are the building's carrying structure. For the uppermost ceiling, the use of a Dippelbaumdecke has been approved, which is a ceiling where beams of wood are built in a continuous wooden layer which brings a much better result in case of fire.

Generally, such wooden ceilings are still in a good condition, but there are very susceptible to water damage. Another problem results from the fact that many of these constructions are not stiffened for thrust which leads to a different reaction in earthquake situations.

2.1.5. Cornice

An important part of the building is the cornice. While the cornice does not have a carrying function, many buildings have a cornice as an architectural element of the facade. Typically, cornices do not carry themselves because the balance point does not lie in the center of the under wall. It is therefore necessary to secure them to prevent falling to the street below and injuring a pedestrian. The risk will increase in situation where an additional force will be present, for example, in the case of an earthquake.

2.2. Foreseeable Problems

The main problem with these buildings is that they are not 100 percent resistant against earthquakes.

These buildings are old and were built in a different time. 100 years ago, building professionals did not have today's knowledge or technology. In general, these houses could be called non-standard from the current technology which makes it difficult to adopt the technical standards of the current time.

For improved earthquake resistance, changes have to be made in the foundation, in stiffening the ceilings for thrust, etc.

Another point is that these buildings have often been changed from their original function. For example, many buildings now host large retail stores in the ground floor. With this change from the original use, cross walls are often removed to create an open plan sales area. There are also changes to the upper floors, for example in apartments or offices. The living space of today is much bigger than when the buildings were designed. However, every wall that is removed results in a reduction in earthquake safety because only walls that extend through the whole building can bleed off the horizontal force of an earthquake.

The original construction materials used in Wilhelminian buildings lack the structural integrity of modern construction products. Time as well leaves its mark on the structure. So in any given building, there could be found different types of damage which leads to structural weakness for example damage from the WWII, water damage on ceilings, etc.

It is important to understand that a change in technical standards may not increase the earthquake safety, but it should help pinpoint areas which, if improved, would lead to improved earthquake resistance.

3. Guidelines for building and material

With this chapter, the technical standard for Austria should be described to understand how the course of action is defined in the country. With the comment of the requirements and properties to a building and its material should be declared the knowledge of Europe and in detail Austria.

The European Committee for Standardization (CEN) has developed with all member countries guidelines for common structural building and civil engineering structures, also known as the Eurocodes. These documents are transformed into European Standards and serve as a common set of guidelines and requirements. In deference to the fact that different countries have different architectural and cultural influences, the documents allow in some parts a scope for own definitions in the particular country. If a country decides to use these options for specialization, they can be found in a national annex which applies only for the single country. Austria has the ÖNORM EN as general standard and ÖNORM B as national standard.

For Wilhelminian buildings, two standards are particularly important. The first one is EN 1990 which shows general requirements for building structures and the second one is EN 1996 which caters especially to masonry buildings.

3.1. Guidelines trough EN 1990 – Basis of structural design

3.1.1. Basic requirements [3]

EN 1990 states that buildings have to be constructed in such a way that they can be used through the whole defined useful life, whilst taking into account reliability and cost effectiveness. It is therefore necessary that the structure bear up all influences and actions without losing qualities for its intended use. These influences could be for example permanent, variable or accidental actions like

[3] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter 2.1

explosion, earthquake, wind and snow loads among others. Which influences are essential is individual to the building.

In general, the standard differentiates between three main parts in evaluating a given structure:

- structural resistance
- serviceability and
- durability

To meet the requirements it is important to minimize risk of damage through the choice of suitable material, planning or controlling.

The reliability can vary depending on how important the building is. Factors are economic value, risk to persons inside the building or how expensive it is to reduce the risk. [4]

Therefore the standard defines three reliability classes RC1, RC2 and RC3. They are associated to the consequences classes (CC1, CC2 and CC3), to the design supervision levels (DSL1, DSL2 and DSL3) and the inspection levels (IL1, IL2 and IL3). Wilhelminian buildings will fall into the consequences class CC2 – residential buildings with medium consequences. [5]

3.1.2. Durability [6]

For durability, the structure should be planned and built in a way, that time-dependent changes of performance during its expected useful life can change the characteristics of the structure in an unforeseeable way.

[4] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter 2.2

[5] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter B.3

[6] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter 2.4

Therefore it is important to identify the main facts during the phase of planning to enable the necessary action for durability.

For a durable structure, the following aspects should be taken into account:

- use of the structure
- environmental conditions
- building material
- building ground
- choice of the structural system
- design of parts
- maintenance during length of life

As an example, for masonry buildings, a micro-environment condition must be taken into account if the completed masonry will be exposed to chemicals, salt or moisture. Therefore five classes ranging from MX1 to MX5 define how intense the attack is which the masonry has to resist. [7]

3.1.3. General design situations

There are different situations of action against which a building must resist. In general there are four different classifications which vary in time of their appearance. Persistent design situations with permanent actions, transient design situations with variable actions, accidental design situations like fire or explosion and seismic design situations like earthquakes with accidental actions. [8]

[7] Österreichisches Normungsinstitut: ÖNORM EN 1996-2. Bemessung und Konstruktion von Mauerwerksbauten. 15.11.2009. Chapter 2.1.2.1

[8] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter 3.2

Assessment requirements must be met both in structural resistance and serviceability.

In calculating the theoretical integrity of the design, it should be considered that a combination of different design situations can appear at the same time.

The calculation has to include all relevant design values for actions, material and product properties and geometrical data. Also partial factors are defined which have to be included into the calculated value; for example the partial factor for materials γ_M. [9]

For properties of building materials or products should be given characteristic values with express a low value with a 5 % fractile and a high value with a 95% fractile. [10]

3.1.4. structural resistance

The ultimate limit states of structural resistance are defined with the safety of people and structure. [11]

[9] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter 6.4.3

[10] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter 4.2

[11] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter 3.3

There are different verifications for limit states, as follows: [12]

EQU: for losing of static equilibrium of the structure

STR: for internal failure or excessive deformation of the structure

GEO: for failure or excessive deformation of the ground

FAT: for fatigue failure of the structure

For the proof of the EQU it has to verify that the design value of stabilizing actions are greater than or equal to the design value of destabilizing actions.

For the proof of STR and GEO it has to verify that the design value of the resistance is greater than or equal to the design value of the effects.

3.1.5. Serviceability

The limit states for serviceability are focus on effects that could change the functioning of the carrying structure, or the comfort of the users, or the appearance of the edifice. This includes the prevention of damages to the structure or building material such as cracks in walls, or vibrations, deformation or displacement which leads to a decline in serviceability. [13]

Therefore these criteria should be determined as soon as possible as they will affect the choice of materials and construction method. Every building has different criteria which are specific for the chosen usage. Such criteria can be for example the stiffness of the floor or roof or displacement of stories. The standard defines such criteria through figures such as Fig. 3 and Fig. 4.

---

[12] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter 6.4

[13] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter 3.4

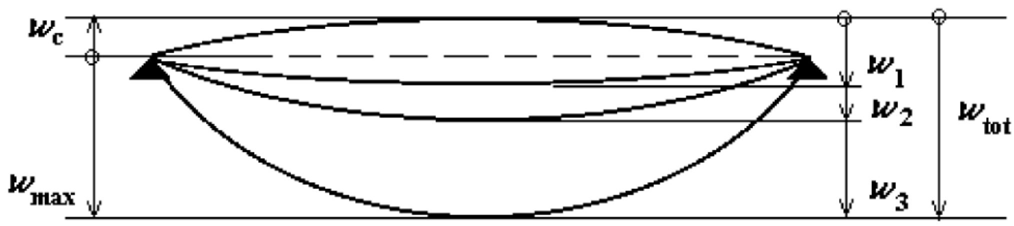


Fig. 3: Definitions of vertical deformations

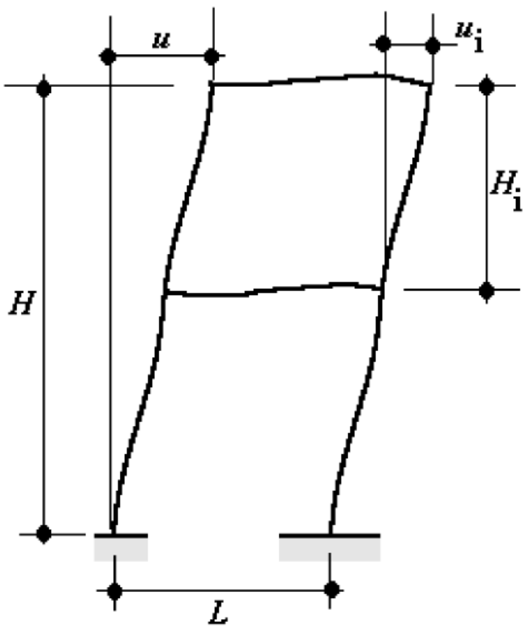


Fig. 4: Definition of horizontal displacements

The maximum values for Austria are defined in the national annex. [14]

For the proof of the criteria it has to verify that the design values of the criterion for serviceability are greater than or equal to the design value of the effects of action on the specific criterion. [15]

---

[14] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter A 1.4

[15] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter A 6.5

3.2. Guidelines trough EN 1996 – Design of masonry structures

Because Wilhelminian buildings are mostly made of bricks, the standard ÖNORM EN 1996 is very important. The standard EN 1996-1-1 is specific for masonry structure for the requirements of safety, serviceability and durability. It uses the partial factor method for limit states.

3.2.1. Material

Masonry Units [16]

To evaluate the compressive strength of masonry units, the normalized mean compressive strength f_b is used. The value of the normalized mean compressive is provided by the manufacturer. If the manufacturer makes no declaration, converting the value is permitted. For existing buildings with no information of the properties, testing of materials must be conducted.

Mortar [17]

Mortar is used to create the adhesion between the masonry units. There are different types of mortar varying by their composition: general purpose mortar, thin layer mortar and lightweight mortar. Mortar should be called M followed by their compressive strength in N/mm². For example M5. The compressive strength is expressed with the variable f_m.

[16] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006. Chapter 3.1

[17] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006. Chapter 3.2

Characteristic compressive strength of masonry [18]

The characteristic compressive strength of masonry f_k is depending on material properties of the masonry unit and the mortar. To determine f_k it is necessary to conduct laboratory tests with samples of the masonry of the project. If there are already values for the given masonry in a database, it is permitted to use such data. When the single components are known, the characteristic compressive strength could be determined by means of the formula 3.1. f_k is the characteristic value and base for the design value. To compute the design compressive strength of masonry f_d it is necessary to multiply f_k with the partial factor for the material.

$$f_k = K \cdot f_b^{\alpha} \cdot f_m^{\beta} \quad (3.1)$$

where:

f_k is the characteristic compressive strength of masonry in N/mm²

K is the constant through table in national annex

α, β are the constants through table in national annex

f_b is the normalized mean compressive strength of units in N/mm²

f_m is the compressive strength of the mortar in N/mm²

If the masonry has joints parallel to the wall (masonry bond), the calculated characteristic compressive strength of masonry hast to be reduction for by 20 percent. Therefore a parameter of 0,80 has to multiplied with the result. [19]

For the design of the vertical load resistance N_{Rd} can the following formula be used:

---

[18] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006. Chapter 3.6.1

[19] Österreichisches Normungsinstitut: ÖNORM B 1991-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 1.3.2009. Chapter 4.2

$N_{Rd} = \phi_s * f_d * A$ (3.2)

where

ϕ_s reduction factor depending on the position inside the building

f_d the design compressive strength of the masonry

A is the loaded gross-section area

Characteristic shear strength of masonry [20]

The value of characteristic shear strength of masonry can also be determined through tests in the laboratory with test samples of the masonry of the project or through existing values in a database. Under given circumstances, the characteristic shear strength could be determined with the formula 3.3 when joints comply with the requirements or 3.4 when perpend joints are unfilled.

$f_{vk} = f_{vko} + 0{,}4\,\sigma_d$ but not greater than 0,065 f_b (3.3)

and

$f_{vk} = 0{,}5\,f_{vko} + 0{,}4\,\sigma_d$ but not greater than 0,045 f_b (3.4)

where

f_{vk} is the characteristic shear strength of the masonry

f_{vko} is the characteristic initial shear strength through tests or via the national annex table

σ_d is the design compressive stress perpendicular to the shear

f_b is the normalized mean compressive strength of units

[20] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006. Chapter 3.6.2

Shear resistance of the wall[21]

For the ultimate limit state it has to be shown that the shear resistance of the wall is big enough to resist the shear load. $V_{Rd} \geq V_{Ed}$ To calculate the shear resistance of an unreinforced masonry wall, the following formula 3.5 can be used:

$$V_{Rd} = f_{vd} \cdot t \cdot l_o \quad (3.5)$$

where

f_{vd} is the design value of the shear strength of the masonry f_{vko}/γ_M

t is the thickness of the wall

l_o is the effective length of the wall

The maximum horizontal load on a shear wall can be reduced by 15 percent when other walls parallel to the shear wall are able to gather this force.

For rigid diaphragms can the horizontal forces distributed to the stiffness of the Shear wall. Shear walls which are connected to intersecting walls can be influenced through a contributive flange which allows to increase the stiffness of the wall. [22]

Characteristic flexural strength of masonry [23]

For the characteristic flexural strength of masonry two values are defined. The first one is defined with the failure plane parallel to the bed joints and is expressed with the variable f_{xk1}. The second value is defined with the failure plane perpendicular

[21] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006. Chapter 6.2

[22] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006. Chapter 5.5.3

[23] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006. Chapter 3.6.3

to the bed joints and is expressed with the variable f_{xk2}. The values can be determined through tests on the masonry or can be taken from a defined table.

Elastic modulus [24]

The short term secant modulus of elasticity E should be determined through tests as described in EN 1052-1. If no values are available, the short term secant modulus of elasticity can be calculated with 1.000 times f_k.

Modulus of rigidity [25]

The modulus of rigidity, or also shear modulus G can be calculated with 40 percent of the E.

In order to create a design value, the partial factor γ_M is needed. Therefore the characteristic value of a feature must be divided by the partial factor to be sure that every part is sufficiently dimensioned. This is part of the safety concept which is adopted for Austria.

The partial factor that has to be used in accordance with EN 1990 can be found in EN 1996-3. It is defined by different classes and by the category of the units. In Austria only class 3 may be used.[26]

The slenderness ratio of a wall can be calculated through h_{ef}/t_{ef} and is limited with 27.[27]

---

[24] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006. Chapter 3.7.2

[25] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006. Chapter 3.7.3

[26] Österreichisches Normungsinstitut: ÖNORM EN 1996-3. Vereinfachte Berechnungsmethoden für unbewehrte Mauerwerksbauten. 01.07.2006. Chapter 2.3

[27] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006. Chapter 5.5.1.4

4. Guidelines for earthquake

When a building is situated in an earthquake prone area, the building must meet additional requirements. These demands on quality and material are defined in the Eurocode 8, which in turn is reflected in the European Standards ÖNORM EN 1998. The first part gives the basic information for seismic actions. Part three is dealing with assessment and retrofitting of existing buildings.

These technical standards represent the course of action of Austria. All requirements arise through the knowledge of today's research and development results.

In the following paragraphs, the main points of each standard are explained.

4.1. Guidelines trough EN 1998-1 – Design of structures for earthquake resistance

4.1.1. Basic principles of conceptual design[28]

To ensure safety against an earthquake hazard, some important facts should be taken into account as early as possible. For new building, the conceptual design should include these principles right from the start. But also existing buildings should utilize these principles for renovations or structural alteration as a result of changes of use. With these criteria, it should be possible to create a better starting position for earthquake resistance.

- Improved safety can be reached with structural simplicity, which can define clear and direct paths for the transmission of the seismic forces. Buildings which were built in consideration of structural simplicity have reliable,

---

[28] Österreichisches Normungsinstitut: ÖNORM B 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 4.2

predictable behavior during earthquakes because they have a less uncertainty.

- Another criterion is the uniformity and symmetry on plan view. When the bearing components are evenly spread in both directions, it enables a short and direct transmission of the inertia forces. Not only on the plan view is the uniformity important, the height of the structure should be symmetrical.

- Because of the fact that seismic motion is a bi-directional force, the building should be given a bi-directional resistance in both directions. The best way to create resistance and stiffness is to place the walls in an orthogonal pattern.

- Another point is torsional resistance. When the bearing elements are placed in an uneven way it leads to torsional motions. To counteract this, it is useful to place the most important parts close to the periphery.

- A further criterion is that the ceilings should work as a plate. They play a very important role in the overall seismic behavior of the building. When they are built in a way they can act like stiff diaphragms, they are able to collect and transmit the inertia forces to the vertical structural elements.

- The last point is the foundation. The construction of the foundation is important for the connection with the structure above. With the right choice, it ensures that the whole building is exposed to an even seismic excitation.

4.1.2. Fundamental requirements

To meet the demands of no collapse requirements, the building must withstand the design seismic action. The design seismic action is expressed in terms of the reference seismic action and the importance factor γ_I.

The reference seismic action could be described with a reference probability of exceedance of 50 years which means P_{NCR} = 10 % or a reference return period of T_{NCR} = 475 years.

For the damage limitation requirements, the structure should be constructed to withstand a larger probability of occurrence than the design seismic action. Therefore the standard recommends a probability of exceedance of 10 years which means P_{DLR} = 10 % and a reference return period of T_{DLR} = 95 years. [29]

With regard to damage limitation, a reduction factor v goes into account to consider a lower return period. The value depends on the importance classes and is calculated through multiplying the importance factor γ_I with the reduction factor v. [30]

To achieve the requirements it is necessary to prove at first the ultimate limit state, whereby influences that could lead to a construction failure wherein the safety of people could be in danger and second the damage limitations states to preserve the functionality for defined services. [31]

Importance factor

In order to calculate dependableness across different buildings, the design seismic action is multiplied with the importance factor.

Therefore the standard defines four importance classes, which incorporate how important the building is and its corresponding damage mitigation requirements. Factors that influence the importance classes can be a high amount of people

---

[29] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 2

[30] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 4.4.3.2

[31] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 2.2

inside the building - for instance a school; or economic consequences - such as damage to a power plant; or even the potential of any earthquake hazard. A detailed description of these classes can be found in the general standard. Respectively, the corresponding values for γ_I can be found in the national annex of the country.

In general, the importance classes roughly equate to the consequence classes CC1 to CC3 which are defined in EN 1990.[32]

4.1.3. Foundation and ground condition

The foundation of a building looms large, because the stiffness of the foundation is responsible for the transmitting of the actions into the ground. The condition of the ground gives additional parameters for the seismic action. Therefore categorization of different ground types, from A to E, exists. Also an outcome of this is the soil factor S which will be required in the following chapters.

In areas with higher risk of earthquakes, additional investigations should be performed. In Austria investigations are recommended for zones 3 and 4 and the importance class of 3 and 4. [33] [34] [35]

4.1.4. Seismic zones

Every country has to section their land area into seismic zones that define relative earthquake hazard. Austria has defined the zones from 0 to 4 where 0 equates to a low risk of earthquake and 4 means a high risk of earthquake.

---

[32] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 4.2.5

[33] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 2.2.4.2

[34] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 3

[35] Österreichisches Normungsinstitut: ÖNORM B 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 7

The description of these hazards occurs through the parameter of the reference peak ground acceleration a_{gR}. This value is provided by the National Authorities and refers to T_{NCR} = 475 years and a importance factor of γ_I = 1,0. The relevant value can be found in the location index. As an example, most parts of Vienna are in zone 3, where the value for a_{gR} is defined with 0,8 m/s².

Unreinforced masonry can be used up to a value of 0,2 g.[36]

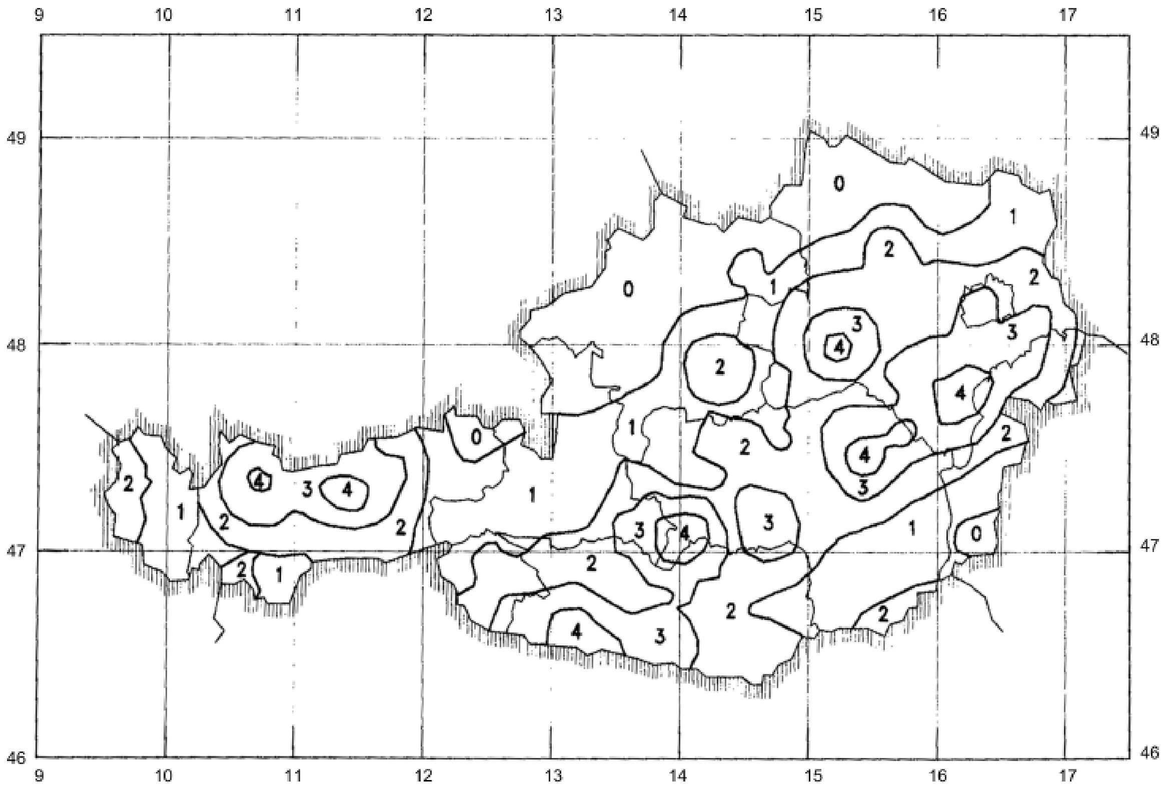


Fig. 5: Zone classification of Austria

If $a_g \cdot S$ is not greater than 0,1 g (0,98 m/s²), it is called low seismicity which allows a simplified calculation for seismic design.

If $a_g \cdot S$ is not greater than 0,05 g (0,49 m/s²), it is called very low seismicity. In this case, the standards at EN 1998 do not needed to be observed. [37] [38]

---

[36] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 9.3

[37] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 3.2.1

4.1.5. Seismic action [39] [40]

Earthquake motion is represented at a given point on the surface by an elastic ground acceleration response spectrum. In combination of the ground type it describes the different phases of an earthquake. EN 1998 describes two types of horizontal elastic response spectrums, whereas Austria uses only type 1. It is used for both limit states.

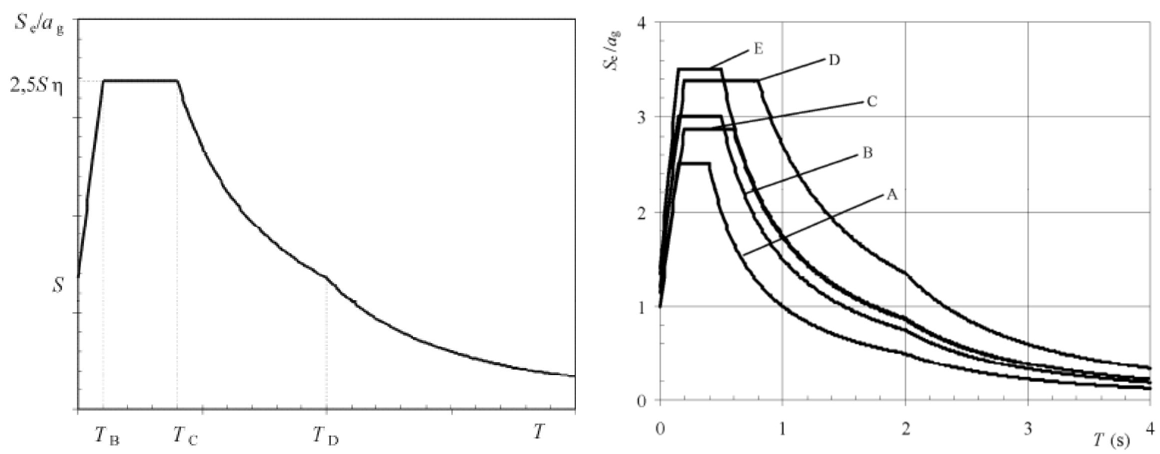


Fig. 6: Shape of the elastic response spectrum

For the horizontal course, a_g is set in full amount and for the vertical course the amount of a_{vg} is set by 2/3 of a_g. If a_{vg} is smaller or equal to 0,25 g, the vertical component is insignificant. Accordingly, the vertical component is not relevant in Austria.

[38] Österreichisches Normungsinstitut: ÖNORM B 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 7.2.2

[39] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 3.2.2

[40] Österreichisches Normungsinstitut: ÖNORM B 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 7.2.5

4.1.6. Combination of the seismic action with other action

The influences on a building can affect the structure in combination. Therefore, the EN 1990 gives a formula for calculation of the design value.[41]

$$\mathrm{Ed} = \sum_{j\geq 1} G_{k,j} + P + A_{Ed} + \sum_{i\geq 1} \Psi_{2,i}\, Q_{k,i} \quad (4.1)$$

With $\Sigma G_{k,j} + \Sigma \psi_{E,i}\, Q_{k,i}$ the masses, permanent as well as variable, of the whole building are going into account for the calculation of design value. Because it is unlikely that all variable loads are present at the time during an earthquake, the calculation considers the combination coefficient ψ_{Ei}. Usually ψ_{Ei} is define as 0,3 because:

$$\psi_{Ei} = \varphi \cdot \psi_{2,I} \quad (4.2)$$

In Austria φ is set by 1,0. That means that $\psi_{Ei} = \psi_{2,I}$. [42]

The declaration for the value of ψ_2 can be found in the annex of the EN 1990. For residential or office buildings, which Wilhelminian buildings are mostly used, is ψ_2 set by 0,3. [43]

A masonry wall has to reach a minimum thickness depending on different applications. In order to be a load bearing wall, the thickness should reach a minimum of 17 cm and in order to stiffen 12 cm.[44]

---

[41] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter 6.4.3.4

[42] Österreichisches Normungsinstitut: ÖNORM B 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 8.1.2

[43] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003. Chapter A 1.2.2

[44] Österreichisches Normungsinstitut: ÖNORM B 1991-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 1.3.2009. Chapter 4.4

4.1.7. Methods of analysis [45]

The standard defines four different methods for calculation: two types of linear elastic analysis and alternatively two types of non-linear analysis.

Linear-elastic methods

- lateral force method of analysis
- modal response spectrum analysis

Non-linear methods

- non-linear static analysis (pushover)
- non-linear time history analysis (dynamic)

Lateral force method of analysis

The lateral force method can be used for buildings whose response is essentially unaffected by modes of vibration higher than the fundamental mode in each principal direction. This condition can be considered as having been achieved if the building has regularity in elevation and the fundamental periods of vibration T_1 are not in excess a specified value.

The lateral force method is chosen for the calculation of a theoretical building. Therefore the analysis will be described more detailed during the practical example calculation in chapter 6.1.

Modal response spectrum analysis

Modal response spectrum analysis can be used for buildings which do not meet the conditions of the lateral force method of analysis. This type of analysis works with the response of all modes of vibration which add to the global response. Therefore all modes which have a effective modal mass greater than 5 % of the

[45] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 4.3.3

total mass of the building must be taken into account and the sum of all the effective modal masses must include at least 90 % of the total mass.

The maximum value E_E of a seismic action effect must be calculated with all relevant modal responses.

$$E_E = \sqrt{\sum E_{Ei}^2} \quad (4.3)$$

where

E_E is the seismic action effect under consideration

E_{Ei} is the value of this seismic action effect due to the vibration mode i

For non-linear methods, a mathematical model which includes the strength of structural elements and their post elastic behavior is used. For masonry buildings, it is important to use the elastic stiffness of the bilinear force deformation of the cracked section. If no specific definition exists, the values for properties of the material should base on mean values.

The non linear static analysis pushover can be used for evaluating new and existing buildings. It considers the forces under conditions of constant gravity loads and monotonically increasing horizontal loads. Depending on the regularity, criteria can be used a spatial model or two planar models. For the lateral loads, two different vertical distributions should be used.

When the control displacement lies between zero and 150 percent of the target displacement, the relation between base shear force and control displacement can be determined through the pushover analysis.

With the non linear time history analysis it is possible to calculate the time dependent response of the structure to represent the ground motions. If at least 7 nonlinear time history analyses are obtained, the average of the response quantities from all of these analyses should be used as the design value of the action effect E_d. Otherwise the most unfavorable value among the analyses should be used.

4.1.8. Specific rules for masonry buildings [46]

The requirements through standard EN 1996 apply to masonry buildings. The standard EN 1998-1 gives additional rules for certain materials where there is seismic risk.

The minimum strength of the masonry unit is defined in the national annex depending on the intensity of the seismic action. If a_g times S is smaller or equal to 0,10 multiplied with the acceleration of gravity, the minimum strength is set at 2,5 N/mm². Otherwise the minimum is 5 N/mm².

The same applies for the minimum strength for mortar. If a_g times S is smaller or equal to 0,15 multiplied with the acceleration of gravity, the minimum strength is set at 1,0 N/mm². Otherwise the minimum is 2,5 N/mm².

That means, when a Wilhelminian building cannot meet this requirement, the building cannot achieve the assumption against earthquake. Often, this is a problem because according actual measurements, buildings show an average of 1-3 N/mm² for the strength for mortar. [47]

---

[46] Österreichisches Normungsinstitut: ÖNORM B 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 13.1

[47] Hollinsky, Karlheinz: Kapitel 04 – Wände im Bestand-Wandertüchtigung, FH Campus Wien, Skriptum. WS 2011/2012

4.2. Guidelines trough EN 1998-3 – Assessment and retrofitting of buildings

Standard EN 1998-3 defines three limit states, near collapse, significant damage and damage limitation. In Austria only the second, significant damage, has to be proven. The main difference between these limit states is the definition of the reference return period.

4.2.1. National Annex B 1998-3[48]

Austria uses a passive program for seismic evaluation and retrofitting. This means that in general, an owner cannot be compelled to make constructive improvements. However, in some cases it can become mandatory and the owner has to make these improvements. Such cases include changes in the construction or using which are not counts as a minor impact.

Given that existing buildings were built at a different time, standards require the retrofitted building meet the minimum requirement in place at the time of the building license. In no case may the building's earthquake resistance be reduced. Exceptions are minor impacts which are defined as a downgrade of a maximum of 3/100 or if the building achieved a higher reliability than was claimed at the time of construction.

The acceptable reliability level Z_{red} and the collapse probability $P_{f,ist,max}$ for existing buildings can be found in the national annex depending on the reliability and consequences classes. For Wilhelminian buildings, which mostly belong in CC2, the value for $P_{f,ist,max}$ is set by 1×10^{-5}.

[48] Österreichisches Normungsinstitut: ÖNORM B 1998-3. Beurteilung und Ertüchtigung von Gebäuden. 01.01.2013. Anhang A

In most cases, the existing buildings cannot achieve the required resistance to withstand the effects.

Therefore the earthquake performance factor α shows the ratio of the design value of the resistance and the design value of the effects. If the value of α is higher or equal to 1,0, it means that the building can achieve the requirements to 100%. If the value is smaller, the collapse probability is higher than it is claimed from the standard. Existing buildings must achieve at least the minimum earthquake performance factor α_{min} which is defined by 0,25 for buildings in class CC2.

α	0,04	0,07	0,12	0,19	0,25	0,31	0,38	0,44	0,50	0,57	0,63	0,70	0,76	0,82	0,89	0,95	1,01	1,08	1,14
$P_f \times 10^{-5}$	60,00	14,71	3,44	1,56	1,00	0,72	0,53	0,41	0,32	0,26	0,21	0,17	0,13	0,11	0,09	0,07	0,05	0,04	0,02

Fig. 7: correlation between earthquake performance factor α and the collapse probability

5. American Standards

The American standard should offer a different perception of the course of action in seismic design. With the comment of the description of requirements and properties to American technical standard should be able to compare the knowledge between America and Austria.

In contrast to Austria America has a huge variety of technical standards and building codes. Not all standards are legislatively obligatory but still can be used for design. Because the United States has many different land areas with different influences and focal points on the environment and natural disasters, it is difficult to create a general standard for the whole country. One general standard is the International Building Code. The IBC is developed to consolidate existing building codes into one uniform code that could be used nationally and internationally to construct buildings. Other standards are focused on a special area, for example San Francisco has additional requirements for earthquake resistance. The standards are different for special buildings like hospitals or schools, or different materials such as masonry, steel or concrete. Therefore there are many different institutes which offer different building codes. Each organization has its own focus and research field. Some of these fields overlap with other research fields and so there are also code edition which are published cooperatively. On the other hand, there are codes which describe the same field with different conclusions.

As long as the standard is not adopted by government regulation, it has no legal status and is therefore called a model building code. Organizations which offer information for seismic design are the Earthquake Engineering Research Institute (EERI), American Society for Testing and Materials (ASTM), American National Standard Institute (ANSI), American Society of Civil Engineers (ASCE), The Masonry Society (TMS), American Concrete Institute (ACI), National Institute of Standards and Technology (NIST), and more.

For the reason that many codes are available, the description should be focused on one institute. Therefore the standards of ASCE are chosen.

5.1. ASCE/SEI 7-10 Standard – Minimum Design Loads for Buildings and other Structures

The size of an earthquake is described in terms of its magnitude and resulting intensity of ground shaking. Typically the Richter scale is used wherein the magnitude is about two thirds of the European Macroseismic Scale magnitude. Magnitude is a measure of the overall energy released in an earthquake, while the resulting intensity of the ground shaking is a description of the effects experienced at a particular location during the earthquake. For measuring intensity, the United States use the Modified Mercalli Intensity scale (MMI) from 1 to 12. [49]

5.1.1. Serviceability

In the United States, the strength limit states are specified in building codes because they control the safety of the structure. The serviceability limit states define a level of quality involving the perceptions and expectations of the owner or user. Therefore serviceability is a contractual matter between the owner and the designer and is for the most part not included within the model United States building codes. It only offers guidance for design in order to maintain the function of the building and helps to provide for the comfort of occupants during their usage. This code includes standards for deflections, vibration, drift and durability. The fact that serviceability limit states are usually not codified should not decrease their importance.[50]

For story drift and deformation, a recommended limit for the allowable story drift Δ_a exists. The factor of the limited drift is defined depending on the material and the

[49] American Society of Civil Engineers: Guidelines for Seismic Evaluation and Design of Petrochemical Facilities. 31.01.2011. Chapter 3.2.2

[50] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter Appendix C1.3.2

risk category. For masonry structures, the factor lies between 0,7% and 1% of the story height.[51]

Note: The European standard EN 1990 describes the defined serviceability limit state which is found in the chapter 3.1.5. In the national annex B 1990-1, the maximum story drift is defined with the height of the story divided by 300, that equates to 0,33% of the height. This results in a much stricter limitation for the story drift than recommended in the ASCI7-10.

5.1.2. Structural Design Basis [52]

The building structure should include complete lateral and vertical force resisting systems to provide the strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand.

The design ground motions assume motion along any horizontal direction of a building structure which applies as the same in the Austrian course of action.

Risk Categorization[53]

Depending to the degree of risk to human life, health, and welfare associated with their damage to property or loss of use or functionality, a whole building can be classified into four risk categories. Category I is for buildings with a low amount of human lives at risk and category IV is used for high occupancy structures. For example, risk category I for buildings with less than 5 persons at risk, while residential and office buildings are among those in category II.

---

[51] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 12.12.1

[52] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 12.1

[53] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 1.5

Within this category, an importance factor for Snow (I_s), Ice (thickness I_i and wind I_w) and Earthquake Loads (I_e) is defined. When a building is at a different risk to each load, multiple risk categories can be taken into account. When a building is divided into independent structural systems separate calculations can be made.

If any part of a building falls in the occupancy category, the overall risk category shall not be taken as lower than the occupancy category.

Note: The risk category and the resulting importance factor are comparable to the importance class and factor defined through the European standard described in chapter 4.1.2. While the risk category mostly depends on the occupancy, the importance class from Austria also takes the environmental and economical relevancy into account.

Soil classes

The site classification occurs through specific geotechnical properties of the top 100 feet (which equates to about 30,5 meter) of the subsurface soil. These properties can be the average shear velocity, the average standard penetration test blow counts or the average shear strength. But the shear wave velocity is considered most reliable and is the preferred approach for site classification. If multiple layers of soil exist, a specific formula can be used to calculate the average value for the property.

The subsurface conditions can be classified in five soil classes ranging from A to E where site class A represents very hard rock and site class E very soft soil. An additional category is class F which is only used for liquefiable soils and soils that become unstable in an earthquake.

If the soil properties are not known in detail the site class D should be used besides the class E or F could be supposed in the particular area.[54]

[54] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 11.4.2

Seismic Design Category[55]

In earthquake standards, a seismic design category is also needed which is related to the risk category. The seismic design categories range from A to F. With the aid of the response acceleration parameters and the risk category, the seismic design category can be determined through tables in the standard.

Buildings with category A need only to achieve the minimum requirements for general proofs.

Ground shaking[56] [57]

Because ground acceleration varies, the response spectrum is used for representation of ground shaking, which describes the maximum response of a series of single degree of freedom oscillators (SDOF) of known period and constant damping plotted as a function of their period of vibration.

The design response spectrum represents an accumulation of a number of possible time histories that define the design demand that the structure is expected to meet. This spectrum is the most common risk assessment method used in design. In special cases where more detailed information is needed, the “acceleration time histories” is used.

The standard refers to this extremely rare event as the Maximum Considered Earthquake (MCE_R). So it has to be reduced with the factor 2/3 to create the design level force.

---

[55] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 11.6

[56] American Society of Civil Engineers: Guidelines for Seismic Evaluation and Design of Petrochemical Facilities. 31.01.2011. Chapter 3.3

[57] American Society of Civil Engineers: Guidelines for Seismic Evaluation and Design of Petrochemical Facilities. 31.01.2011. Chapter 3.4

The ground motion is defined with having a 2 % probability of exceeding in 50 years that means an earthquake which is expected to occur every 2.475 years. But there are still areas with very active faults, like costal California where these assumptions are not sufficient. In this case the ground motion is considered as upper bound motions from a single scenario earthquake. The scenario earthquake mainly represents the maximum earthquake that a particular fault could produce, also known as a characteristic earthquake.

Note: In standards prior to 2000, the probability of exceeding was defined as 10 % in 50 years which mean a return period of about 500 years. This determination is used nowadays for the European description of the reference seismic action.

Design Response Spectrum [58]

The ground motion of the Maximum Considered Earthquake (MCE_R) is based on ground motion maps for the US developed by the United States Geological Survey (USGS). These maps provide the values S_S for spectral acceleration for short period (spectral acceleration at 0,2 seconds), and the value S_1 for 1,0 second spectral period.

These values belong to soil class B. If a different class is present, the spectral response acceleration parameter has to be multiplied with a site coefficient, depending on the amount of parameter and the soil class to get S_{MS} and S_{M1}. For class B the site coefficient is always 1,0. [59]

---

[58] American Society of Civil Engineers: Guidelines for Seismic Evaluation and Design of Petrochemical Facilities. 31.01.2011. Chapter 3.4.1

[59] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 11.4.3

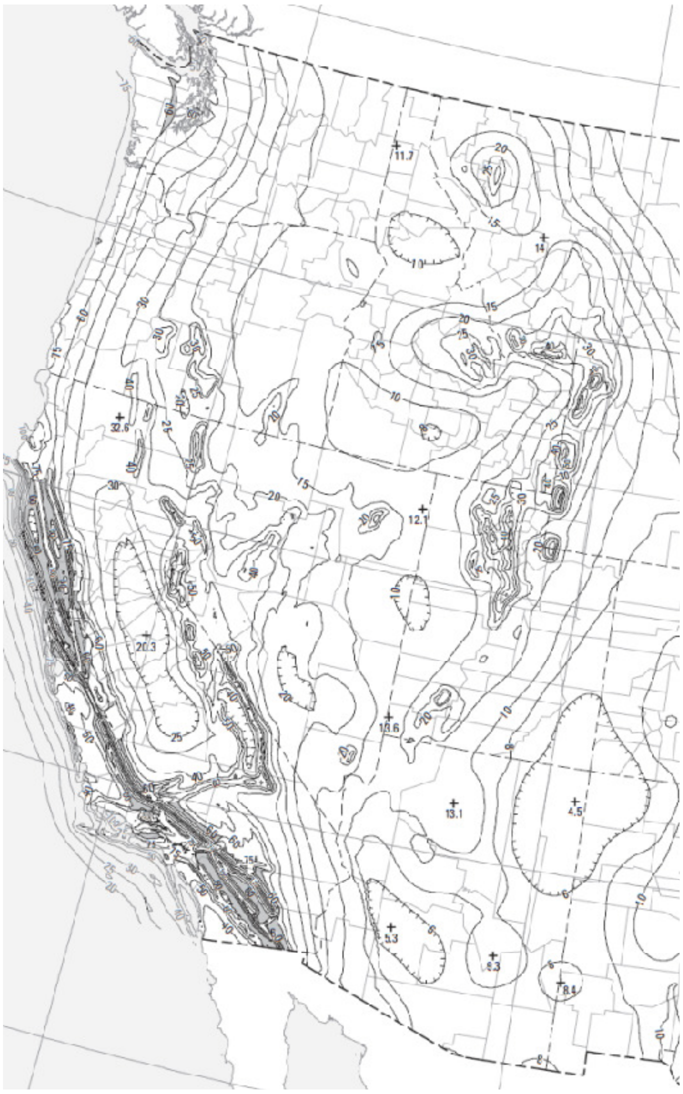


Fig. 8: Cutout of the westcoast ground motion map

For the MCE_R design parameters, the values S_{MS} and S_{M1} have to be reduced by the factor 2/3 that specifies the inherent over strength, to receive the short period ordinate S_{DS} and the 1,0 second period ordinate S_{D1}. These two design parameters are considered sufficient to construct the design response spectrum.[60]

The design response spectrum can be idealized into three distinct regions: spectral acceleration (S_a), spectral velocity (S_v) and spectral displacement (S_d).

[60] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 11.4.4

The region of the spectral acceleration is defined from T_0 to T_S and can be calculated with the following formulas. [61]

$T_0 = 0{,}2 \frac{S_1}{S_S}$ (5.1)

and

$T_S = \frac{S_1}{S_S}$ (5.2)

where

S_S is the spectral response acceleration parameter for short period

S_1 is the spectral response acceleration parameter for 1,0 second spectral period

Historically, the transition from velocity to displacement has been taken as 4,0 seconds. ASCE/SEI 7-10 changed this into variable parameter depending on the region of the country. Nowadays the value for the parameter can be found in the T_L maps.

[61] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 11.4.5

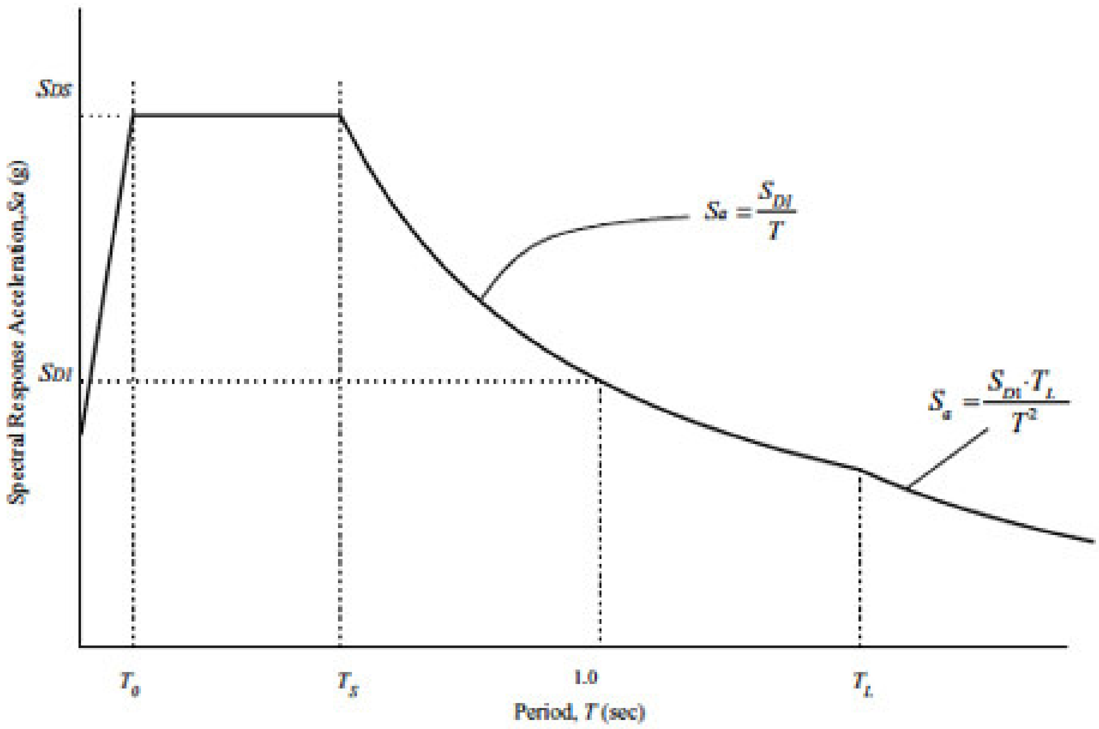


Fig. 9: Design response spectrum defined ba ASCE 7-10

Site-specific design response spectrum[62]

Site-specific design response spectrum can be calculated through a probabilistic seismic hazard analysis and a deterministic seismic hazard assessment. The difference is that the first one is taken as 5% damped response spectrum with 2% probability of exceeding in 50 years and limited by a peak spectral acceleration of 1,5 g and 1,0 second spectral acceleration of 0,6 g.

The second one as 150% of the maximum 5% damped response spectra of maximum earthquakes on all known active faults in the region.

[62] American Society of Civil Engineers: Guidelines for Seismic Evaluation and Design of Petrochemical Facilities. 31.01.2011. Chapter 3.4.2

Redundancy factor [63]

For calculation of the seismic load, a redundancy factor ρ goes into account of the calculation. In most cases ρ is defined by 1,0 for example of seismic design category B or C. For the other cases ρ is equal 1,3. Given that seismic design category A could use a simplified calculation, the redundancy factor is not needed.

Seismic Load effects and combinations

Live Loads [64]

Live load is the load produced by use and occupancy of the building that does not include construction or environmental loads. In table 4-1 of the standard ASCE 7-10 can be found the defined live loads. For example the live load for office areas is set at 50 psf (2,39 kN/m²) and in residential areas at 40 psf (1,92 kN/m²). In case of office buildings where partitions are arranged, the load should be increased by a minimum of 15 psf (0,72 kN/m²). This increase can be omitted when the live load already reaches 80 psf (3,83 kN/m²) or more.

Under compliance with some formalities, a reduction of uniform live loads is permitted. The rules for this reduction are that after a reduction the live load is not allowed to be less than 50 percent of the unreduced load for one floor and not less than 40 percent of the original load for two or more floors. Also the area of the uniform load must cover a minimum area of 400 ft² (37,16 m²). An additional rule exists that it is not permitted for special areas like for example garages or areas with heavy live loads of more than 100 lb/ft² (4,79 kN/m²).[65]

[63] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 12.3.4

[64] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 4

[65] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 4.7

$$L = L_o \left(0,25 + \frac{x}{\sqrt{K_{LL} \cdot A_T}}\right) \quad (5.3)$$

where

L is the reduced live load less than 40 or 50 % of L_o

L_o is the unreduced live load per ft^2 or m^2 of the area

x is 15 for dimensions of ft^2 or 4,57 for dimension of m^2

K_{LL} is the live load element factor from 1 to 4 from table 4-2

A_T is the area in ft^2 or m^2

For one- and two-family dwellings with more than one floor loads, the reduction of live load could also be calculated with the alternative formula:

$$L = 0,7 \cdot (L_{o1} + L_{o2} + \dots) \quad (5.4)$$

where

L_{o1}, L_{o2}, … are the unreduced floor live loads regardless of the tributary area

Effective Seismic Weight [66]

The effective seismic weight W of a building has to be determined for the calculation of the seismic force. It has to include all dead loads, live loads and other loads above the base.

Other loads can be permanent equipment, snow load or also loads of roof greening.

[66] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 12.7.2

Load Combinations [67]

The standard defines seven different load combinations for the strength design. These combination can include dead load (D), live load (L), roof live load (L_r), snow load (S), rain load (R) and earthquake load (E).

Different to Austria, is there no single defined combination which has to be proved for a special situation. All seven combinations have to be verified to determine the worst load combination for a building.

These combinations are:

1. 1,4 D
2. 1,2 D + 1,6 L + 0,5 (L_r or S or R)
3. 1,2 D + 1,6 (L_r or S or R) + (L or 0,5 W)
4. 1,2 D + 1,0 W + L + 0,5 (L_r or S or R)
5. 1,2 D + 1,0 E + L + 0,2 S
6. 0,9 D + 1,0 W
7. 0,9 D + 1,0 E

Earthquake load[68]

For the combinations 5 and 7 where the seismic load effect E is included, E has to be determined. It is defined to use the formula 5.5 for the load combination 5 and formula 5.6 for the load combination 7.

[67] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 2

[68] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 12.4

$$E = Eh + Ev \quad (5.5)$$

$$E = Eh - Ev \quad (5.6)$$

where

E is the seismic load effect

E_h is the effect of horizontal seismic forces

E_v is the effect of vertical seismic forces

The horizontal load effect E_h can be determined with the formula

$$E_h = \rho \cdot Q_E \quad (5.7)$$

where

ρ is the redundancy factor

Q_E is the effects of horizontal seismic forces from the total design lateral force or shear at the base V

The vertical seismic load effect E_v can be determined with the formula

$$E_v = 0{,}2 \cdot S_{DS} \cdot D \quad (5.8)$$

where

S_{DS} is the design spectral response acceleration parameter

D is the effect of the dead load

5.1.3. Analysis procedure

The standard ASCE 7-10 describes a selection of permitted analysis procedure for all seismic design categories. Therefore four basic procedures are recommended:

- the equivalent lateral Force analysis
- the modal response spectrum analysis
- the linear response history analysis and
- the nonlinear response history analysis

The nonlinear pushover analysis is not described in the standard because it is not provided as an approved analysis procedure.[69]

This means that the standard does not recommend the analysis through pushover, but it also does not explicitly prevent its use. If the design calculation is based on another code which includes this kind of analysis, it is permitted.

Note: Generally, the European standard EN 1998-1 recommends the same types of analysis. The difference is that the pushover analysis is a procedure which is permitted and described in the standard.

Under consideration that for the calculation through the Austrian standard is used the lateral force method, the American equivalent lateral Force analysis is used. This method describes a way which is comparable to the Austrian way which offers a better starting position for the detailed consideration.

[69] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter C12.6

5.1.4. Equivalent lateral force procedure

Seismic Base Shear [70]

The seismic base shear V can be calculated with the followed formula:

$$V = C_s \cdot W \quad (5.9)$$

where

C_s the seismic response coefficient

W the effective seismic weight (lb)

The seismic response coefficient C_s can be determined in accordance with the formula [71]

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \quad (5.10)$$

where

S_{DS} is the design spectral response acceleration parameter

R is the response modification factor belonging to the material

I_e is the importance factor for earthquake loads

[70] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 12.8.1

[71] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 12.8.1.1

Vertical distribution of seismic forces [72]

To calculate the seismic forces induced at each level, the following formulas can be used:

$F_x = C_{vx} \cdot V$ (5.11)

$C_{vx} = \frac{w_x \cdot h_x^k}{\sum\limits_{i=1}^{n} w_i \cdot h_i^k}$ (5.12)

where

C_{vx} is the vertical distribution factor

V total design lateral force or shear at the base (kip or lbf)

w_i, w_x are the effective seismic weight of the storey (lb)

h_i, h_x are the heights from the base to the storey (ft)

k is an exponent related to the period (1 <= 0,5s; 2 >= 2,5s; between interpolation)

Horizontal distribution of forces [73]

The seismic design story shear should be spread to the vertical elements of the bearing system. It can be calculated with the equation 5.13.

$V_x = \sum_{i=x}^{n} F_i$ (5.13)

where

F_i is the seismic forces induced at level i

n is the number of stories

[72] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 12.8.3

[73] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter 12.8.4

For buildings with stiff diaphragms, the story shear forces shall be distributed to the lateral-force resisting elements based on their relative rigidities. For buildings with flexible diaphragms, the story shear shall be calculated separately for each line of lateral resistance.

5.2. Masonry

Masonry is a material seldom used in America, and as a result, the standards include less information relevant to this material. The following chapter refers to different sources.

The standard ASCE 7-10 describes the general requirements for earthquake. The detailed requirements for the material are part of other codes, for example ASCE standard “Seismic Evaluation of existing Building” and the code “Building Code Requirements and Specification for Masonry Structures” which arose for masonry structures through the cooperation of the organizations The Masonry Society, the American Concrete Institute and the Structural Engineering Institute.

5.2.1. Building Code Requirements and Specification for Masonry Structures

General

For masonry buildings, the design assumptions include the use of a lateral force resisting system. The distribution of the lateral load to the members of the barring system is a function of the stiffness of the structural system and of the horizontal diaphragms. Lateral loads from wind and seismic forces are normally considered to act in the direction of the axes of the structure. These loads cause forces in perpendicular and parallel direction of the walls. [74]

The analysis of lateral load distribution should be in accordance with accepted procedures. The calculation should consider the effects of openings in shear walls

[74] Joint ACI/ASCE/TMS Committee: Building Code Requirements and Specification for Masonry Structures. 2011. Chapter 1.7

and whether the masonry above the openings allows them to act as coupled or non coupled shear walls.

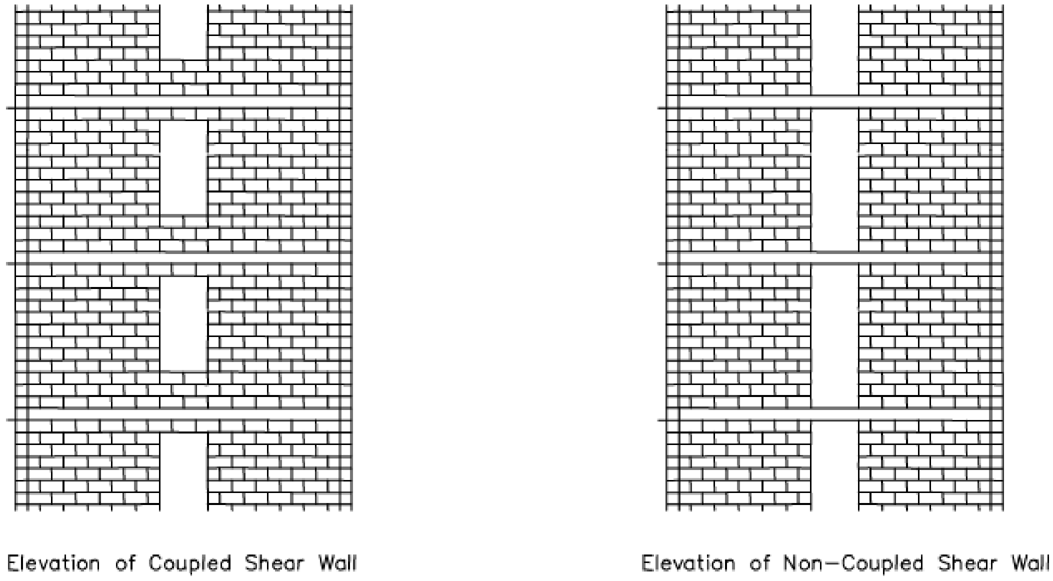


Fig. 10: coupled and noncoupled shear walls

For the computation of the stiffness of shear walls, shearing and flexural deformations must be considered. A guide for solid shear walls with no openings is given in Figure 11. [75]

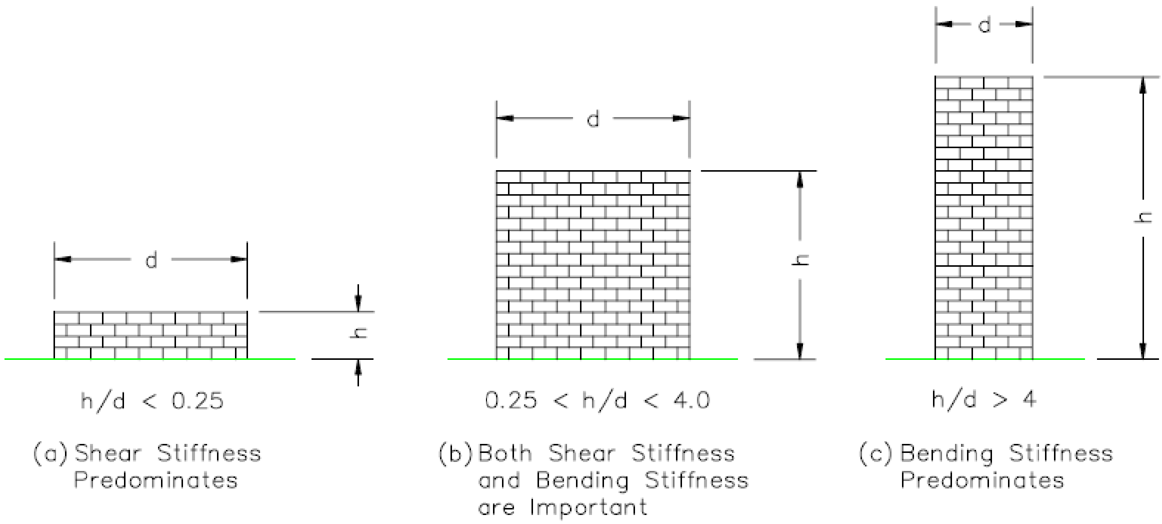


Fig. 11: Shear wall stiffness

75 Joint ACI/ASCE/TMS Committee: Building Code Requirements and Specification for Masonry Structures. 2011. Chapter 1.7.6

Other effects for forces and deformations can affect through prestressing, vibration, impact, shrinkage, expansion, temperature changes, creep, and differential movement. The extent of these forces could be influenced by the choice of materials, structural connections and geometric configuration. All these effects have to be considered for the dimension of the structural system. [76]

Seismic design requirements for masonry[77]

Minimum seismic loading requirements are taken from the legally adopted building code. In circumstances where the code does not contain defined criteria for the determination of seismic forces, the Code requires the use of ASCE 7-10, which represents the state of the art in seismic design at the time when the requirements for masonry were developed.

Elastic modulus[78]

Modulus of elasticity E_m for masonry built of clay tiles has traditionally been taken as 1.000 f `m in previous masonry codes, but research has indicated that there is a large variation in the relationship of elastic modulus and the compressive strength of masonry. Therefore the conclusion arises that lower values are more typical. Therefore the definition of the value E_m for clay masonry has been changed into 700 times the characteristic compressive strength of masonry.

Note: The Austrian standard defines the elastic modulus with 1.000 times the characteristic compressive strength of masonry which is in accordance with the older American code.

---

[76] Joint ACI/ASCE/TMS Committee: Building Code Requirements and Specification for Masonry Structures. 2011. Chapter 1.7.5

[77] Joint ACI/ASCE/TMS Committee: Building Code Requirements and Specification for Masonry Structures. 2011. Chapter 1.18

[78] Joint ACI/ASCE/TMS Committee: Building Code Requirements and Specification for Masonry Structures. 2011. Chapter 1.8.2

Modulus of rigidity[79]

The modulus of rigidity E_v can be calculated with 40 % of the E_m which comply with the Austrian definition of G.

Bearing wall systems made of masonry [80]

For new constructions, the choice of masonry is strictly limited. There are three different types of unreinforced masonry walls:

Empirical masonry shear walls are permitted to be used only in seismic design category A. They are not designed for or required to contain reinforcement.

Ordinary plain masonry walls are permitted to be used in seismic design categories A and B. They are designed as unreinforced masonry although they may contain reinforcement.

Detailed plain masonry walls are designed as unreinforced but contain reinforcement in horizontal and vertical direction which does not reach the minimum reinforcement requirements. They are also only allowed to be used in seismic design categories A and B, but through the reinforcement the seismic design parameters are higher than plain masonry shear walls.

In the other categories, an unreinforced masonry structure is not permitted.

[79] Joint ACI/ASCE/TMS Committee: Building Code Requirements and Specification for Masonry Structures. 2011. Chapter 1.8.2.2.2

[80] Joint ACI/ASCE/TMS Committee: Building Code Requirements and Specification for Masonry Structures. 2011. Chapter 1.18.3.2

Existing Building Provision[81]

If additions are made to an existing building, standards provide for two different variants as follows:

An addition which is structural independent from the rest of the building has to meet the requirements for new structures without consequences to the existing structure.

If the addition is not independent to the existing structure, the alteration must be constructed that the whole building conforms to the seismic force resistance requirements for new structures. This can be neglected if the addition complies with the requirement for the new structure, it does not increase the seismic force in any structural element by more than 10 % and the addition does not decrease the seismic resistance of any structural element.

Where a change of use occur, such as higher occupancy like the category which was defined during the calculation at construction, it has to be proved that the whole building still conforms to the requirements of new structures.

5.2.2. Seismic Evaluation of existing buildings

For existing buildings there is a separate guideline for meeting seismic evaluation standards.

Depending on the level of seismicity, the level of performance and the construction itself, lead to the necessary evaluation phase for seismicity or a combination of phases. Therefore are 3 different sections Tier 1 – Screening Phase, Tier 2 – Evaluation Phase and Tier 3 – Detailed Evaluation Phase. Every Tier has it one checklist for the ascertainment of the required information. [82]

---

[81] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter Appendix 11B

[82] American Society of Civil Engineers: Seismic Evaluation of Existing Buildings. 01.01.2003. Chapter 1

Levels of Seismicity Definitions [83]

The intensity of the seismicity can be classified into three levels and are defined through the following table with the values of S_{DS} and S_{D1}.

Level of Seismicity	S_{DS}	S_{D1}
Low	< 0,167 g	< 0,067 g
Moderate	≥ 0,167 g and < 0,500 g	≥ 0,067 g and < 0,200 g
High	≥ 0,500 g	≥ 0,200 g

Tab. 1: Levels of seismicity definitions

If circumstances are such that S_{DS} and S_{D1} lead to different level of seismicity, the site should be classified as moderate.

Level of Performance [84]

The performance level has to be defined prior a seismic evaluation procedure because they require a different amount and preciseness of information. The standard differs between two performance levels, the life safety and the immediate occupancy performance level.

Life safety performance level is defined as follows: damages that occur during a design earthquake where partial or total structural collapse does not occur and on the other hand the damages to nonstructural components are non-life-threatening.

Immediate occupancy performance level is defined as follows: damages occur during a design earthquake where the damages are not life-threatening, so as to permit immediate occupancy of the building after a design earthquake, and on the other hand sustained damages are repairable while the building is occupied.

[83] American Society of Civil Engineers: Seismic Evaluation of Existing Buildings. 01.01.2003. Chapter 2.5

[84] American Society of Civil Engineers: Seismic Evaluation of Existing Buildings. 01.01.2003. Chapter 2.4

Analysis [85]

For the evaluation process of existing building, quick checks are used to calculate the stiffness and strength of certain building components. The calculation method differs slightly from the procedure described in the code ASCE 7-10.

The use of default values for material properties is permitted for the evaluation. If there are no determined values though material examinations, for the compressive strength of masonry f'_m is set at 1.000 psi (6,89 N/mm²) and the mortar shear strength v_{te} is set at 10 psi (0,0689 N/mm²). In some cases, for example for detailed calculations of Tier 2 or Tier 3, standards could require tests of the material properties to get detailed information. [86]

For quick check calculation of the seismic shear force, the pseudo lateral force is used. The pseudo lateral force does not represent an actual lateral force that the building must resist in traditional design codes. The difference is that the normal calculation of the seismic base shear, described in chapter 5.1.4 with the equation 5.9, includes a response modification factor R that leads to a base shear which is representative for the internal forces during a design earthquake. However the calculated building displacements are less than real displacements during the design earthquake. In other words, the procedure is based on equivalent lateral forces and pseudo displacements.

In the analysis procedure through the standard “Seismic Evaluation of Existing Buildings” is based on pseudo lateral forces to obtain a higher lateral force but also the actual displacements during a design earthquake.

To calculate the pseudo lateral force, the equation 5.14 should be used. The horizontal and vertical distribution of the seismic forces is determined through the

---

[85] American Society of Civil Engineers: Seismic Evaluation of Existing Buildings. 01.01.2003. Chapter 3.5

[86] American Society of Civil Engineers: Seismic Evaluation of Existing Buildings. 01.01.2003. Chapter 2.2

same procedure as described in ASCE 7-10 with the difference that the pseudo lateral force is used.

$$V = C \cdot S_a \cdot W \quad (5.14)$$

where

C is the modification factor for displacements, for unreinforced masonry 1,0

S_a is the response spectral acceleration

W is the effective seismic weight of the building (lb)

The used spectral acceleration S_a can be computed in accordance with the formula:

$$S_a = \frac{S_{D1}}{T} \quad \text{but not greater than } S_{DS} \quad (5.15)$$

where

S_{D1} is the design parameter of the spectral acceleration at 1.0 second period

S_{DS} is the design parameter of the short period spectral acceleration

T is the fundamental period of vibration

Shear Stress

The calculation of the shear stress can be computed in accordance with the Tier 1 and Tier 2 phase with a separate course of action. Depending on the grad of detail, the average shear stress or the expected masonry strength is used to define the influences to a wall.

As a rough estimate of the shear stress in the shear walls, the calculation of the average shear stress v_x^{avg} can be used. It can be calculated with the area of all shear walls in the level. For unreinforced masonry walls, the shear stress should not exceed 30 psi (0,2 N/mm²). If v_x^{avg} exceeds the maximum value, a more detailed calculation is necessary. [87] [88]

$$v_x^{avg} = \frac{1}{m} \cdot \left(\frac{V_x}{A_w}\right) \quad (5.16)$$

where

m is the component modification factor which is defined for unreinforced masonry with 1,5

V_x is the seismic design story shear at the level x (lb)

A_w is the amount of the horizontal cross-sectional area of the shear walls (in²)

For detailed calculations in Tier 2 phase, the expected unreinforced masonry strength v_{me} is used. In this case is the mortar shear strength through in-place mortar test necessary. v_{me} can be determined in accordance with the equation: [89]

$$v_{me} = 0{,}56 \cdot v_{te} + \frac{0{,}75\, P_D}{A_n} \quad (5.17)$$

where

v_{te} is the mortar shear strength (psi)

P_D is the superimposed dead load at the top of the wall (lb)

A_n is the net mortared area of the wall (in²)

This formula is part of the detailed calculation and as a consequence it requires an in-place testing of the mortar shear strength. The value is defined as the test result

---

[87] American Society of Civil Engineers: Seismic Evaluation of Existing Buildings. 01.01.2003. Chapter 3.5.3.3

[88] American Society of Civil Engineers: Seismic Evaluation of Existing Buildings. 01.01.2003. Chapter 4.4.2.5.1

[89] American Society of Civil Engineers: Seismic Evaluation of Existing Buildings. 01.01.2003. Chapter 3.5.3.3

which is exceeded by 80 percent of all mortar shear tests. If the value is less than 30 psi (0,207 N/mm²) the test has to be repeated. As a comparison, Austria often has to handle with mortar shear strength less than 30 psi. Therefore should this requirement be neglected to allow taking a lesser shear strength of the masonry like it is common in Austria.[90]

Calculation of single walls

For a single wall it is necessary to calculate the shear wall strength to compare the resistance and the influences. The story force distribution to a single shear wall at any level V_{wx} must be determined. The sum of all story forces results in the shear wall action that the wall is exposed to.

Shear wall strength

In additional to the shear wall strength of an unreinforced masonry wall V_a should be calculated. The openings inside a wall have to been taken into account for the calculation. Each pier of the wall has to be computed separated.

$$V_a = 0{,}67 \cdot v_{me} \cdot D \cdot t \quad (5.18)$$

where

v_{me} is the expected masonry shear strength (psi)

D is the length of the undisturbed wall (in)

t is the thickness of the wall (in)

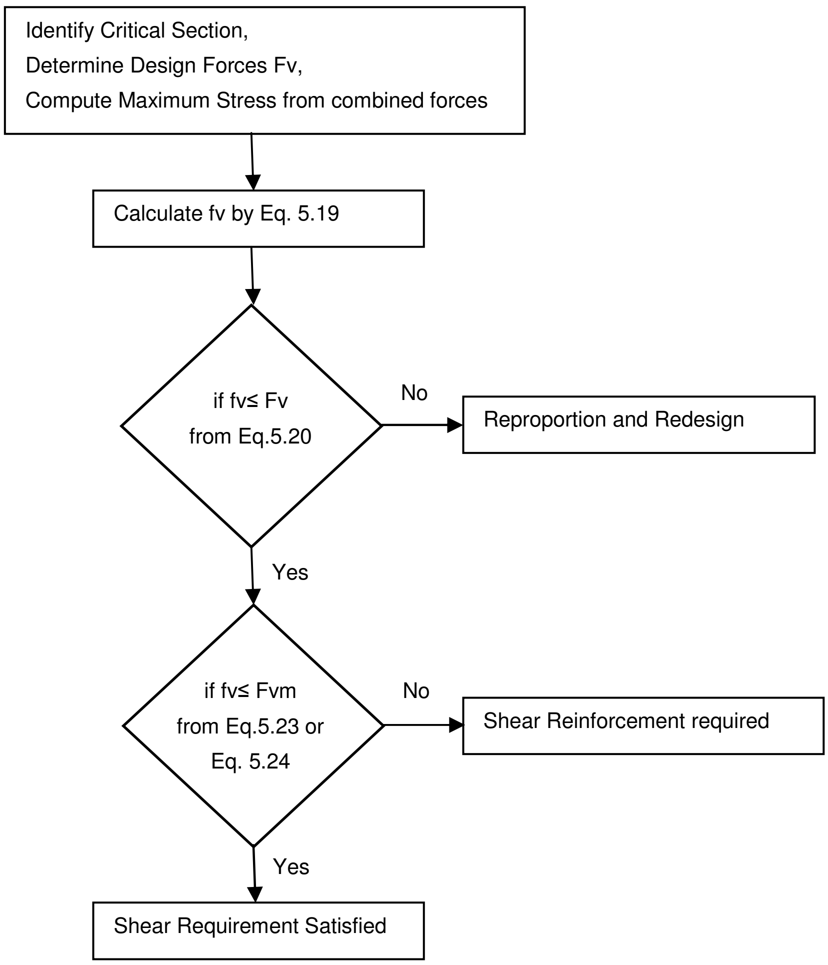
The determined strength has to exceed the shear.

[90] American Society of Civil Engineers: Seismic Evaluation of Existing Buildings. 01.01.2003. Chapter 4.2.6.2.2.1

5.2.3. Shear Requirement [91]

Another possibility as the calculation for existing buildings offers the standard Building Code Requirements and Specification for Masonry Structures. Interesting is, different to the other course of action and Austria, that the calculation only consider the shear stress of the masonry which is only influenced by the compressive strength of masonry. With the proof of compliance of the allowable shear stress of the masonry is the requirement achieved.

To calculate the shear stress can occur through the step by step guidance in accordance with the followed flow process chart:



[91] Joint ACI/ASCE/TMS Committee: Building Code Requirements and Specification for Masonry Structures. 2011. Chapter 2.3.1.1

Calculated shear stress per square inch in masonry shall be determined by the relationship:

$$f_v = \frac{V}{A_{nv}} \quad (5.19)$$

where

f_v is the calculated shear stress in masonry, psi

V is the shear force, lbf

A_{nv} is the net cross-sectional area of member, in^2

The calculated shear stress f_v shall not exceed the allowable shear stress F_v where F_v shall be computed using Equation 5.20 and either Equation 5.21 or Equation 5.22 as appropriate.

$$F_v = F_{vm} + F_{vs} \quad (5.20)$$

where

F_v is the allowable shear stress, psi

F_{vm} is the allowable shear stress resisted by the masonry, psi

F_{vs} is the allowable shear stress resisted by the reinforcement, psi

Due to the fact that masonry cannot carry any kind of tension, all axial and flexural tension must be resisted entirely by steel reinforcement. As Wilhelminian buildings can appear as unreinforced masonry, the possibility of reinforcement for tension forces should be neglected. Therefore F_v should only be determined with F_{vm}.

For further calculation the Shear Ratio M/Vd is needed. The moment is defined with force times distance and Vd describes the force times the effective depth where only compression stress appears. Under canceling the force, the ratio of the distance to depth is given. So, the M/Vd ratio simplifies to the shear span over effective depth radio (a/d).

F_v shall not exceed the following values from equation 5.21 when $M/V_d < 0{,}25$ and equation 5.22 when $M/V_d > 1$. For values between 0,25 and 1 can be interpolated.

$$Fv < 3\sqrt{f'_m} \quad (5.21)$$

$$Fv < 2\sqrt{f'_m} \quad (5.22)$$

where

f'_m is the compressive strength of masonry

The allowable shear stress resisted by the masonry, Fvm, shall be computed using

$M/V_d < 1$

$$F_{vm} = \frac{1}{2} \cdot \left[\left(4{,}0 - 1{,}75 \cdot \left(\frac{M}{V_d} \right) \right) \cdot \sqrt{f'_m} \right] + 0{,}25 \cdot \frac{P}{A_n} \quad (5.23)$$

$M/V_d > 1$

$$F_{vm} = \frac{1}{4} \cdot \left[\left(4{,}0 - 1{,}75 \cdot \left(\frac{M}{V_d} \right) \right) \cdot \sqrt{f'_m} \right] + 0{,}25 \cdot \frac{P}{A_n} \quad (5.24)$$

where

P is the axial load

A_n is the net area, in^2

f'_m is the compressive strength of masonry

6. Calculation of a wall through lateral force method of analysis

An example calculation will illustrate how both standards compare and show whether application of either standard results in the same outcome. If the same outcome is not achieved, the example should illustrate the different factors that influence each standard.

The following step-by-step outline should summarize the calculation of seismic force and structural resistance to it.

The values used below correspond to the requirements of the standard.

6.1. Calculation by the Austrian standard

6.1.1. Influence through seismic force

As already described in section 4.1.7 the lateral force method can be used for buildings whose response is essentially not affected through modes of vibration higher than the fundamental mode in each principal direction. This can be deemed to be satisfied if the building is proved to have regularity in elevation and the fundamental periods of vibration T1 is smaller than the following defined values.

$$T_1 \leq \begin{cases} 4 \cdot T_c \\ 2{,}0\ s \end{cases} \quad (6.1)$$

Therefore T_1 can be calculated with an equation which applies for buildings with a maximum high of 40 meters.

$T_1 = C_t \cdot H^{3/4}$ (6.2)

where

C_t is a factor set by 0,050 for other structures than steel or concrete frames

H is the height of the building

The horizontal components of the seismic action, the design spectrum $S_d(T_1)$, should be determined. Essentially, this depends on the type of soil, the material and the intensity of the area.

The following formula is defined:[92]

$S_d(T_1) = a_g \cdot S \cdot \frac{2,5}{q}$ (6.3)

where

a_g is the design ground acceleration

S is the soil factor

q is the behavior factor

The design ground acceleration is the outcome of reference peak ground acceleration, depending on the defined zones in Austria, and the importance factor.

The behavior factor for unreinforced masonry differs through the compressive strength. If f_m is smaller than 2,5 N/mm² the behavior factor is set by 1,5, if f_m is higher the value rises to 2,0.[93]

---

[92] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 3.2.2.5

[93] Österreichisches Normungsinstitut: ÖNORM B 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 13.2.4

With the design spectrum it is possible to calculate the base shear force for the whole building.

This can be calculated through the formula: [94]

$$F_b = S_d(T_1) \cdot m \cdot \lambda \quad (6.4)$$

where

$S_d(T)$ is the design spectrum

m is the total mass of the building

λ is the correction factor

For the total mass of the building it is necessary to consider all masses as described in chapter 4.1.6.

The correction factor is set at 1,00 if the building has less than three stories. Otherwise the factor is set by 0,85 to take into account that the effective modal mass is on average 15 % smaller than the total building mass.

To turn-over the shear base force, it can be approximated that the horizontal displacement increases linearly along the height of the building.

The horizontal forces in each storey can be determined with the formula:[95]

[94] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 4.3.3.2.2

[95] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 4.3.3.2.3

$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j}$ (6.5)

where

Fb	is the seismic base shear force
zi, zj	are the heights of the storey masses above the level of attack of the seismic force
mi, mj	are the storey masses

These horizontal forces attack at the floors of each story and then spread to the load bearing construction which belongs into the calculated horizontal direction.

If the ceiling is of plate construction with thrust stiffening, the force will split into all walls depending to their length and width. The positive point is that a stronger wall can bear more force.

6.1.2. Resistance of masonry

The most important proof is the shear resistance of the walls. To determine the resistance of the wall V_{Rd}, the equation 3.5 can be used for calculation. For the ultimate limit state, the shear resistance has to resist the shear load during the earthquake.

For the input parameters it is necessary to convert the characteristic initial shear strength f_{vko} with the partial factor γ_M the design value of the shear strength of the masonry $f_{vd.}$ The partial factor for masonry made with units of category II and class 3 is set at 2,5. As described in the standard EN 1998-1, for the seismic design situation it is recommended that the material partial factor γ_M is taken as 2/3 of the value specified in the national annex but not less than 1,5. Under this instruction, a partial factor of 1,67 can be computed.[96]

[96] Österreichisches Normungsinstitut: ÖNORM B 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011. Chapter 9.6

For the effective length, the eccentricity of the force has to be considered. It is computed through M/N. If the force is not in the center, the wall may be subject to tensile strength. Because tensile load cannot be transferred from the foundation bed to the ground soil, it is not allowed to consider them. The length subject to tensile loading appears as a gaping joint. Therefore the total length of the wall has to be reduced if the eccentricity is outside the core of the section (defined by one sixth of the length of the wall). As a safety against overturning, the eccentricity is limited to one third of the length. Should the determined eccentricity be higher than the defined l/6 and less than the maximum l/3, the effective length can be calculated through the formula 6.6.

$$l_o = \left(\frac{l}{2} - e\right) \cdot 3 \quad (6.6)$$

where

l_o is the resulting effective length

l is the total length of the wall

e is the eccentricity

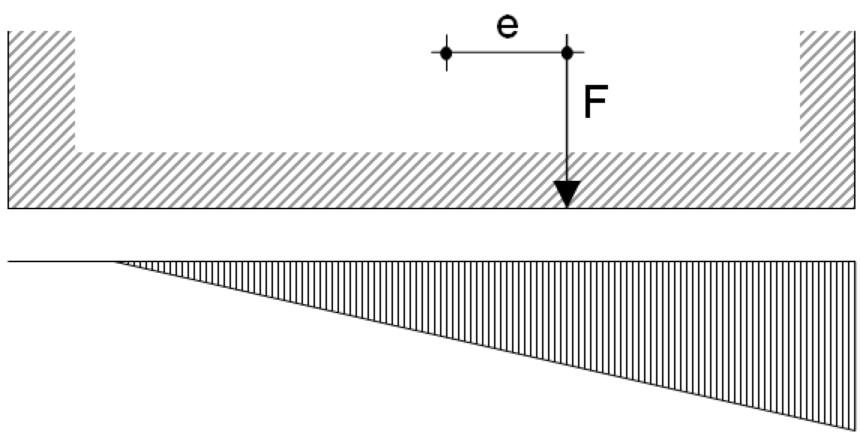


Fig. 12: Figure of the effective length through eccentricity

The resistance of the wall, as calculated, must be greater than the acting forces. If V_{Rd} is higher than V_{Ed} is the condition fulfilled. If the resistance is not enough, the ratio of the design value of the resistance and the design value of the effects can be determined with the earthquake performance factor α. For existing buildings, the value of the resistance has to achieve at least 25% of V_{Ed}.

6.2. Calculation by the American standard

In the American standards, described in chapter 5, two different calculations for lateral force analysis appear: the equivalent lateral force and the pseudo lateral force calculation. Because this example focuses on the shear resistance of a masonry wall, the normal lateral force analysis has been chosen because the input is the reduced equivalent base shear which is more representative for the force during the design earthquake.

When a detailed calculation of the building displacement is desired, the pseudo lateral force calculation should be used.

6.2.1. Influence through seismic force

Influence through the seismic force can be determined as described in chapter 5.1.4. The base shear should be calculated through the equivalent lateral force method with formula 5.9. The part of the seismic share of the weight in ratio to the earthquake is included in the seismic response coefficient C_s defined in equation 5.10.

For the purpose of calculating effective seismic weight, 20 percent of the snow load is used if the roof snow load exceeds 30 psf (1,44 kN/m²). This additional load is disregarded in this calculation.

The distribution of the force to each level is calculated using equations 5.11 and 5.12. For the vertical distribution factor C_{vx} is the exponent related to the period required. Therefore the calculation of the fundamental period can be used.

The fundamental period of the structure should be determined through analysis. As an alternative to the analysis, the standard permits use of the approximate building period T_a which can be calculated through formulas. For masonry shear walls the following equations can be used: [97]

$$T_a = C_t \cdot h_n^x \quad (6.7)$$

where

C_t is a period parameter

h_n is the structural height (ft)

x is a period parameter

C_t and x are period parameters which describe the vibration of the structure depending on the material. For structures made of masonry, the value C_t is set at 0,02 and the parameter x is defined with 0,75.

If stiff diaphragms are present, the distribution of the determined story shear can be attributed in accordance with the relative rigidities of the walls inside the building. For a Wilhelminian building, the ceilings should be considered to be flexible diaphragms.

[97] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010. Chapter C12.8.2.1

6.2.2. Shear resistance of masonry

For shear resistance of the masonry the proof has to be made that the shear stress is not exceeding the allowable shear stress. To calculate shear stress, equation 5.19 is used.

Although the allowable shear stress F_v results from the allowable shear stress resisted by the masonry F_{vm} and the allowable shear stress resisted by the reinforcement F_{vs}, should only F_{vm} be considered because the walls of Wilhelminig buildings were often made without reinforcement. Therefore should be considered that plain masonry cannot transfer tension force. This is respected through a decrease of the length into the effective length.

But for the effective length, also the area of the net mortared section of the joints goes into account and can lead to a decrease. For clay masonry units which are at least 75 percent solid, the net area can be taken as equal to the gross sectional area. This can be applied for solid units such as the bricks used in Wilhelminian buildings.[98]

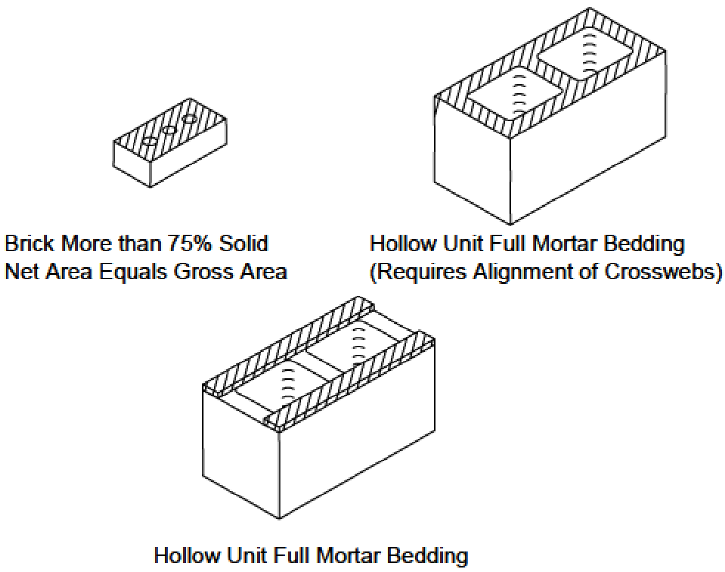


Fig. 13: Net mortared area of bricks

[98] Joint ACI/ASCE/TMS Committee: Building Code Requirements and Specification for Masonry Structures. 2011. Chapter 1.9

6.3. Indication and Results

As calculation example a fictitious building is used. The structural and material information reflect typical values of Wilhelminian Buildings in Vienna, and it will be assumed that the walls are positioned constantly in the plan to meet the criteria for regularity. The net area of the floors was estimated at 80 percent of the gross area. For simplification, the live load of the whole building will be defined as, category A1 (with regard to the Austrian code) and as residential – private rooms and corridors (with regard to the American code).

The parameters for the response acceleration S_s and S_1 are taken from the seismic ground motion maps. For the sake of comparison, it is not useful to choose an arbitrary location. Therefore an area has to be chosen where the design spectrum $S_d(T)$ is nearly the same as the design spectral response acceleration S_{DS}. With this in mind, the city Florence in the state of Arizona has been chosen because the seismic intensity of this area is approximately comparable with the seismic hazard of Vienna.

In America unreinforced masonry structures are limited to the soil class A and B. Therefore both calculations in this example use the soil class B.

The seismic force for the building should be determined in the calculation with particular attention to the behavior of a masonry wall. Therefore the resistance of one of the end walls and the effects of distributed force upon the wall should be calculated. The results can be found in the summary of table 2. Should it be necessary to review the detailed course of action, the whole calculation can be found in the annex.

Note: America uses a different system of measurement than Europe. The length is specified with inches and feet, while weight is specified in pounds. Not all of the equations are compatible for international system units and it is not obvious if all parameter and coefficients are independent of the dimension used within the equation. For this reason, the data of the example will be converted into the normal used units to avoid faults through wrong equation inputs. To facilitate comparison, the intermediary results and the final outcome will be converted into metric units.

	Result through Austrian Standard	**Result through American Standard**
dead load of the building	15.810,00 kN	15.580,98 kN
live load of the building	432,00 kN	751,68 kN
total load of the building	16.242,00 kN	16.332,66 kN
fundamental period	0,36 sec	0,35 sec
base shear force	2.208,91 kN	1.808,12 kN pseudo lateral force 2.299,79 kN
horizontal force 3rd floor	512,19 kN	411,52 kN
horizontal force 2nd floor	848,36 kN	698,30 kN
horizontal force 1st floor	565,57 kN	465,54 kN
horizontal force ground floor	282,79 kN	232,77 kN
shear wall strength	217,26 kN	170,20 kN
shear wall action	368,15 kN	301,35 kN
performance factor required	0,25	1,00
actual performance factor	0,59	0,56
evaluation	compliant	non compliant

Tab. 2: Results of the calculation through the Austrian and American Standard

7. Comparison

By comparing the technical standards of Austria and America, it becomes clear that the methods of calculations of the seismic force are quite similar. Both countries use the same analysis steps in the course of action.

This similarity of procedure would suggest that there has already been some exchange of information between the two countries

The lateral force analysis is a common method used in the countries. This can be seen by the huge amount of information which is provided through the codes. It leads to the assumption that the method has approved and established.

Both codes make use of the design response spectrum with the difference that the American standard defines a additional parameter for the 1,0 second period. The periods themselves are defined in different ways. In Austria, the values are defined with the soil factor and in America the periods are determined by calculation through equations.

Also the allocation of the values is comparable. The intensity of an earthquake is defined by a rough partition of the country, while detailed information is obtainable through a gazetter.

After calculation, the distribution of the base shear force to the story force and the wall under consideration goes the exactly a similar way.

The effective seismic weight of the building is different because the live load goes with a different amount into account.

Some differences can be found in the detail such as in parameters and coefficients inside an equation. It can be seen that the results of the base shear force for a building with the same dimensions and nearly the same seismic effect will differ between the standards.

Due to the fact that America has higher earthquake risk, it could be expected that the calculated force arrived at by using the American standard is higher than that of the Austrian standard because a higher risk would lead to more safety. As shown in table 2, it can be seen that the value trough EN 1998-1 exceeds the base shear force through ASCE 7-10.

This arises as a result of different input factors or the position where such factors enter into the calculation.

In an attempt to create an equal starting position for the response spectrum curve with the assumption of a similar design spectrum, the influences of the seismic force is demonstrated. For clarification, it can be said that after summarizing and canceling of all input parameters of the equation can be defined, that the seismic force is calculated on the one side with the mass, the response acceleration and the correction factor 0,85. $m * SD * 0,85$

On the other side, the seismic force can be computed with the mass, the response acceleration, the importance factor and the modification factor belonging to the material. After summarizing of the American outcome can be determined the equation $m * SD * 0,66$.

In Austria the importance factor and the behavior factor (the counterpart to the modification factor) are included as well, but they are already part of the design spectrum. As a result, the American outcome is minimized by 0,66 while the Austrian outcome is minimized by 0,85 which leads to a higher lateral force.

With this having been recognized, the presumption of equal design spectra can be seen in a critical way.

As described in the standard for existing buildings is the determined lateral force through ASCE7-10 more representative than the pseudo lateral force. Under comparison of the three results of the base shear can be seen, that the outcome of the Austrian course of action is more close to the pseudo lateral force. This could be an evidence, that the computed force would be also exceed the actual force of the design earthquake.

More significant differences between the two codes can be found in the calculation of masonry strength. Austria uses a shear strength of masonry with a defined value of a table. The dimension of the value is depending on the compressive strength of mortar. For the reason that the weakest part of masonry against shear force is the mortar joint, the compressive strength of mortar is adequate for the definition.

In America, the shear strength of masonry is calculated with the equation 5.23 or 5.24. Under inspection of the input parameters can be seen that only the compressive strength of masonry goes into account as a material parameter. Because the masonry strength is much higher than the shear strength of masonry which goes into account in the Austrian calculation, the resistance results in a much higher value and commensurate to a performance factor of 3,42. If the result would be based on the mortar properties as well, the resistance would arise as 8,70 psi and then would the shear stress in the masonry with 10,55 also exceed the allowed shear stress with a performance factor of 0,82.

Little similarity of approach can be found in the formulas through the calculation based on existing buildings. There goes the mortar shear strength into account. With the calculation of the Tier 2 phase can the resistance be determined with 170,20 kN which would be commensurate to a performance factor of 0,56.

Austria works with the safety concept. Thereby most properties are predefined as characteristic and design values which differ through the integration of a partial factor. This kind of security cannot be found in the American standard. It could be supposed that the fixed value of 0,56 in the equation 5.17 of the expected unreinforced masonry strength v_{me} works like a partial factor but an exact explanation of the duty or the defined high could not be found.

The unequal results through the calculations of the different sources are giving cause of concern. All courses are permitted in America but do not offer the same result.

Plain masonry has poorer material properties than other material such as concrete or steel. Better results can be scored if a combination can be used in the structure for instance reinforced masonry. As the wall is considered to be unreinforced masonry, such as can be found in Wilhelminian buildings, some results do not achieve the required specifications of a newly built structure. Only through application of the Austrian defined performance factor can the wall be evaluated as compliant. In additional should be said, that in most cases the end wall is the wall with the highest resistance because it normally shows the longest length and has no openings. The walls inside the building cannot feature resistance as a high as an end wall.

8. Conclusion

A main difference is the build-on structure of the used codes in a country.

While Austria uses a general standard for specified fields, in America a huge variety of codes is available. Through the free choice of the applied standard to an evaluation or calculation, it exist a competition between the different institutions. This leads to different course of action through different technical standards with may not offer the same result. The question arises if it would be more useful to generate a common level of standard through an oversight organization such as in Europe with the European Committee for Standardization.

In spite of the availability of many codes covering the field of masonry structures, the amount of information belonging to unreinforced masonry is limited. Due to the fact that new structures made of unreinforced masonry is strictly limited to specific soil classes and American codes uses an active program for seismic evaluation and retrofitting, the number of existing buildings with the same conditions like Wilhelminian buildings is low.

In contrast, Europe contains a vast number of masonry buildings. This prevalence has resulted in a broad knowledgebase through the years. The mass of information does not give a guarantee for better quality, but the American standards for unreinforced masonry dealing with facile requirements and do not deal with the deeper connection of the properties. As it could be seen in the comparison of chapter 7 do different ways not lead to the same result. In this can be seen a failure through the codes. As a result the conclusion should be drawn that the European and Austrian standards give better guidelines through calculation and evaluation of masonry.

The conclusion can be drawn that the two countries are on about the same level of knowledge for the seismic requirements.

Nonetheless, some disparities between the codes can be found, for instance as described in Chapter 7, the American standard uses two values S_{DS} and S_{D1} to define the design response spectrum. This cannot, however be determined a disadvantage.

The pseudo lateral force procedure could be interesting for adapting to the Austrian standard. If the equivalent lateral force is similar to the Austrian lateral force method it could mean that the pseudo lateral force procedure would also create a more actual force to a design earthquake.

The differences which arise through different seismic hazard such as the reference return period are chosen for a reasonable case. Adopting such requirements would not be useful in Austria because the country has not to handle with this intense of earthquake.

The basic concept of a performance factor identical to that of a new building is comprehensible, but it is arguable if this requirement is reasonable. Progressive improvement of the standards would lead inevitably to a fast obsolescence of the specifications of a given structure. If a structure had always to comply with the newest standard, it would lead to a constant need to retrofit. A certain tolerance should be conceded. However, it should not be claimed that the given performance factor of 0,25 from the Austrian standard is the right assumption.

Although no huge differences have been arisen through the comparison of the technical standard can the small differences still animate to think about it. Maybe the differences are not decisive enough to arrange changes in the standard.

But it does not suppose that the countries cannot learn from each other. America is continuously researching in all kind of fields such as material properties and offers active avenue for further development. It may be well the topic of unreinforced masonry is antediluvian and not current enough for comparison with the Austrian standards.

Annex

Calculation:

Building Information:

Length l:	20,00 m	65,62 ft
Width br:	15,00 m	49,21 ft
Stories:	4	4
Story high h_i:	3,5 m	11,48 ft
Gross floor area A_{brutto} = l*br:	300,00 m²	3.229,09 ft²
Net floor area $A_{netto} = A_{brutto}$ * 0,8	240,00 m²	2.583,28 ft²
Wall area $A_{wall} = A_{brutto} – A_{netto}$	60,00 m²	645,82 ft²
Dead load ceiling dl_c:	4,00 kN/m²	83,54 psf
Dead load roof ceiling dl_r:	1,50 kN/m²	31,33 psf
Shear wall thickNess b_{rw}:	0,30 m	0,98 ft
Specific weight of masonry ɣ:	16,00 kN/m³	99,88 lbf/ft³
Live load L_o:	2,00 kN/m³	40,00 psf
Live load reduction L:	0,60 kN/m²	21,80 psf

Austria

4.1.6 $\psi_{Ei} = \psi_{2,I} = 0,3$ see Tab A1

$\Sigma\psi_{E,i}\, Q_{k,i} = 0,3 * 2,00 = 0,60$ kN/m²

America

(5.3) $L = L_o \left(0,25 + \frac{x}{\sqrt{K_{LL} \cdot A_T}}\right)$ = 40,00+(0,25+(15 / (√(1*2.583,28))) = 21,80 psf

L > 0,5 Lo = 21,80 > 20,00 compliant

Soil class:	class B	class B

Calculation Austria:

Total weight 1.624.200,00 kg

Dead load

3rd floor: A_{brutto} *dl_r + A_{wall}* ɣ*h_l/2 =300*1,50 + 60 *16* 3,5/2 = 2.130,00 kN

2nd floor: A_{brutto} *dl_c + A_{wall}* ɣ* h_l =300*4,00 + 60 *16* 3,5 = 4.560,00 kN

1st floor: A_{brutto} *dl_c + A_{wall}* ɣ* h_l =300*4,00 + 60 *16* 3,5 = 4.560,00 kN

ground floor: A_{brutto} *dl_c + A_{wall}* ɣ* h_l =300*4,00 + 60 *16* 3,5 = 4.560,00 kN

Live load

3rd floor: no live load

2nd floor: A_{netto} *L =240*0,6 = 144,00 kN

1st floor: A_{netto} *L =240*0,6 = 144,00 kN

ground floor: A_{netto} *L =240*0,6 = 144,00 kN

Total = 16.242,00 kN = 1.624.200,00 kg

compressive strength of mortar fm: 1,00 N/mm²

compressive strength of masonry unit fb: 15,00 N/mm²

char. compressive strength of masonry fk: 2,79 N/mm²

Reduction for joints parallel to the wall (masonry bond) 0,80

K = 0,60 see Tab A2

α = 0,65 see Tab A2

β = 0,25 see Tab A2

(3.1) $f_k = K \cdot f_b{}^{\alpha} \cdot f_m{}^{\beta}$ = 0,80 * 0,60 *15^0,65 * 1,00^0,25 = 2,79 N/mm²

Fundamental Period: 0,36 sec

(6.2) $T_1 = C_t \cdot H^{3/4}$ = 0,050* (4 * 3,50)^3/4 = 0,36 sec

T_c = 0,4 see Tab A3

(6.1) $T_1 \leq \begin{cases} 4 \cdot T_c \\ 2{,}0\ s \end{cases}$ = 4 * 0,4 = 1,6 sec and 2,0 sec compliant

Reference peak ground acceleration 0,8 m/s²

a_{gR} = 0,8 m/s² see Tab A4

Zone 3 see Tab A4

importance category 2 for residential buildings

importance factor γ_I = 1,00 see Tab A5

$a_g = \gamma_I * a_{gR}$ = 1,00 * 0,8 = 0,8 m/s²

Design spectrum $S_d(T)$ 1,6 m/s²

S = 1,20 see Tab A3

q = 1,50 if fm < 2,5 N/mm²

(6.3) $S_d(T_1) = a_g \cdot S \cdot \frac{2{,}5}{q}$ = 0,8 *1,2 * 2,5/1,5 = 1,6 m/s²

Base shear force 2.208,91 kN

λ = 0,85 for buildings with more than 3 stories

(6.4) $F_b = S_d(T_1) \cdot m \cdot \lambda$ = 1,6 * 1.624.200,00 * 0,85 = 2.208.912,00 N

Horizontal force distribution in each storey

(6.5) $F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j}$

Ceiling over	mi [kN]	zi [m]	mi*zi	Fi [KN]
3rd floor	2.130,00	14,00	29.820,00	512,19
2nd floor	4.704,00	10,50	49.392,00	848,36
1st floor	4.704,00	7,00	32.928,00	565,57
ground floor	4.704,00	3,50	16.464,00	282,79
∑ mi * zi			128.604,00	

Horizontal force distribution to end wall

influence area i= 16,67 %

Ceiling over	V [kN] Fi * area	zi [m]	Md [kNm] V * zi	Nd [kN] l * h* ɣ
3rd floor	85,37	14,00	1.195,11	1.008,00
2nd floor	141,39	10,50	1.484,63	756,00
1st floor	94,26	7,00	659,84	504,00
ground floor	47,13	3,50	164,96	252,00
∑ =	368,15		3.504,54	2.520,00

Eccentricity of force

e= M/N = 3.504,54 / 1.008,00 = 3,48 m

l/6 = 15,00 / 6,00 = 2,5 m > e not compliant

l/3 = 15,00 / 3,00 = 5,00 > e complaint

l_o = ((l/2) - e) * 3 = ((15,00/2) -3,48) *3=12,07 m

Shear resistance of the wall

f_{vko} = 0,10 N/mm² see Tab A6

γ_M = 2,50 * 2/3 = 1,67 see Tab A7

$f_{vd} = f_{vko} / \gamma_M$ = 0,10 / 1,67 = 0,06 N/mm² = 60,00 KN/m²

(3.5) $V_{Rd} = f_{vd} \cdot t \cdot l_o$ = 60,00 * 0,30 * 12,07 = 217,26 KN

$V_{Ed} \leq V_{Rd}$ = 368,15 ≤ 217,26 not compliant

Performance factor

$\alpha = V_{Rd} / V_{Ed}$ = 217,26 / 368,15 = 0,59

α_{min} = 0,25 see Tab A8

$\alpha \geq \alpha_{min}$ = 0,59 ≥ 0,25 compliant

Tables:

Tabelle A.1.1 — Empfehlungen für Zahlenwerte für Kombinationsbeiwerte im Hochbau

Einwirkung	ψ_0	ψ_1	ψ_2
Nutzlasten im Hochbau (siehe EN 1991-1-1)			
Kategorie A: Wohngebäude	0,7	0,5	0,3
Kategorie B: Bürogebäude	0,7	0,5	0,3

Tab. A1: Cutout of Table A.1.1 from EN 1990

Tabelle 2 — Beiwerte K und Exponenten α, β zur Ermittlung der Druckfestigkeit von Mauerwerk

Mauersteinart		Normalmörtel			Dünnbettmörtel[a] Dicke 1 mm bis 3 mm			Leichtmörtel mit einer Rohdichte 600 kg/m³ bis 800 kg/m³			Leichtmörtel mit einer Rohdichte über 800 kg/m³ bis 1500 kg/m³		
		K	α	β	K	α	β	K	α	β	K	a	β
Ziegel	Gruppe 1	0,60	0,65	0,25	0,90	0,70	0,00	0,35	0,65	0,25	0,50	0,65	0,25
	Gruppe 2	0,55	0,65	0,25	0,70	0,70	0,00	0,30	0,65	0,25	0,40	0,65	0,25
	Gruppe 3	0,50	0,65	0,25	0,50	0,70	0,00	0,25	0,65	0,25	0,30	0,65	0,25
Beton	Gruppe 1	0,60	0,65	0,25	0,75	0,85	0,00	0,50	0,65	0,25	0,55	0,65	0,25
	Gruppe 2	0,55	0,65	0,25	0,70	0,85	0,00	0,45	0,65	0,25	0,50	0,65	0,25
	Gruppe 3	0,50	0,65	0,25	0,60	0,85	0,00	–[b]	0,65	0,25	–[b]	0,65	0,25
Porenbeton	Gruppe 1	0,60	0,65	0,25	0,75	0,85	0,00	0,50	0,65	0,25	0,55	0,65	0,25

[a] Mörteldruckfestigkeit $f_m \geq 10$ N/mm^2

[b] Es liegen keine gesicherten Prüfungsdaten vor, die Druckfestigkeit von Mauerwerk muss mittels Prüfungen ermittelt werden.

Tab. A2: Table 2 from B 1996-1-1

Tabelle 3.2 — Parameterwerte zur Beschreibung der empfohlenen elastischen Antwortspektren vom Typ 1

Baugrundklasse	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Tab. A3: Table 3.2 from EN 1998-1

Tabelle A.1 (fortgesetzt)

Ort	Zonen-gruppe	a_{gR} m/s^2		Ort	Zonen-gruppe	a_{gR} m/s^2
Wien						
Wien südwestl. d. Donau	3	0,80		Wien nordöstl. d. Donau	2	0,70

Tab. A4: Cutout of Table A.1 from B 1998-1

Tabelle 1 — Festlegung der γ_{I}-Werte

Zonengruppe	Bedeutungskategorie γ_{I}			
	I	II	III	IV
0	0,8	1,0	1,0	1,0
1	0,8	1,0	1,0	1,0
2	0,8	1,0	1,1	1,2
3	0,8	1,0	1,2	1,4
4	0,8	1,0	1,4	1,4

Tab. A5: Cutout of Table 1 from B 1998-1

Tabelle 3 — Werte für die Anfangsscherfestigkeit (Haftscherfestigkeit) f_{vk0} von Mauerwerk

<table>
<tr><th>Mauersteinart</th><th colspan="2">Normalmörtel mit einer Festigkeitsklasse[a]</th><th>Dünnbettmörtel (Lagerfugendicke 0,5 mm bis 3 mm)</th><th>Leichtmörtel</th></tr>
<tr><td rowspan="3">Ziegel</td><td>≥ M10</td><td>0,30</td><td rowspan="3">0,30</td><td rowspan="11">0,15</td></tr>
<tr><td>M2</td><td>0,20</td></tr>
<tr><td>M1</td><td>0,10</td></tr>
<tr><td rowspan="3">Kalksandstein</td><td>≥ M10</td><td>0,20</td><td rowspan="3">0,40</td></tr>
<tr><td>M2</td><td>0,15</td></tr>
<tr><td>M1</td><td>0,10</td></tr>
<tr><td rowspan="3">Beton</td><td>≥ M10</td><td>0,20</td><td rowspan="5">0,30</td></tr>
<tr><td>M2</td><td>0,15</td></tr>
<tr><td>M1</td><td>0,10</td></tr>
<tr><td>Porenbeton</td><td>M2 bis M10</td><td>0,15</td></tr>
<tr><td>maßgerechter Naturstein</td><td>M1 bis M2</td><td>0,10</td></tr>
<tr><td colspan="5">[a] Zwischenwerte für Mörtelfestigkeiten von M1 bis M10 sind linear zu interpolieren. Für Mörtelfestigkeiten unter M1 ist der Ansatz einer Anfangsscherfestigkeit nicht zulässig.</td></tr>
</table>

Tab. A6: Table 3 from B 1996-1-1

Tabelle 1 — Teilsicherheitsbeiwerte für das Material

Material	γ_M
Mauerwerk aus:	
Steinen der Kategorie I und Mörtel nach Eignungsprüfung[a]	2,00
Steinen der Kategorie I und Rezeptmörtel[b]	2,20
Steinen der Kategorie II[a,b,c]	2,50
Verankerung von Bewehrungsstahl	2,20
Bewehrungsstahl und Spannstahl	1,15
Ergänzungsbauteile[d,e]	2,20
Mauerwerksstürze nach ÖNORM EN 845-2 mit tragender Übermauerung[f]	2,20

[a] Anforderungen an Mörtel nach Eignungsprüfung sind in den ÖNORMEN EN 998-2 und EN 1996-2 angegeben.
[b] Anforderungen an Rezeptmörtel sind in den ÖNORMEN EN 998-2 und EN 1996-2 angegeben.
[c] Sofern der Variationskoeffizient der Steine nach Kategorie II nicht größer als 25 % ist.
[d] Abdichtungen gegen Feuchtigkeit sind ebenfalls mit γ_M abgedeckt.
[e] Deklarierte Werte sind Mittelwerte.
[f] Stürze ohne tragende Übermauerung sind nach den entsprechenden Konstruktionsnormen zu bemessen.

Tab. A7: Table 1 from B 1996-1-1

Tabelle A.3 — Mindesterdbebenerfüllungsfaktoren α_{min}

Schadensfolgeklasse / Versagensfolgeklasse	Erdbebenerfüllungsfaktor α_{min}
RC1 – CC1	0,09
RC2 – CC2	0,25
RC3 – CC3	0,85

Tab. A8: Table A.3 from B 1998-3

Calculation America:

Total weight 1.633.266,00 kg

Dead load

3rd floor: A_{brutto} *dl_r + A_{wall}* ɣ*h_l/2 =3.229,09*31,33 + 645,82 *99,88* 11,48/2 = 471,49 kips

2nd floor: A_{brutto} *dl_c + A_{wall}* ɣ* h_l =3.229,09*83,54 + 645,82 *99,88* 11,48 = 1.010,42 kips

1st floor: A_{brutto} *dl_c + A_{wall}* ɣ* h_l =3.229,09*83,54 + 645,82 *99,88* 11,48 = 1.010,42 kips

ground floor: A_{brutto} *dl_c + A_{wall}* ɣ* h_l =3.229,09*83,54 + 645,82 *99,88* 11,48 = 1.010,42 kips

Live load

3rd floor: no live load

2nd floor: A_{netto} *L = 2.583,28*21,80 = 56,33 kips

1st floor: A_{netto} *L =2.583,28*21,80 = 56,33 kips

ground floor: A_{netto} *L =2.583,28*21,80 = 56,33 kips

Total = 3.671,73 kips = 3.600.735,05 lb = 1.633.266,00 kg

response spectral acceleration for short period S_s = 0,254

response spectral acceleration for 1,0 second period S_1 = 0,076

site coefficient F_a = 1,00 see Tab B2

site coefficient F_v = 1,00 see Tab B1

Design response spectral acceleration for short period 0,169

S_{DS} = 2/3 *F_a * S_s = (2/3) * 1 * 0,254 = 0,169

Design response spectral acceleration for 1,0 second period 0,051

S_{D1} = 2/3 *F_v *S_1 = (2/3) * 1 * 0,076 = 0,051

Fundamental Period: 0,35 sec

C_t = 0,02 for other structures like masonry

x = ¾ for other structures like masonry

(6.7) $T_a = C_t \cdot h^x$ = 0,02* (4 * 11,48)^3/4 = 0,35 sec

Equivalent lateral force 1.808,13 kN

risk category = II

response modification factor R = 1,50 see Tab B4

Importance factor I_e = 1,00 see Tab B3

(5.10) $C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$ = 0,169 / (1,50/1,00) = 0,11288

(5.9) $V = C_s \cdot W$ = 0,11288 * 3.600.735,05 = 406.482,98 lbf = 1.808.125,72 N

Pseudo lateral force 2.299,79 kN

C = 1,00 see Tab B5

S_a = S_{D1}/T = 0,051 / 0,35 = 0,14

(5.14) $V = C \cdot S_a \cdot W$ =1,00*0,14*3.600.735,05=517.014,42 lbf=2.299.793,91 N

Horizontal force distribution in each storey

(5.11) $F_x = C_{vx} \cdot V$

(5.12) $C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k}$

Ceiling over	wx [kips]	hx [ft]	wx*hx	C_{vx}	Fx [lbf]
3rd floor	462.372,33	45,93	21.237.316,04	0,23	92.513,67
2nd floor	1.046.120,91	34,45	36.037.191,47	0,39	156.984,66
1st floor	1.046.120,91	22,97	24.024.794,31	0,26	104.656,44
ground floor	1.046.120,91	11,48	12.012.397,16	0,13	52.328,22
∑ wx*hx			93.311.698,97		

Horizontal force distribution to end wall 301,35 KN

influence area i= 16,67 %

Ceiling over	V [lbf] Fx * area	hx [ft]	M [ft-lbf] Fx*hx	P [lbf]
3rd floor	15.418,94	45,93	708.210,63	55.549,07
2nd floor	26.164,11	34,45	901.311,70	55.549,07
1st floor	17.442,74	22,97	400.582,98	55.549,07
ground floor	8.721,37	11,48	100.145,74	55.549,07
∑ =	67.747,16		2.110.251,06	222.196,28

67.747,16 lbf = 301,35 KN

Eccentricity of force

e= M/P = 2.110.251,06 / 222.196,28 = 9,50 ft

l/6 = 49,21 / 6,00 = 8,20 m > e not compliant

l/3 = 49,21 / 3,00 = 16,40 > e complaint

d = ((l/2) - e) * 3 = ((49,21/2) -9,50) *3 = 45,33 ft

a = M/ Fx = 2.110.251,06 / 67.747,16 = 31,15 ft

M/Vd = a/d = 31,15 / 45,33 = 0,69

Shear resistance of the wall 708,99 kN

compressive strength of masonry f'm: 2,79 N/mm² = 404,75 psi

shear stress in masonry:

(5.19) $f_v = \frac{V}{A_{nv}}$ = 67.747,16 / (45,33*12 * 0,98*12) = 10,55 psi

allowable shear stress resisted by the masonry:

$F_v = F_{vm}$

(Interpolation between 5.21 and 5.22) $Fv < 2{,}41\sqrt{f'_m}$ = 48,55 psi

(5.23) $F_{vm} = \frac{1}{2} \cdot \left[\left(4{,}0 - 1{,}75 \cdot \left(\frac{M}{V_d}\right)\right) \cdot \sqrt{f'_m}\right] + 0{,}25 \cdot \frac{P}{A_n} =$

0,5*((4,0-1,75*0,69)*√404,75)+0,25*222.196,28/(45,33*12*0,98*12) = 36,10 psi

(For the whole wall 159,39 kips = 708,99 kN)

$f_v < F_{vm}$ =10,55 < 36,10 compliant

Performance factor

$\alpha = F_{vm} / f_v$ = 36,10 / 10,55 = 3,42 compliant

Tables:

Table 3-5. Values of F_v as a Function of Site Class and Mapped Spectral Acceleration at a One Second Period, S_1

Site Class	Mapped Spectral Acceleration at One-Second Period[1]				
	$S_1 < 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 > 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*	*	*	*	*

[1]Note: Use straight-line interpolation for intermediate values of S_1.

* Site-specific geotechnical investigation and dynamic site response analyses required.

Tab. B1: Table 3-5 from Seismic Evaluation of Existing Buildings

Table 3-6. Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration, S_s

Site Class	Mapped Spectral Acceleration at Short Periods[1]				
	$S_s < 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.00$	$S_s > 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

[1]Note: Use straight-line interpolation for intermediate values of S_s.

* Site-specific geotechnical investigation and dynamic site response analyses required.

Tab. B2: Table 3-6 from Seismic Evaluation of Existing Buildings

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads[a]

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_i	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

[a]The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it is dependent on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

Tab. B3: Table 1.5-2 from ASCE7-10

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R[a]	Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] – Seismic Design Category B	C	D[d]	E[d]	F[e]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[l, m]	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls[l]	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls[l]	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls[l]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls[l]	14.2	4	2½	4	NL	NL	40[k]	40[k]	40[k]
6. Ordinary precast shear walls[l]	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP

Tab. B4: cutout of Table 12.2-1 from ASCE7-10

Table 3-4. Modification Factor, *C*

Building Type[1]	Number of Stories: 1	2	3	=4
Wood (W1, W1A, W2) Moment Frame (S1, S3, C1, PC2A)	1.3	1.1	1.0	1.0
Shear Wall (S4, S5, C2, C3, PC1A, PC2, RM2, URMA) Braced Frame (S2)	1.4	1.2	1.1	1.0
Unreinforced Masonry (URM) Flexible Diaphragms (S1A, S2A, S5A, C2A, C3A, PC1, RM1)	1.0	1.0	1.0	1.0

[1]Defined in Table 2-2.

Tab. B5: Table 3-4 from Seismic Evaluation of Existing Buildings

Bibliography

[1] American Society of Civil Engineers: Seismic Evaluation of Existing Buildings. 01.01.2003.

[2] American Society of Civil Engineers: Minimum Design Loads for Buildings and other Structures, ASCE 7-10. 12.05.2010.

[3] American Society of Civil Engineers: Guidelines for Seismic Evaluation and Design of Petrochemical Facilities. 31.01.2011.

[4] Hollinsky, Karlheinz: Kapitel 01 – Bauweisen von Gründerzeithäusern, FH Campus Wien, Skriptum. WS 2011/2012

[5] Hollinsky, Karlheinz: Kapitel 04 – Wände im Bestand-Wandertüchtigung, FH Campus Wien, Skriptum. WS 2011/2012

[6] Joint ACI/ASCE/TMS Committee: Building Code Requirements and Specification for Masonry Structures. 2011.

[7] Österreichisches Normungsinstitut: ÖNORM EN 1990. Grundlagen der Tragwerksplanung. 1.3.2003.

[8] Österreichisches Normungsinstitut: ÖNORM EN 1996-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 01.03.2006.

[9] Österreichisches Normungsinstitut: ÖNORM EN 1996-3. Vereinfachte Berechnungsmethoden für unbewehrte Mauerwerksbauten. 01.07.2006.

[10] Österreichisches Normungsinstitut: ÖNORM B 1991-1-1. Bemessung und Konstruktion von Mauerwerksbauten. 1.3.2009

[11] Österreichisches Normungsinstitut: ÖNORM EN 1996-2. Bemessung und Konstruktion von Mauerwerksbauten. 15.11.2009.

[12] Österreichisches Normungsinstitut: ÖNORM EN 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011.

[13] Österreichisches Normungsinstitut: ÖNORM B 1998-1. Grundlagen, Erdbebeneinwirkung und Regeln für Hochbauten. 15.06.2011.

[14] Österreichisches Normungsinstitut: ÖNORM B 1998-3. Beurteilung und Ertüchtigung von Gebäuden. 01.01.2013.

[15] Zentralanstalt für Meteorologie und Geodynamik: Übersicht. In: http://www.zamg.ac.at/cms/de/geophysik/erdbeben/erdbeben-in-oesterreich/uebersicht_neu (last access: 3.7.2013)

List of figures

List of tabes