DIPLOMARBEIT

Seismic Design of Industrial Buildings

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Abstract

Industrial construction is characterised by capital-intensive equipment. Earthquakes could badly damage this equipment, which would result in high losses due to facility downtime. Therefore, a main focus of seismic engineering for industrial construction should be to protect this equipment and provide operability after an earthquake.

Seismological basics, provisions for the preliminary design and different methods of structural response analyses are explained in the first chapters of this thesis.

Alternatively to the prescriptive seismic design codes, the performance-based design approach is introduced afterwards. This approach allows the owner of an industrial facility to set higher performance objectives and to apply flexible design solutions in order to reduce the losses.

The thesis concludes with a comparison of earthquake-risk and the prescriptive seismic design codes of three different countries.

Keywords: seismic design, industrial buildings, performance-based design, nonstructural components, California, Chile, Austria

Kurzzusammenfassung

Ein großer Kostenfaktor im Industriebau sind die mechanischen und elektrischen Anlagen und weitere zahlreiche, nichttragende Bauteile, welche für den Produktionsprozess gebraucht werden. Wenn diese Bauteile und Anlagen durch ein Erdbeben beschädigt werden entstehen hohe Reparaturkosten und zum Teil immense Verluste aufgrund des notgedrungenen Stillstands des Werkes. Aus diesem Grunde sollte bei der Bemessung einer Industrieanlage ein besonderes Augenmerk auf den Schutz solcher kostenintensiven Komponenten gelegt werden.

Zu Beginn dieser Arbeit wird auf die Entstehung von Erdbeben und auf die Grundlagen der Erdbebenbemessung eingegangen. Wesentliche Überlegungen und Grundsätze für den Tragwerksentwurf werden aufgezeigt und verschiedene Methoden zur Berechnung des Tragverhaltens unter seismischer Belastung, beschrieben.

Im Weiteren wird eine Alternative zum herkömmlichen normenbasierten Bemessungsansatz vorgestellt – das „Performance-Based Design“. Durch das Abweichen von vorgeschriebenen Bemessungsprozeduren bestehender Normen ermöglicht dieser Ansatz dem Betreiber einer Anlage eigene - höhere - Anforderungen an das Tragverhalten seiner Anlage zu stellen und somit höheren Reparaturkosten vorzubeugen.

Zu guter Letzt wird das Erdbebenrisiko und die Normenkonstrukte dreier Länder untersucht und gegenübergestellt.

Stichwörter: Erdbebenbemessung, Industriebau, Leistungsorientierte-bautechnische Vorschriften, nichttragende Bauteile, Kalifornien, Chile, Österreich

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Preface

Industrial construction is characterised by capital-intensive equipment and numerous interfaces of the structural, mechanical and electrical components. Earthquakes could badly damage these components and equipment, which would result in direct (reparation costs due to the damage) and indirect (economic deficit due to facility downtime) losses.

Therefore, a main focus of seismic engineering for industrial construction should be to protect these components and equipment and provide operability after an earthquake.

Most of the existing prescriptive codes for seismic engineering pursue a performance objective, which focuses on the protection of human lives, avoiding a total collapse and limit the damage such that repair costs are not disproportionately high compared to the construction costs itself.

This objective does often not suffice the requirements of the owner of the facility. With the aim to reduce the losses in case of a seismic event, higher performance objectives, which allow only minor damages to provide post-earthquake operability are requested.

This could be achieved with the performance-based design approach, which allows deviation from prescriptive codes and makes way for individual, innovative and flexible design solutions.

The performance-based seismic design approach, with respect to its applicability for industrial construction, is investigated in this thesis.

Furthermore seismological basis, important preliminary design decisions and the specific characteristics for the seismic design of the industrial construction are explained.

The international aspect behind this thesis was my exchange quarter at California Polytechnic State University. As California is a highly seismic region, studying there and working on this topic was an obvious choice.

1 Earthquakes

1.1 Origin of Earthquakes

The thickness of the Earth's crust is varying between 10 km (oceanic crust) and 70 km (the Alps) *[see Figure 1]*. This thin layer (in proportion to the whole earth-radius) is rigid and brittle and swims on a viscous-plastic subsurface. Due to geothermal flows the Earth's crust is continuously moving. These movements cause a continuous change of the Earth's stress state. If these tensions exceed the breaking strength of the crust, a sudden rupture with powerful shakings and ground-displacements will happen; often in inhomogeneous areas which have been damaged by former movements, like in between tectonic plates or faults[1].

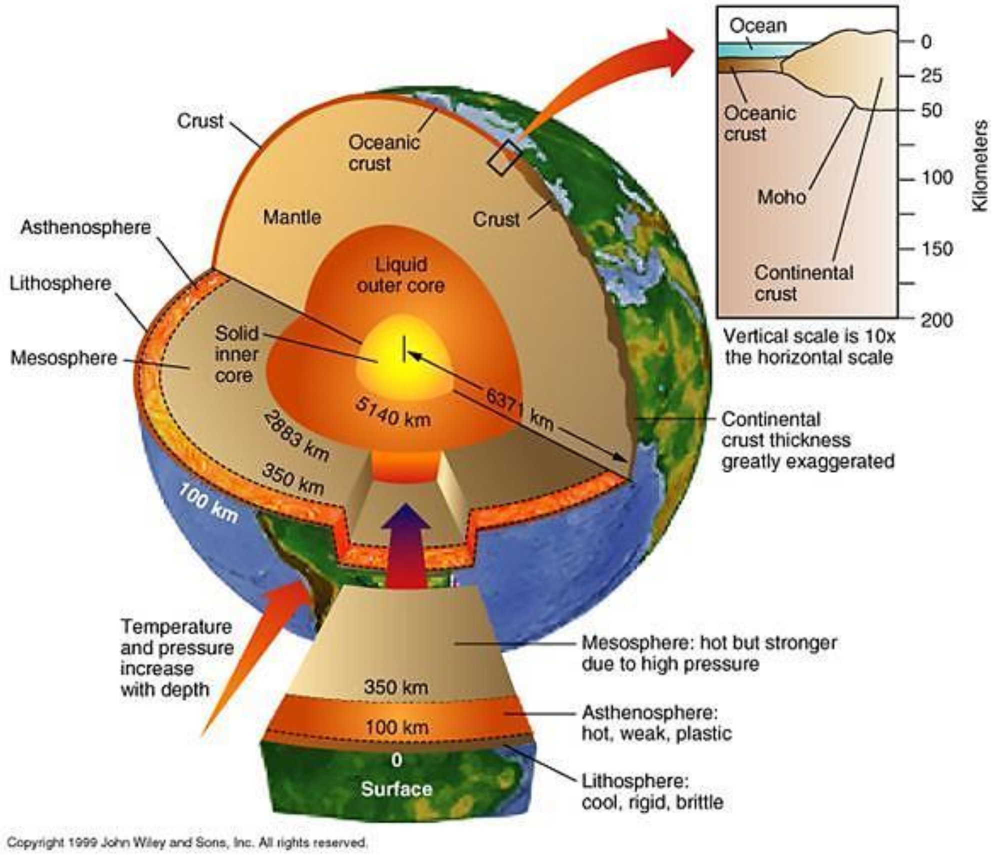

Figure 1 -– Earth's Layers[2]

[1] cf. Bachmann, Hugo: Erdbebensicherung von Bauwerken. Basel: Birkhäuser 2002, p. 9

[2] http://www.ucl.ac.uk/EarthSci/people/lidunka/GEOL2014/Geophysics7%20%20Deep%20Earth/Earth%20Structure_files/image179.jpg

1.2 Effects of Earthquakes[3]

The aim of this section is to illustrate the consequences of earthquakes. The primary effects of an earthquake – the shaking and ground rupture – cause loss of life or major property damages. People are also killed or properties are damaged by the natural after-effects or the secondary effects, following the primary effects.

The various ways of how an earthquake can affect people and environment are described as follows:

1.2.1 Primary Effects

Earthquakes generate seismic waves, especially surface waves, which cause the shaking of the ground. The intensity of ground shaking depends on local geological conditions, size of the earthquake (intensity and duration) and the distance from the epicenter.

Ground rupture is a visible breaking or displacement in the Earth's crust. It generally occurs only along the fault zone that moves during the earthquake.

The shaking and the ground rupture are the primary effects accompanying an earthquake.

1.2.2 Natural After-Effects

The shaking and ground rupture can cause the following natural after-effects:

1.2.2.1 Tsunami

Earthquakes with their hypocenter on the seabed can trigger water movements, which are hardly noticeable at the place of origin. But these movements can travel thousands of kilometres towards a coast and form massive waves as they hit the shallow sea level depths near the coast.

This after effect could cause damages on the other side of the ocean, hours after the earthquake occurred offshore.

Figure 2 – landslide 2001 in El Salvador[4]

1.2.2.2 Landslides and Avalanches

In mountainous regions ground shaking may trigger rock and debris falls, slides or avalanches. *[see Figure 2]*

1.2.2.3 Soil Liquefaction

Certain sandy or silty soils have in general a good load-bearing capacity, despite their often large water content. Soil liquefaction occurs when sandy or silty soils act like a liquid due to shaking and vibrating. It starts acting like a liquid and cause rigid structures to sink or even fall over. *[see Figure 3]*

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[3] cf. *http://www.tulane.edu/~sanelson/Natural_Disasters/eqhazards%26risks.htm*: Earthquake Hazards and Risks. Nelson, Stephen. [09.09.2014]

[4] *http://landslides.usgs.gov/learn/photos/international/various_international_landslides/elsalvadorslide.jpg*

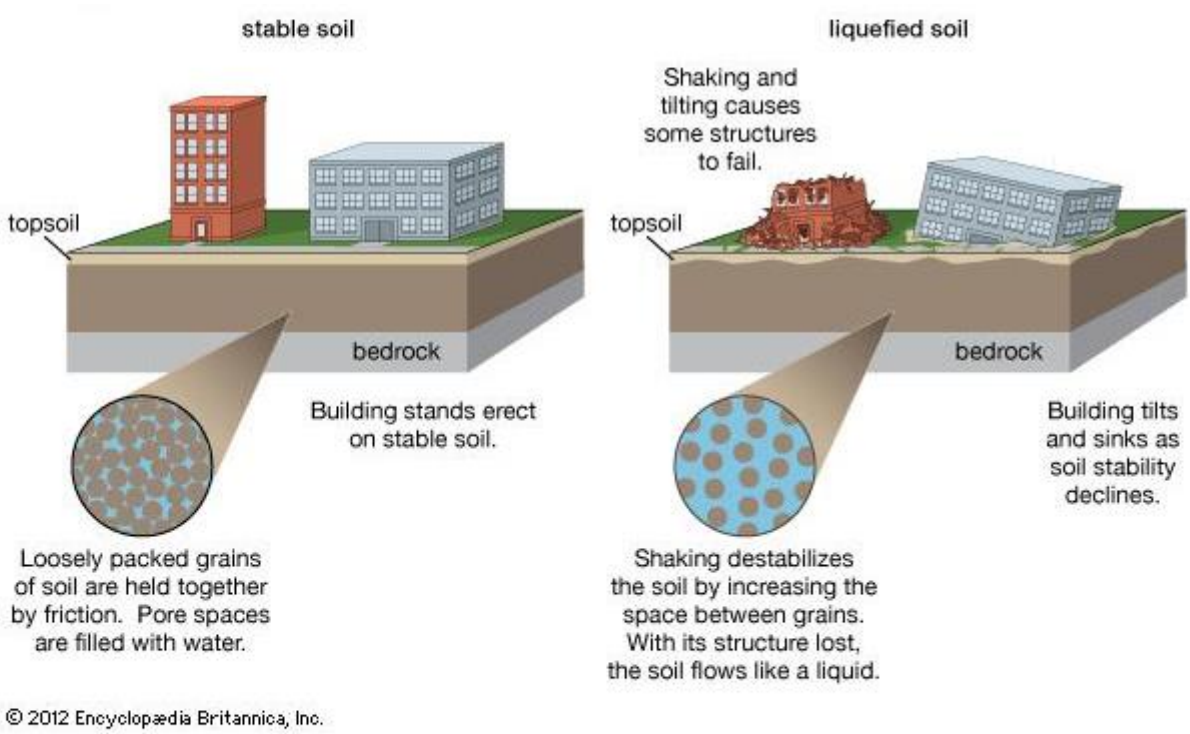

Figure 3 – Soil Liquefaction[5]

1.2.3 Secondary Effects

Secondary-effects like fire, explosions and environmental damages are caused by the facility collapse or damage, as a consequence of the primary or the natural after-effects. The impacts of the after-effect-damages are often worse, economically as well as ecologically, than the primary-effect-damage-consequences[6].

1.2.3.1 Fire

Electric or gas power lines, located below the ground, are often damaged due to ground rupture or structural damages, which can initiate a fire as consequence.

The problem becomes critical if water lines are broken too, since there will be no water supply to extinguish the fire once they have started. In recent years a big percentage of the damages to buildings were caused by fire.

1.2.3.2 Flooding

If human made dams or levees become ruptured due to tsunamis, floods may occur and often affect a huge geographical span and the people living or working there.

1.2.3.3 Environmental Damages

Damages to industrial plants often result in badly polluting or contaminating the environment and people around it.

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[5] http://www.britannica.com/science/soil-liquefaction
[6] cf. Bachmann: Erdbebensicherung von Bauwerken., p. 8

1.3 Risk vs. Hazard[7]

This section defines the terms risk and hazard.

The hazard is defined as the probability of a certain event occurring in allocated areas.

In seismic engineering the earthquake hazard means the probability of an earthquake occurring at the location of the construction site. If the affected area is located across a fault zone or on convergent or diverging plate boundaries, you can expect a high earthquake hazard. The hazard is expressed with the assumed ground acceleration for each area defined in local earthquake hazard maps.

The term risk is defined as hazard (probability of an event occurring) times the potential damage or expected loss in case of this event occurring. *[see Equation 1]*

$$Risk = \ Hazard * Expected\ Damage$$

Equation 1 – formula of risk

In seismic engineering the risk is measured in terms of expected casualties (fatalities and injuries), direct economic losses (repair and replacement costs) and indirect economic losses (income lost during downtime resulting from damage).

1.4 The Task of Seismic Engineering[8]

"Earthquakes do not kill people, buildings do!"

Most deaths from earthquakes or their collateral secondary effects are caused by buildings or other human constructions falling down. Earthquakes in uninhabited areas rarely cause any deaths or major economic losses. Thus the earthquakes risk increase exponentially with the population or number of inhabitants. But not to forget, there is a factor which can be directly influenced by humans – the construction quality. The better the properties are constructed, the better the performance will be and that means less losses.

The level of the seismic hazard (the ground acceleration of the seismic event) together with the number of people and properties that are exposed to seismic hazards and how vulnerable these people and structures are to the hazard, these three factors are describing the seismic risk.

Engineers can have considerable influence on the third factor, the vulnerability of buildings, properties or facilities. The decision about the location of a new construction would influence the second factor, but there is no way to affect the first factor, the seismic hazard which is naturally given.

In order to reduce the consequences of earthquakes, engineers forced their investigations and studies to reduce the vulnerability.

[7] cf. *http://www.fema.gov/your-earthquake-risk:* Your Earthquake Risk. Official website of the Department of Homeland Security. [24.04.2015]

[8] cf. *http://www.tulane.edu/~sanelson/Natural_Disasters/eqhazards%26risks.htm*: Earthquake Hazards and Risks. Nelson, Stephen. [09.09.2014]

They managed to do that by introducing strict building codes for the design and construction of buildings and other structures and by the induction of precautionary measures for emergency preparedness after an earthquake. In the last decade a completely new design approach developed and found their way into the construction industry – the performance-based design. *[see Chapter 3: Performance-Based Design on page 42]*

Despite these achievements there are some critical points within the design process which are not precisely predictable. Each earthquake is unique and each building a prototype. The effects and reactions can be very different, even if the impulse is of similar strength. The already existing seismic design codes are developed on the base of many assumptions and that fact should be considered during any seismic design process.

The task or the contribution of an engineer is to reduce the seismic risk by reducing the seismic vulnerability of existing structures and by avoiding additional new vulnerable constructions be built. *[An example of engineers failed to save a plant from secondary effects and damages – see Figure 4]*

Figure 4 – burning pretroleum refining plant in Shiogram/Japan, after an earthquake and tsunami 03/13/2011[9]

[9] http://www.vosizneias.com/wp-content/uploads/2011/03/5-725x495.jpg

2 Seismic Design

The previous section explained the risks and hazards of earthquakes. This section gives an insight on how to limit/reduce structural damages, loss of life or injuries due to this natural hazard.

This chapter is subdivided in three sections:

I. Seismological Basics
II. Seismic Design Process
III. Two Design Approaches for Seismic Design

In the first section, the seismological basics are explained, which are required to understand the Seismic Design Process, explained in the second section. The last section explains and compares two different seismic design approaches; the common design approach based on prescriptive codes and the performance-based design approach.

2.1 Seismological Basics[10]

The seismic force is not an externally applied load like wind caused by external pressures. Instead it is a result of dynamic, cyclic motions at the base causing inertial forces by acceleration of the structure. This section should explain the basic seismic terms, characteristics and dynamic mechanism, which are necessary to determine the building's response and to understand the basics and limitations of the seismic design and assessment process.

2.1.1 Seismic Waves

The sudden displacements occurring with an earthquake generate several seismic waves (may be slow and long or short and abrupt). These waves are able to travel through the underground, and spread out over different wave paths with different speeds. That will create manifold ground motions, recorded in a seismogram at the location of the affected building. The bigger the distance is between epicenter and location, the more waves will reach the location, due to reflections.

The length of a full cycle in seconds is the period of the wave and is the inverse of the "excitation"-frequency (f_{ex}).

[10] cf. *http://www.wbdg.org/resources/seismic_design.php:* Seismic Design Principles. Gabor Lorant [15.03.2012] AND cf. Bachmann: Erdbebensicherung von Bauwerken, p. 17

2.1.2 Movement-Recording and Measurements[11]

The “time history plot” of the ground motions of incoming seismic waves at a given point can be recorded by a seismometer for minor earthquakes or an accelerometer for strong earthquakes.

A seismogram, the recorded varying amplitudes of ground oscillations, is made by a spring-mass-dashpot device named seismograph and its detecting and recording part, the seismometer. A seismometer is mounted on the ground and measures the displacement of the ground with respect to a stationary reference point. Three seismometers are needed to record all components of the ground motion, because each can record only in one orthogonal direction. The time, location and magnitude can be determined from the recorded data.

An accelerometer is located in buildings for recording large acceleration; therefore it is also known as strong motion seismometer. It does not run continuously, but will be activated by the earthquake itself when exceeding a certain threshold and runs for a fixed period (mostly 60 sec) of time. Unfortunately it gives only little information about the very low frequency components and furthermore heavy buildings nearby have a filtering effect biasing the frequency content.

2.1.3 Intensity and Magnitude Scale[12]

The Intensity and Magnitude scale help to classify the power of an earthquake. These classifications are not useable for seismic calculations.

The Richter Magnitude scale measures the emitted energy (in form of elastic waves) of the hypocenter of an earthquake. The magnitude is a logarithmic scale, which means that each whole number increase in magnitude mean an increase of 30 times of energy. The scale is expressed in whole numbers and decimal fractions from 1.0 to 10.0. E.g. a moderate earthquake be rated at 5.3; the strongest earthquake measured so far was classified with a magnitude of 8.7; Earthquakes with magnitudes 2.0 or less are rarely felt by people[13].

There are several similar intensity scales, mostly defining 12 levels, expressed in roman numbers to measure the noticeability and damaging power of an earthquake without using any mathematical base. This scale is more meaningful to nontechnical people and illustrates that an earthquake with the same magnitude can result in many deaths and damage in densely populated areas, whereas it may only frighten wildlife in remote areas. Definition for e.g. level V: “Felt by nearly everyone, many awakened. Some dishes, windows broken, unstable objects overturned. Pendulum clocks may stop”[14].

[11] cf. Booth, Edmund/Key, David: Earthquake design practice for buildings. London: Thomas Telford Publishing 2006, p. 26
AND cf. Lindeburg, Michael R./Baradar, Majid: Seismic Design of Building Structures. A Professional’s Introduction to Earthquake Forces and Design Details. Eighth edition. Belmont CA: Professional Publications 2001, p. 15
[12] cf. Booth/Key p. 21-22 AND cf. Lindeburg/Baradar p. 10-11
[13] Richter Magnitude Scale by Charles F. Richter, 1935
[14] Modified Mercalli Intensity Scale by Harry Wood and Frank Neumann, 1931)

2.1.4 Representation of Ground Motion[15]

Since intensity and magnitude scale are not useful for calculation, three other values (including their specific peak value, frequency content and duration of strong motions) are of interest for engineers *[example see Figure 5]*:

- <u>horizontal acceleration (**a_g**)</u>
  most important value for calculations; measured by seismometers;
  Acceleration is the rate of change of speed for example 0.001 g is perceptible by people; 0.02 g causes people to lose their balance; 0.50 g is very high, significant damages or collapse of buildings, measured in "g-force" whereas 1g = 980 cm/s^2
- <u>velocity (**v_g**)</u>
  obtained by integration of acceleration and it is the rate of change of position, measured in "cm/s"
- <u>displacement (**d_g**)</u>
  obtained by integration of acceleration and it is the distance from the point of rest, measured in "cm"

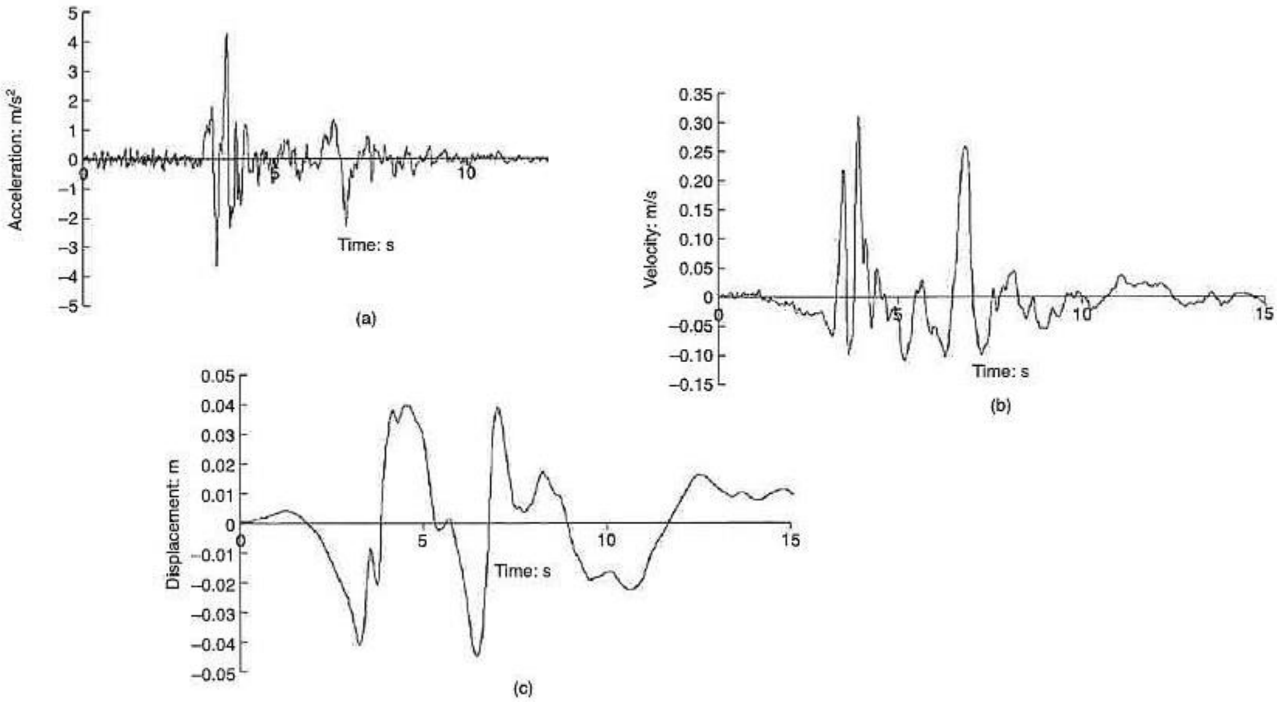


Figure 5 – time-history plots for a record from the Northridge, California earthquake of 1994: (a) acceleration; (b) velocity; and (c) displacement[16]

Several institutions and governments provide seismic hazard maps, where the local horizontal acceleration can be determined quickly. *[see Figure 41 on page 76]*

If the level of acceleration is combined with duration, the power of destruction is defined. Usually, the longer the duration, the less acceleration the building can withstand.

[15] cf. *http://www.wbdg.org/resources/seismic_design.php:* Seismic Design Principles. Gabor Lorant [15.03.2012]
[16] Booth /Key, p. 27

2.1.5 Earthquake Response Spectra[17]

The measured seismogram has two negative aspects, first it is difficult to determine the frequency content out of the recordings and second it is specific for the one measured earthquake, which was unique and won't be repeated with same power and amplitudes.

The earthquake response spectrum offsets both negative aspects *[see Figure 6]*. The structure is idealised as linear, elastic, SDOF, spring-mass-dashpot system subjected to the earthquake ground motion excitation. The peak responses of numerous spring-mass-dashpot systems, each with the same damping level but different natural frequencies, were determined and displayed as a function of the spectral acceleration [m/s^2] plotted again the natural frequency [Hz] or period [s]. The earthquake response spectrum contains more than one of these function-plots for different damping levels (usually 0%, 5% and 10%).

The peak in the spectra shows the natural period of which the structure has to reckon with severe damages by given "earthquake excitation". Whereby "earthquake excitation" means several time history plots of ground motion have been averaged and generalised to smooth out the spikes in the response, to provide a more general representation of possible ground motions.

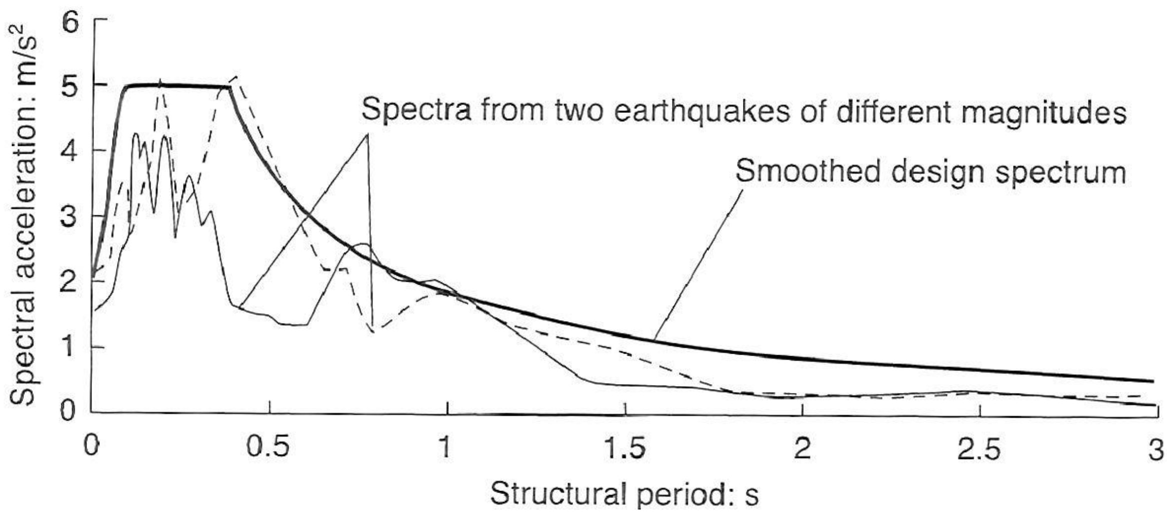


Figure 6 – smoothed design spectrum[18]

[17] cf. Booth/Key, p. 26 AND cf. Bachmann: Erdbebensicherung von Bauwerken, p. 38
[18] Ibid. p. 28

2.1.6 Damping[19]

Damping makes the cyclic excitation ceasing and the response tending to die away. Damping is a crucial characteristic of the dynamic properties of a structure. The influence on the structural behaviour is almost as important as structural period.

For analysis the damping is often assumed to be 'viscous' which means that the damping force varies with the velocity of the system.

The damping is expressed in terms of percentage of critical damping (where 100% of critical damping is the lowest level at which a system disturbed from rest returns to equilibrium without oscillation).

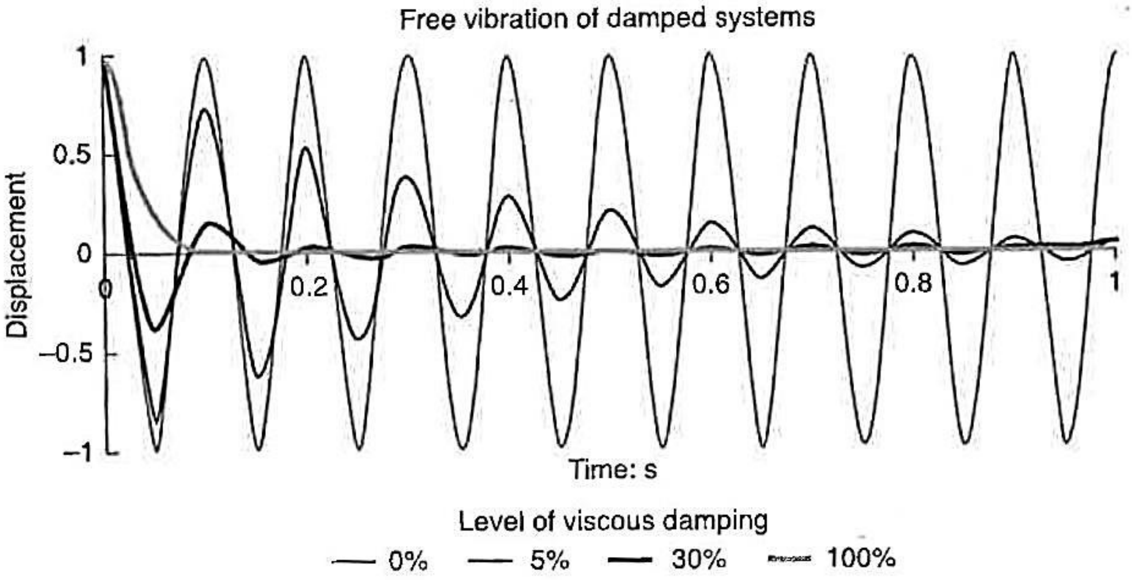


Figure 7 – effect of viscous damping level on the decay of free vibrations[20]

Figure 7 shows the damping reduction very well. The response becomes infinite as damping falls to zero and a significant reduce from an initial displacement from 1 to 0.7 can be seen after the first cycle at the 5% damped system.

A 5-7% damping can be found in most building structures without installing special damping devices, due to aerodynamic drag, friction in connection and cladding, damping associated with the soil and foundations and bond slip and cracking in reinforced concrete (when stresses are generally below yield; plastic yielding would give rise to a different source of energy dissipation = hysteretic damping)

With special energy dissipation devices a 30% damping level can be achieved. Some of these devices are shown in Section 2.2.3.3 on page 34.

[19] cf. Booth/Key, p. 42
[20] Ibid.

2.1.7 SDOF and MDOF[21]

Almost all practical structures are more complex than a single degree of freedom (SDOF) spring-mass-dashpot system; e.g. as used for the earthquake response spectrum.

Structures can be idealised as SDOF (e.g. water tower), which means that only the fundamental (first) mode of vibration is considered.

To determine the behaviour, including the stiffness and mass distribution of more complex structures, higher modes of vibration need to be considered as well – which are idealised as (MDOF) multiple degrees of freedom systems.

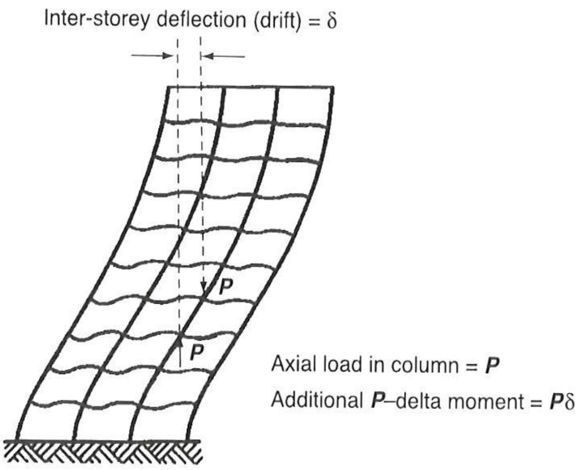
As long as a linear elastic behaviour is assumed each mode can be calculated separately and then combined, that is possible because each mode has an associated unique period and also unique mode shape. Therefore each mode of the MDOF system is an SDOF system.

2.1.8 Storey Drift[22]

The story drift is the relative, lateral displacement between the top and bottom of a storey. These movements in the upper stories accompanied by large second bending moments (P-delta effect) can badly influence the whole structural system. In a severe earthquake where structures yield, modern high-rise buildings will drift of approximately 2% of its total height at the roof level.

2.1.9 P-Delta Effect[23]

The columns in a structure are loaded in compression by vertical loads. Normally these loads are concentrically loaded on the column, but when a lateral force is applied the vertical loads become eccentric and additional forces, moments and increased story displacements will occur *[see Figures 8 and 9]*. That additional force increases the possibility that the column will fail or frames will buckle.



To consider this effect appropriately for the building's response a 3D model of the structure and the non-linear dynamic analysis is recommended. *[See Section 2.3.2.2 "Non-Linear Dynamic Analysis – NDA (or NDP)" on page 39.]*

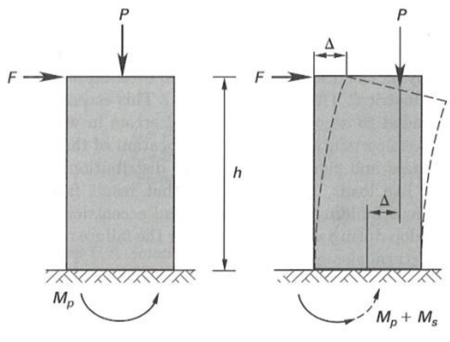


Figure 8 and 9 – P-delta moments[24]

[21] cf. Booth/Key, p. 49
[22] cf. Lindeburg/Baradar, p. 43
[23] cf. ibid., p. 44-45

2.1.10 Torsional Shear Stress[25]

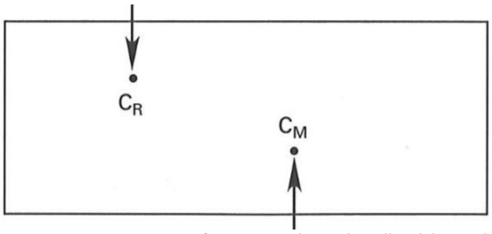


Figure 10 – center of mass and rigidity (building plan view)[25]

The total lateral seismic force *[see Figure 10]* is assumed to act on the building's center of mass.

Each structural member helps to counteract this lateral force, even though each member has a different rigidity level and thus a different lateral resisting force. The building's center of rigidity is a point through which the resultant of all the resisting forces acts. In the case that the centres of mass and rigidity do not coincide coupled lateral-torsional response occurs *[see Figure 11].* To consider this effect appropriately a 3D model of the structure and a non-linear dynamic analysis are required.

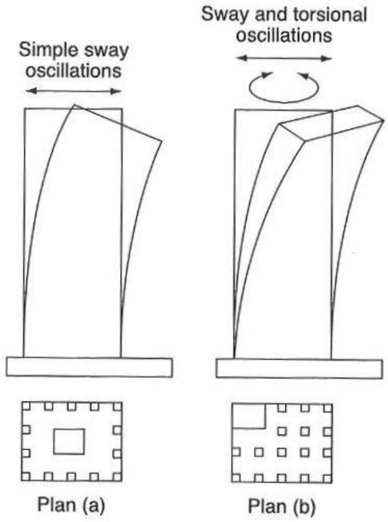


Figure 11 – lateral torsional response[26]

2.1.11 Overturning Moment[26]

The earthquake force is triangularly distributed over the height of a structure. The sum of moments due to the distributed lateral force is the overturning moment. This moment will increase the compressive stress in outer columns on the opposite side of the building and should be used to design these columns and to size the foundation. The tensions must be resisted with only the self-weight of the foundation. A safety factor of 1.5 against uplift, should be included according the Chilean seismic design codes[27]. *[see Figure 12]*

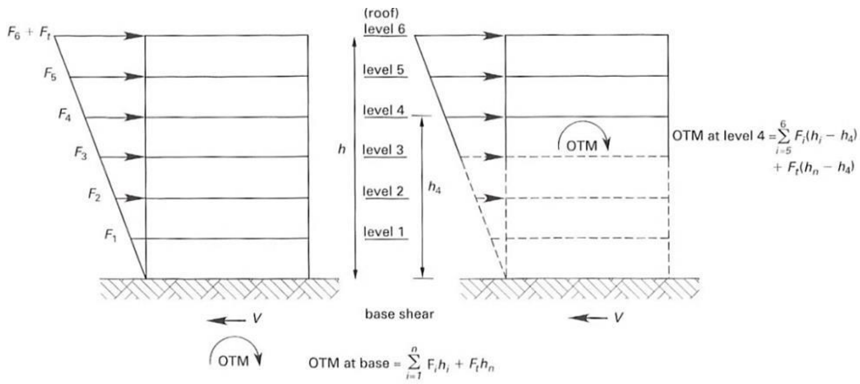


Figure 12 - overturning moment[28]

[24] Lindeburg/Baradar, p. 45 AND Booth/Key, p. 55
[25] cf. Lindeburg/Baradar, p. 45 AND cf. Booth/Key, p. 54
[26] cf. Lindeburg/Baradar, p. 48
[27] Instituto Nacional de Normalizacion (ed.): Official Chilean Standard. NCh2369.Of2003. Earthquake-resistant design of industrial structures and facilities. Santiago: INN 2003
[28] Lindeburg/Baradar, p. 48

2.1.12 Non-Linear Response

It makes a difference whether the building's response is considering material behaviour until yield strength is reached (linear – elastic deformation) or post-yield behaviour (non-linear – plastic deformation). It is generally uneconomical to design a structure to resist the earthquake in the elastic state without plastic deformations. Therefore most structures are designed to yield in order to exploit the materials potential[29].

"Ductility is the ability to withstand repeated cycles into post-elastic range without a significant loss of strength"[30].

This characteristic, together with the strength, the resistance against lateral forces are crucial for the seismic performance of a building. The relationship between these two characteristics[31] is:

a. Focus on Strength - the more the resistance against lateral forces is provided the less ductility behaviour is required;
b. Focus on Ductility - the more ductility behaviour is provided the less resistance against lateral forces is required to withstand a severe earthquake.

It is the engineer's choice in the design process on which of these alternatives he wants to focus.

a. Focus on Strength:
Building will resist earthquake without major plastic deformations

This option may be uneconomical and more expensive but it provides a massive and rigid structure, no problems with the nonstructural elements, the conventional design according to prescriptive codes is sufficient and damages are only expected by really strong earthquakes.

b. Focus on Ductility:
Building will be damaged, but does not collapse!

This option may cost less and it is more likely to withstand extremely strong earthquakes without a collapse by yielding substantially. However damages and plastic deformation will occur even due to minor earthquakes. Thus the ability of the nonstructural elements to follow the deformation movements and additional capacity design and appropriate structural design is required.

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[29] cf. ibid, p. 45 AND cf. Booth/Key, p. 55
[30] Booth/Key, p. 55
[31] cf. Bachmann: Erdbebensicherung von Bauwerken, p. 61-62

2.1.13 Building's Response to the Excitation[32]

This section should give an overview on how a building reacts and answers on the earthquake excitation.

The three main characteristics which contribute significantly to the building's response are:

- ground acceleration *(see Section 2.1.4 "Representation of Ground Motion" on page 15)*
- duration of strong shaking *(see below)*
- resonance effect *(see Section 2.1.13.3 " Resonance* Effect*" on page 22)*

2.1.13.1 Duration and Displacements of Ground Motions

The ground can move alternating horizontal in all directions (back, forth and sideways) as well as vertical up and down. The ground acceleration of these movements as well as the duration of the strong phase shaking affects the building's response.

The duration of the strong phase or in other words frequency content has a great influence on the extent of damage. But not all of the analysis methods *[mentioned in Section 2.3 "* The Calculation of the Structural Response*" on page 37]* take the frequency content into account, only the linear or non-linear dynamic time-history analysis does.

The movements occurring due to earthquakes with medium magnitude will last about 10 to 20 seconds (strong-phase), for instance. The dimension of the deflection varies, for a magnitude 6 earthquake, between 8 to 12 centimetres.

2.1.13.2 Inertial Force

The ground motions force the foundation to fulfil the same movements. However the upper parts of the building tend to stay at the same place, due its mass inertia. The resulting structural vibrations force the building to undergo several modes of vibration according to the alternating ground motions *[see Figure 13]*. For seismic purpose the fundamental period or first mode is usually the most significant, except for very tall, complex or critical buildings. The general seismic engineering approach is to consider the first 10 modes, because those will move more than 90% of the building's mass.

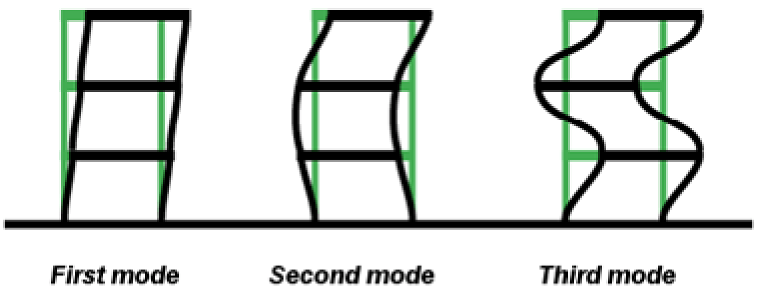

Figure 13 - several modes of building's vibration[33]

[32] cf. http://www.wbdg.org/resources/seismic_design.php: Seismic Design Principles. Gabor Lorant [15.03.2012] AND cf. Bachmann, Hugo: Erdbebengerechter Entwurf von Hochbauten – Grundsätze für Ingenieur, Architekten, Bauherrn und Behörden. Biel: BWG 2002, p. 7

[33] © http://www.wbdg.org/resources/seismic_design.php: Seismic Design Principles. Gabor Lorant [15.03.2012]

The movements by change of the vibration mode generate internal forces within buildings, called the Inertial Force.

$$\boldsymbol{F}_{Inertial} = Mass\ of\ the\ building(\boldsymbol{m})\ x\ Acceleration\ (\boldsymbol{a})$$

Equation 2 – inertial force

The more mass is considered (weight of the building), the greater the internal inertial forces generated which increases the possibility of columns being displaced, out of plumb, and/or buckling under vertical load. Therefore lightweight construction with less mass is typically an advantage in seismic design.

2.1.13.3 Resonance Effect[34]

All objects, including buildings, have a natural, fundamental eigenfrequency at which they oscillate if excited by a shock *[see Figure 14]*. Natural frequencies for houses range from **0.1 to 10 Hz**.

If the frequency (or period) of the shock wave and the natural frequency (or period) of the building coincide, then the building will resonate and its vibration will increase several times which in turn causes most structural collapses and losses.

$F_{ex} = F_I \rightarrow$ *Collapse!!*

The same applies for the inverse of the frequency, the fundamental period.

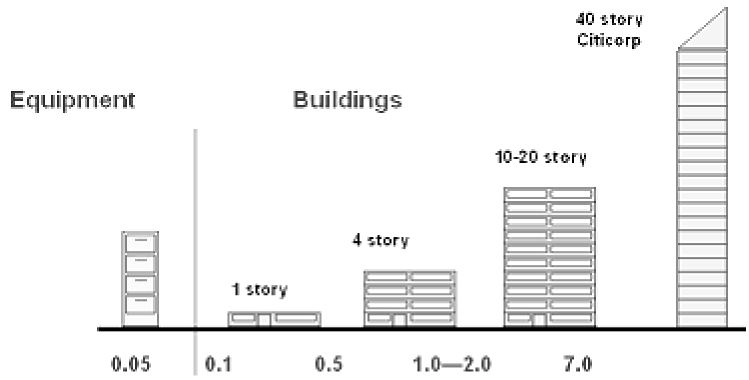

Figure 14 – Fundamental periods of buildings[35]

Each object has its own fundamental period at which it will vibrate. The period is proportionate to the height or more generally the elasticity of the building. The more rigid the structure is the more the natural frequency will go up (accompanied by a short fundamental period), the more flexible, deformable (like a spring) the less the natural frequency will be accompanied by a long fundamental period.

The soil also has a period varying between 0.4 (rock) and 1.5 sec., very soft soil being 2.0 sec. Soft soils generally have a tendency to increase shaking as much as 2 to 6 times as compared to rock.

[34] cf. Ibid. AND cf. Booth/Key, p. 40
[35] © *http://www.wbdg.org/resources/seismic_design.php:* Seismic Design Principles. Gabor Lorant [15.03.2012

In case the period of the soil coinciding with the natural period of the building the acceleration of the building will be greatly amplified.

According to these figures above, it is not unlikely that e.g. a 4-storey building with its associated fundamental period of 0.5 sec match with a firm soil with the same period. For this reason it is essential to always consider the resonance effect.

2.1.14 Conclusion

According to the previous sections it sounds very simple to avoid a collapse. Engineers only have to choose the right ratio of ductility and resistance to lateral force and take care that the natural frequency does not coincide with the excitation frequency. The excitation frequency cannot be influenced by humans in contrast to the natural frequency of the building. Engineers can do that by sticking to some important rules within the preliminary design stage (choosing height, mass or structural system of the building).

Sounds simple? It is not! – A lot of uncertainties make this task very difficult and the accuracy or predictability of the solutions will never ever reach 100%. The engineering task is to design and build structures capable to cover these uncertainties[36].

Uncertainties on the excitation-side:

There are earthquake-related or location-related uncertainties. The ground motion parameters can be varying a lot even despite an equal earthquake magnitude. That depends on distance, direction, depth and mechanism of the fracture zone (hypocenter) or on location-related parameters like soil type, layer thicknesses and shear waves velocity on site.

Uncertainties on the building-response-side:

The response depends on the building's characteristics like natural frequency, structural system, system's ductility and the uncertainties within analysis- and assessment procedures.

[36] cf. Bachmann, Hugo: Erdbebengerechter Entwurf von Hochbauten, p. 7

2.2 Seismic Design Process

The seismic design process consists out of three steps. First step is the preliminary design which includes setting building configurations, selecting the basic structural system and ensuring that continuous a load path from the top to the foundation exists. Furthermore some principles are explained to limit the vulnerability of structures in seismic events. In the second step the developed design concept (from the first step) must be analysed and confirmed by calculation of the expected structural and nonstructural response *[see Section 2.3 "* The Calculation of the Structural Response*" on page 37].* The last step is to ensure the previously developed and agreed construction quality and to verify the progress with current inspections during the construction phase.

2.2.1 Preliminary Design

The decisions made in this early phase should be carefully considered. "Nothing, within the power of a structural engineer can make a badly conceived building into a good earthquake-resistant structure"[37].

This basic decisions run through the entire project and changes would go along with very high costs and in case that the performance-based design approach is applied, an inappropriate design would result in extensive iterations until an appropriate solution is found and the performance objectives are met.

Decisions must be made about:

- the overall structural concept
- the building configuration
- the structural system and material
- energy-dissipating devices

This section closes with the presentation of some principles to limit the vulnerability of structures in seismic events.

2.2.2 Structural Concept and Building configuration

Knowledge of the seismological basics *[in Section 2.1 "Seismological Basics" on page 13]* is essential to develop a good structural concept and set building configuration. The consequences of each decision, of each point which influence building's period, torsion, stiffness, ductility and strength must be understood. *[See Figure 15 - for exemplary structures, which are not able to provide proper earthquake resistance.]*

[37] Booth/Key, p. 96

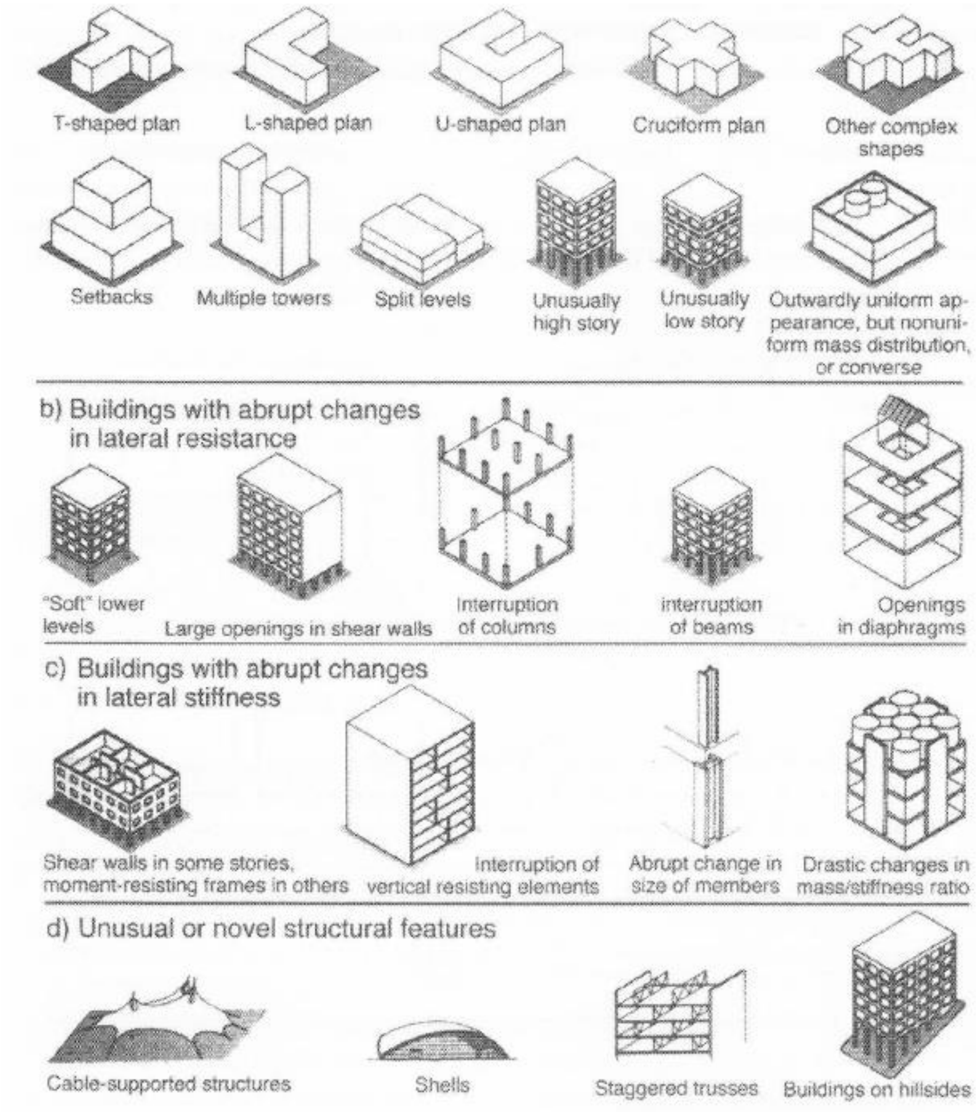

Figure 15 – irregular structures or framing systems acc. SEAOC[38]

2.2.2.1 Building's Period

The building's period depends on the ratio between width and height, the slenderness. The more rigid (low) the structure is, the shorter is the fundamental period. A flexible (high) construction has a long fundamental period. The height and the size of the floor plan mainly influence the buildings period.

2.2.2.2 Torsion

[See Section 2.1.10 "Torsional Shear Stress" on page 19]. It was stated that a torsion force occurs if center of mass and center of stiffness do not coincide.

The center of mass can be influenced by the arrangement the vertical load bearing elements. To avoid a collapse the center of mass should somewhere in the middle of the floor plan and not concentrated in the upper storeys of the building, because that would help increasing the swaying motion. Therefore the center of mass is more or less predefined.

[38] Bachmann, Hugo: Erdbebensicherung von Bauwerken, p. 89

The center of stiffness, on the other hand, is influenced by the arrangement of the load bearing elements which resist the lateral seismic load. The arrangement of these elements is only restricted by the architect's instructions of open spaces or places for windows or door openings.

The shape and layout of the floor plan mainly influence the torsional effect.

The plan shape should be compact to ensure the coincidence of the two center points. Furthermore due to the swaying motions, high stress concentrations is developed in the corners of complex plan shapes (like H, L *[see Figure 16]*, T, U, + etc…) because each extension vibrates separately and follow its own movements. To overcome this problem complex plan shapes should be separated into several compact plan shapes by seismic joints.

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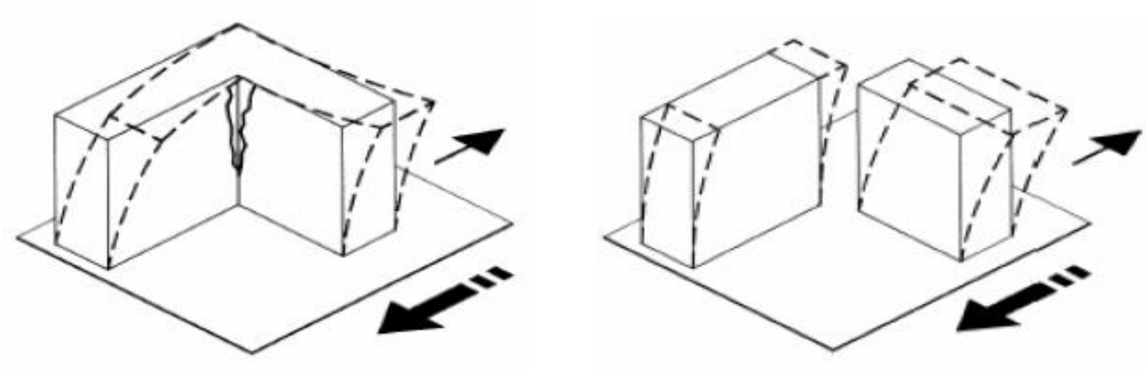

Figure 16 – torsional effect on plan shapes[39]

2.2.2.3 Ductility

[See Section 2.1.12 "b) Focus on Ductility on page 20]. Ensuring structural ductility is the best way to cover uncertainties in prediction earthquake motions and the calculated response and the best insurance policy to protect human lives. One way to do that, is applying the capacity design approach.

<u>The Capacity Design Approach</u>

This approach is based on the idea to allow weak sections plastic deformations (post-yield behaviour) in a controlled way and strengthen the other parts of the construction to ensure the yielding only in designated areas. The yielding in the weak parts itself is controlled and must represent one of the prescribed qualified yielding modes with ductile rather than brittle response (controlled bending is acceptable, buckling or connections-failures not).

[39] Tafreshi, Kamyar Tavoussi: Zum Tragverhalten von mehrgeschossigen Holzbauten unter Erdbebenbeanspruchung. Übertragung von Versuchsergebnissen an teilbiegesteifen Rahmenecken in praxisnahe Rechenmodelle. [Diss. Wien 2004] Wien: TU Wien 2004; A4-1

2.2.2.4 Strength and Stiffness

Failure modes like the storey drift *[see Section 2.1.8 "Storey Drift" on page 18]* occur due to insufficiently strong and stiff structures. These deformations must be limited to prevent nonstructural elements of damage and the P-delta effect *[see Section 2.1.9 "P-Delta Effect" on page 18]* occurring by vertical loaded columns.

Strengthening or making buildings stiffer consequences in shortening the building's period. The local circumstances and the possibility of amplification with the ground motions define whether it is the right approach or a lengthening of the period may be required. A lengthening can be archived by base isolation. *[see Section 2.2.3.3.1 "Base Isolation" on page 34]*

For example, structures build on soft soils need to be designed very stiff to prevent natural period amplifications.

To provide sufficient stiffness, a continuous load path, from top to the foundation, of the structural elements which contribute resisting the lateral force must be ensured. Types of lateral-force-resisting-systems are presented in Section 2.2.3.2 "Lateral-Force-Resisting Structural Systems" 30.

To provide sufficient strength all the load bearing elements including their connections must be designed strong enough to minimize deformations like the storey drift *[Section 2.1.8 on p. 18]*.

The standard load path looks as follows:

The seismic force is delivered to the horizontal diaphragms (floors and roofs), which distribute these forces to verticals member, which transfer the loads to the foundation, which in turn transfer the load to the supporting, surrounding soil[40].

Hereby it is essential to avoid vertical and horizontal irregularities in the arrangement of these lateral load bearing elements. An interrupted load path *[see Figure 17]* significantly reduces the strength and stiffness of the structure and cause additional stress concentration on other elements within the building. The recorded damages due to discontinuity in load path have been 5 to 10 times worse, compared to damages in buildings with continuous, symmetric, stiff elements.

An increasing stiffness over the height of the building results in worse performance than if the stiffness decreases with increasing height[41].

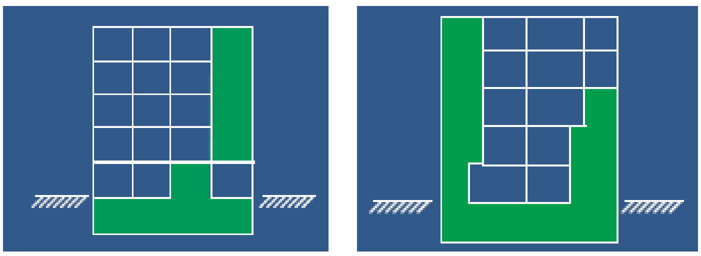

Figure 17 – interrupted load path of lateral-resisting-elements[42]

[40] cf. Shihada, M. Samir: Earthquake-Resistant Systems. Gaza: Handout 3-12. 2012
[41] cf. Bachmann, Hugo: Erdbebengerechter Entwurf von Hochbauten, p. 24-25
[42] Ibed.

Even worse is the performance in case of missing lateral load bearing elements in one whole storey. Columns can be arranged alternatively by architectural reasons. That measure would cause “soft levels”, *[see Figure 18]* whereby the deformations lead to failure in the upper and lower connections of the columns and leads to total collapse of the soft storey and the upper parts falling down.

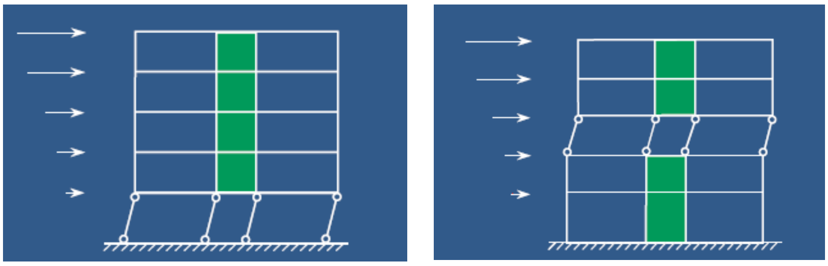

Figure 18 – soft storey effect[43]

2.2.2.5 Influence of the soil[44]

The soil conditions can have major influence on the seismic design. Each soil-type has a specific period range over which the seismic motions will be amplified, if the structural period fall into this range. Deep, soft soils are bad for tall buildings and shallow, stiff soils can be damaging for low-rise buildings. In such cases the structural period must be modified *(more stiffness = shorter period, less mass = shorter period)* to avoid high stresses due to the resonance-effect.

The potential of liquefaction and the stability of slopes nearby the construction site must be known too. These data can be obtained by a proper geotechnical investigation.

2.2.2.6 Conclusion

Below, some principles are listed to summarize the previous chapter:

- the buildings height influences the structural period of the building
- the center of mass should coincide with the center of stiffness by arranging the lateral, “stiff”, load bearing elements
- if the building provides a good ductility the bearing capacity is economically enhanced without strengthening the construction and a total collapse can be avoided by controlled and planned deformations; the capacity design approach should be applied
- to ensure sufficient strength and stiffness, a continuous load path without any vertical or horizontal irregularities of the lateral load bearing elements should be provided
- to avoid amplifications between the structural period and the seismic motions it is essential to know the soil conditions and adapt the structural systems to avoid amplification

Regular-configuration-buildings have a good ductility and load-bearing ability.
Irregular-configuration-buildings have problematic stress concentrations and torsion.

[43] Bachmann, Hugo: Erdbebengerechter Entwurf von Hochbauten, p. 15-20
[44] cf. Booth/Key, p. 97

Regular Configuration buildings generally have[45]:

- a lateral-resisting structure *[see Section 2.2.3.2 "Lateral-Force-Resisting Structural* Systems*" on page 30]*
- low height to base ratios
- equal floor heights
- symmetrical plans
- uniform sections and elevations
- maximum torsional resistance
- short spans and redundancy
- direct load paths

Irregular Configuration buildings are those that differ from this "regular definition"[46].

2.2.3 Structural System and Material

The structural system and the material must be chosen under consideration of the local circumstances. This section describes the advantages and disadvantages of common materials with regard to structural systems which resist lateral forces and add stiffness to the structure.

2.2.3.1 Materials[47]

2.2.3.1.1 Steel

The most common used material in industrial construction is steel.

Steel has a high strength-to-mass-ratio. Furthermore it is easy to make steel members ductile both in flexure and shear. The disadvantages of this material are that it is difficult to make member connections seismic-resistant and that a ductile behaviour cannot be provided if buckling failure occurs.

2.2.3.1.2 Concrete

Concrete buildings have an unfavourable low strength-to-mass-ratio, which will generate a greater seismic impact on the building.

The design quality defines the performance of the concrete beams and columns. They are very brittle in shear or compression but when applying the capacity design approach, they can be made ductile.

Concrete shear walls *[one of the structural systems explained in the chapter below]* have excellent performance records in earthquakes, even if design and construction quality have been less than perfect.

2.2.3.1.3 Other materials

Other materials like masonry and timber are normally not used for industrial construction. Timber shows a very good seismic performance, due to its strength and low weight. Masonry is generally not recommended in seismic areas. Not even as infill, like illustrated in *Figure 19*.

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[45] cf. *http://www.wbdg.org/resources/seismic_design.php:* Seismic Design Principles. Gabor Lorant [15.03.2012]
[46] Ibid.
[47] cf. Booth/Key, p. 103

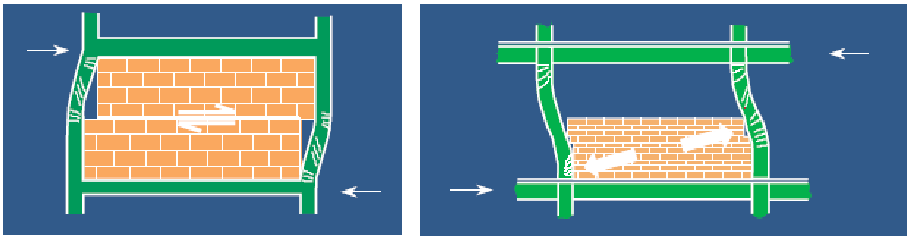

Figure 19 – masonry frame-infill and parapet[48]

2.2.3.2 Lateral-Force-Resisting Structural Systems

Three different lateral-force-resisting systems are described in this section. No matter which system is chosen, a continuous load path must be provided to prevent "soft storeys" *[see Section 2.2.2.4 "Strength and Stiffness" on page 27]*. The three systems are:

- shear walls
- braced frames (concentric or eccentric)
- and moment resisting frames

2.2.3.2.1 Shear Walls[49]

Shear walls are strategically arranged reinforced concrete walls, which are capable to transfer the lateral forces from floors and roofs to the foundation and provide stiffness to the structure. These walls must be designed ductile by capacity design. The shear wall (which runs over the entire height of the building) acts as a beam cantilevered out of the foundation and the force is absorbed by shear and bending motions in the plane of the wall.

This system is suitable for medium rise buildings up to 20 stories. It is essential that shear walls are combined with horizontal diaphragms to provide the transmission of the horizontal earthquake loads to the vertical load-bearing shear walls.

The walls should be arranged symmetrically in both length and width of the building, to prevent a torsional effect *[see Figure 21]*.

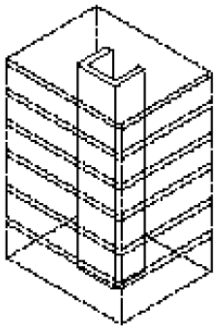

Figure 20 – shear wall building[50]

[48] cf. Bachmann: Erdbebengerechter Entwurf von Hochbauten, p. 29
[49] cf. Shihada, M. Samir: Earthquake-Resistant Systems. Gaza: Handout 3-12. 2012
[50] cf. Ibid.

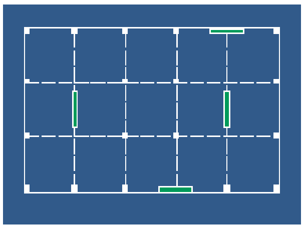

Figure 21 – Arrangements of hear walls[51]

A basic rule is to arrange two reinforced concrete walls (3m–6m width) in each main direction over the entire building height. Walls enclosing stairways, elevator shafts or mechanical shafts are used as shear walls, *[see Figure 20]* because they extend the entire height (from roof to foundation) and leave exterior walls open for windows and other architectural purposes. Openings in shear walls should be avoided as they significantly reduce the ability to resist the lateral force. Shear walls located at the corners of a building should be avoided too.

The thickness of these walls varies between 150 to 400 mm. The most important aspect of shear walls is their ductile behaviour, which must be ensured trough appropriate geometric proportions, the use of the right type and amount of reinforcing and choosing correct connections.

The advantages of shear walls are, that:

+ they are easy to construct
+ they are efficient
  (the construction costs are low compared with their good performance)

The disadvantages of shear walls are, that:

- special detailing is required in highly seismic regions
- they may interfere architectural requirements
- the foundation could be subjected to an uplift under seismic excitation – because *"shear wall acts as a beam cantilevered out of the foundation" [see Section 2.1.11* "Overturning Moment" *on page 19 for respective design considerations]*

2.2.3.2.2 Braced Frames

Braced frames are an efficient lateral force resisting system. The stiffness is provided by a truss system, which resists through axial stresses in the members. The diagonals must account for compression and tension and provide the continuation of the load path to foundation[52]. See a braced-frame building in *Figure 22.*

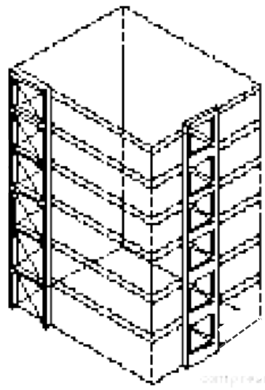

Figure 22 – braced frame building[53]

Unfortunately steel shows an unfavourable performance under cyclic loads. In tension, the diagonals may still perform in elastic range. But if weak sections are subjected suddenly to compression, the diagonals may buckle.

[51] cf. Shihada Handout
[52] cf. Ibid.
[53] cf. Ibid.

Figure 23 – types of concentric bracing[55]

Using this structural system ductility is limited. Problems may occur with the architects as they do not have their desired freedom by arranging doors or windows. However, it is a good way to provide strength and stiffness at low cost[54].

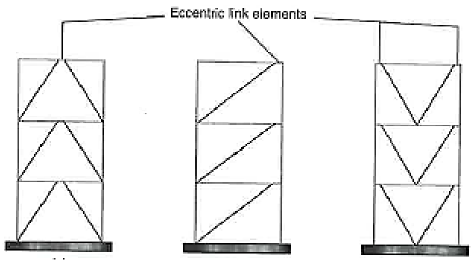

Figure 24 – types of eccentric bracing[56]

A way to enhance the ductility of braced frames is to design them eccentrically braced. *So far the concentrically braced frames have been discussed – see Figure 23.*

Eccentrically braced frames have an excellent energy absorption by designing short weak links which yield before any other (more important) member does. By this way, buckling or irreversible yielding is prevented – see types of eccentric bracing in *Figure 24.*

2.2.3.2.3 Moment-Resisting Frames

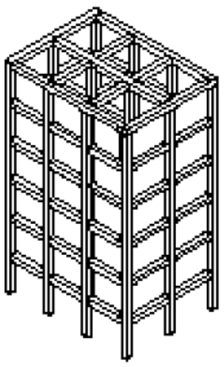

Figure 25 – moment resisting frame building[57]

The third type of lateral force resisting systems is the moment-resisting frame *[see Figure 25].*

This is a complete space frame made either of steel or concrete, consisting of columns and beams, which transfer the lateral and the vertical loads from the roof and floors to the foundations.

The lateral strength is not provided by diagonal bracing members, like it is the case at the braced frame, but from the rigidity of beam-column connections[58].

The column-beam joints and connections must be carefully designed to be stiff and rigid, yet to allow some deformation for the energy dissipation by taking advantage of the ductile behaviour of the steel or due to controlled deflection from the columns bending[59].

[54] cf. Booth /Key, p. 107
[55] cf. Ibid.
[56] cf. Ibid., p. 109
[57] cf. Shihada Handout
[58] cf. Booth /Key, p. 103
[59] cf. Shihada Handout

The advantages of moment-resisting frames are[60]:

+ more freedom in architectural planning, no walls or diagonal members where windows are foreseen
+ good energy absorption
+ being a flexible system with a long structural period, which is good on stiff soil or rock sites

The disadvantages of moment-resisting frames are[61]:

- special attention must be paid on the design of the connections; weak stories and failure in beam-columns joints due to high stress should be avoided
- special attention must paid on the construction quality; good fixing skills and careful concreting is required
- due to the flexible system low stiffness is provided, with the result of high storey drifts; which is a problem for nonstructural components inside the building and adjacent buildings nearby

2.2.3.2.4 Diaphragms[62]

The floor and roofs must act as rigid horizontal slabs and the connection to the vertical elements must be shear-resistant to guarantee the diaphragm-effect. One solid, monolithic element performs way better than several elements connected by a concrete top layer. *[see Figure 27]* Separate prefabricated elements can not provide the diaphragm-effect.

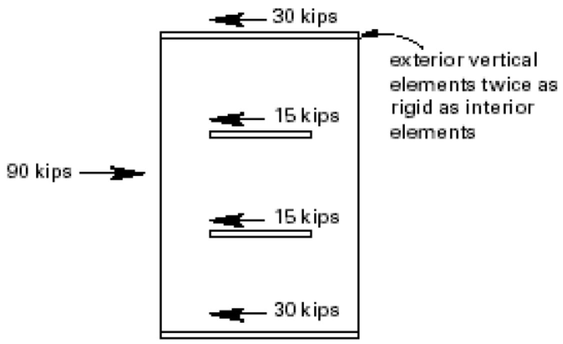

Figure 26 – distribution of lateral force/diaphragm effect[63]

If the effect is provided, the slab acts as a horizontal beam and transmits the lateral force, in proportion to their relative stiffness's, to the vertical resisting elements (shear walls, braced frames or moment-resisting frames). *[See figure 26]*.

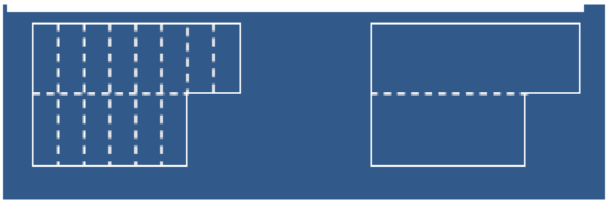

Figure 27 – prefabricated elements vs. monolithic slabs[64]

[60]cf. Shihada Handout AND cf. *http://www.wbdg.org/resources/seismic_design.php*
[61] cf. Shihada Handout AND cf. Booth/Key, p. 104
[62] cf. Shihada Handout
[63] Shihada Handout
[64] Bachmann: Erdbebengerechter Entwurf von Hochbauten, p. 53

2.2.3.2.5 Foundation[65]

The foundation is the last part of the lateral-force-load path. If the structure above provides a high level of ductility, the foundation also needs to perform without brittle failure modes. Piles for example should be flexible (with small diameter of <0.5 m) to accept movements near the surface without suffering large bending stress. Another requirement of the foundation is the ability to resist the overturning moment and uplift forces, caused by shear walls due to their high height-to-width ratio. *[See Section 2.1.11 "Overturning Moment" on page 19].*

2.2.3.3 Energy-Dissipating Devices[66]

Strengthening a structure to prevent damage is the common approach in structural engineering. For the seismic design purpose, strengthening a structure means adding to the building's mass and shortening the structural period, which is accompanied by an undesired increase of the acting seismic base shear force.

Absorption of this lateral force must be provided through controlled deformations due to ductile material behaviour in designated areas and a working energy transfer to the soil by a continuous load path of the lateral-resisting elements. Even both mechanisms are provided, structural and nonstructural damages by an earthquake occurring with stronger magnitudes than expected, can not be avoided.

To overcome this, energy dissipating devices can be installed. Such devices reduce the design inertia force, filtering out the high-frequency accelerations, limiting the storey drift, increasing protection on nonstructural components and enabling that the occupants are less aware of the earthquake motions, which is an important aspect in densely populated areas.

Energy can be dissipated through two principle ways:

- base isolation, or
- mass damping

2.2.3.3.1 Base Isolation

Base isolation means that the building is mounted on bearings with low lateral stiffness (mostly rubber) to absorb a large part of the shock and separate it from the foundation.

The consequence is, that the shaking is slowed down to a tolerable range by lengthening the natural period of the building and taking it away from danger of resonance with the fundamental period of the ground motion. But base isolation is not suitable on soft soil sites, the overturn and uplift forces of tall buildings are enhanced by the bearings and a flexible design of supply line- and duct-inlets and sufficient space between adjacent buildings must be ensured.

Therefore, base isolation is suitable for stiff (building must be designed as a rigid box - "must act as a unit"), low-to medium-rise (8-10 storeys for unbraced frame buildings and 12-15 storeys for shear wall) buildings on stiff soil sites without the risk of overturning or amplification with the earthquake motion (which is the case by tall buildings on soft soils). The isolation effect can be enhanced by placing hysteretic, viscous or frictional damping elements on the bearings.

[65] cf. Booth /Key, p. 102-103
[66] cf. Booth /Key, p. 235-252 AND cf. *http://www.wbdg.org/resources/seismic_design.php*

2.2.3.3.2 Mass Damping

The structural period of tall buildings is already long. No lengthening through base isolation is required. Indeed, quite in contrast, a shortening of the structural period and a method to control and reduce the horizontal displacements is required.

This can be achieved by mass dampers. Mass dampers are huge and massive concrete blocks or steel pendulums mounted on top of tall buildings. The dampers move in opposite to the earthquake-excitation-oscillations of the structure and in doing so, decrease the resonant amplifications of the lateral displacements of the building[67].

2.2.3.3.3 Conclusion

Mounting a building on bearings will lead to in an increase of the structure's period. Using mass damping devices will consequence in a decrease of the structure's period.

To choose an appropriate energy dissipating device the building's period-, torsion-, ductility-, strength- and stiffness-behaviour must be known.

According to a study[68] comparing the seismic performance of nonstructural components in various building types, base isolated buildings significantly showed the best performance in reducing both, drift and acceleration impacts on nonstructural components.

Adding energy-dissipating devices is in the range of 1-2% of the total structural costs. Considering the improvement of performance it is a good investment. Retrofitting with base isolation is, understandably, much more complicated than with dampers.

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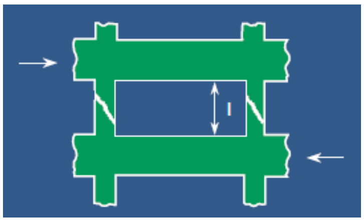
[67] cf. *https://en.wikipedia.org/wiki/Earthquake_engineering#Seismic_vibration_control*: Earthquake Engineering. Wikipedia [13.02.2015]

[68] cf. ATC – Applied Technology Council (ed.): ATC-69. Reducing the Risks of Nonstructural Earthquake Damage. State-of-the-Art and Practice Report. Cooperation with NEHRP, California: Applied Technology Council 2008

2.2.3.4 Some more Principles for Earthquake-Resisting Structures[69]

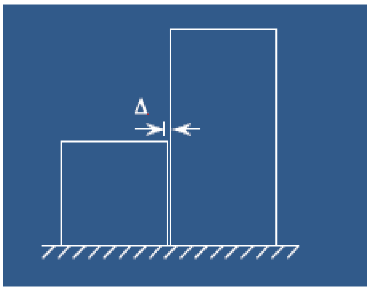
2.2.3.4.1 Avoid Short Columns



Short and stout (in proportion to their height) columns produce huge shear forces under seismic excitation, which result in shear fractures occurring even before deformations happen. By arranging parapets afterwards, this effect is often provoked. *[see Figure 28]*

Figure 28 – avoid short columns[70]

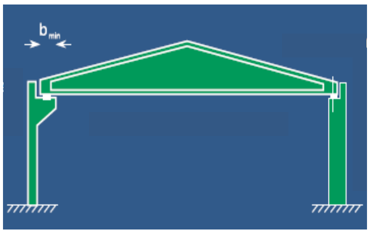
2.2.3.4.2 Providing sufficient Space between Adjacent Buildings



In the case that adjacent buildings of different height shake and pound on each other, the floor or roof slabs of the lower building could damage columns in the higher one. The required width of "seismic joints" can be found in local prescriptive codes. *[see figure 29]*

Figure 29 – space between adjacent buildings[71]

2.2.3.4.3 Ensure strong Connections of Prefabricated Elements



To ensure sufficient strong connections of prefabricated elements, the minimum support width according prescriptive codes must be followed and the use of shear dowels is recommended. The support of the prefabricated beams must be protected against tilting with special mountings. *[see figure 30]*

Figure 30 – support connection of prefabricated elements[72]

2.2.3.4.4 Considering Interaction between Structural and Nonstructural Components

[See Section 4.2 "Design of Nonstructural Components (State of Art)" on page 59.]

[69] cf. Bachmann: Erdbebengerechter Entwurf von Hochbauten
[70] Ibid., p. 42
[71] Ibid., p. 50
[72] Ibid., p. 62

2.3 The Calculation of the Structural Response

The first provision for seismic design was that buildings have to resist a lateral force at equal proportion of the building weight. Within the evolution of design, it became clear that the behaviour of the structure affects the loads generated during an earthquake and vice versa. The concept of response spectra was developed in 1930. Since then earthquake engineering has developed a lot and it was made possible to generate very large and complex structure models by using advanced computer technologies (e.g. explicit finite element analysis)[73].

An accurate seismic analysis of the structural response is still challenging. The unpredictable dynamic ground motion imposes extreme cyclic loads on different materials, which respond in a very complex way. Therefore the results of seismic analyses are beset with uncertainties.

This section describes main linear and non-linear methods, with static or dynamic force inputs.

2.3.1 Linear Elastic Seismic Analysis

This analysis method does not take the plastic material behaviour into account. The material capacities are considered to be used just below yield strength and the post-elastic bearing capacities are neglected. It is recommended to always combine linear analysis with capacity design approach to ensure a satisfactory elastic response as well as controlled post-yield behaviour in selected sections.[74]

2.3.1.1 Equivalent Linear Static Analysis – LSA (or LSP)

That is the most basic procedure. The static aspect herein is, that the dynamic nature of the seismic load is reduced to a set of lateral forces with prescribed vertical distribution, which should be equivalent to the earthquake ground motion effect causing the fundamental mode of structure's response from a given earthquake. This force is determined using the mass of the structure and the appropriate response spectrum acceleration, typically defined by the seismic design response spectrum on the structure's period and damping. Although the procedure is described as linear, geometric nonlinearity such as P-delta effects are considered[75].

Summarized, the linear static analysis replaces the dynamic seismic excitation by a corresponding static force and assumes that the building responds principal in its fundamental vibration mode with a linear model stiffness before yield occurs. For this to be true, the building must be regular in plan and elevation, low-rise and must not twist significantly (no torsion) due to the ground moves[76].

In many building codes the applicability is extended by factors to account for higher buildings with some higher modes, and for low levels of twisting. To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces[77].

[73] cf. *https://en.wikipedia.org/wiki/Seismic_analysis*: Seismic analysis. User: Billinghurst. [29.09.2014]
[74] cf. Booth/Key, p. 63
[75] Williams, Matthew Joseph: Performance Based Analysis of Steel Buildings. [Diss. San Luis Obispo 2009]: California Polytechnic State University, p. 7
[76] cf. *https://en.wikipedia.org/wiki/Seismic_analysis*
[77] cf. *https://en.wikipedia.org/wiki/Seismic_analysis*

2.3.1.2 Linear Dynamic Analysis – LDA (or LDP)

The linear term means that a linear elastic stiffness (material behaviour) and equivalent viscous damping is assumed. Wherein the dynamic term means that the model is subjected to a maximum response force calculated from a smoothed-enveloped linear elastic response spectrum which gives dynamic, generalized values of several different earthquakes. Furthermore not only the fundamental mode of deformation considered, but also higher vibration modes. It is required to consider sufficient modes to capture 90% of the building mass in the building's two orthogonal directions[78].

There are two methods of modelling for linear dynamic analyses: the modal spectral analysis or the time history analysis.

2.3.1.2.1 Modal Spectral Analysis – RSA[79]

In contrast to the time history analysis the advantage of the modal spectral analysis is that the problems accompanying calculating the entire time history are reduced to finding only the maximum response of a limited number of modes of structure. On the other hand this is disadvantageous because when the results are only in terms of peak response, a loss of information on frequency content, phase and number of damaging cycles (fatigue effect), is the consequence. Furthermore it is assumed that the peak responses occur simultaneously, which is practically not the case (for example the axial force is dominated by first mode, whereas bending moment and shear is influenced by higher modes, therefore it will peak at different times) and that all grounded parts have the same input motion.

2.3.1.2.2 Linear Time-History Analysis[80]:

With the linear time-history analysis, the response of the structure to ground motion is calculated in the time domain and all phase information is therefore maintained. That overcomes all disadvantages of RSA, however by a significantly greater computing effort.

2.3.2 Non-Linear Analysis

In comparison to linear analysis the non-linear analysis considers post-yield behaviour in the structural model.

[78] Williams: Performance Based Analysis of Steel Buildings, p. 9
[79] cf. Booth/Key, p. 65-66
[80] cf. Booth/Key, p. 66

2.3.2.1 Non-Linear Static Analysis - NSA (or NSP) [81]

Like the linear static analysis, a pattern of lateral forces (reasonable equivalent to actual earthquake force) with its vertical distribution is applied to a structural model with only the fundamental vibration mode, but the model is able to perform non-linear strain distribution in this case.

A non-linear frame structure is modelled as SDOF *[see Section 2.1.7 "SDOF and MDOF" on page 18];* the peak displacements determined directly from a design spectrum based on ground motions were imposed on the frame to determine the plastic strains and their distribution. The deflection of the top structure is then plotted against the total shear force to define a pushover or capacity curve. Therefore this approach is also known as "displacement-based" method or "pushover" analysis.

Two methods to calculate the maximum deflection of a pushover curve are: the target displacement method acc. to FEMA 356[82] or Capacity spectrum method acc. to ATC-40[83].

These methods are unsuitable when expecting a torsional building response (centres of mass and stiffness do not coincide), because SDOF considers only the translational and not torsional response of the cyclic excitation. The advantage is, like at their linear equivalent, using just the peak values in order to avoid difficulties by choosing suitable ground motion time histories like it is the case at the non-linear dynamic time-history analysis.

2.3.2.2 Non-Linear Dynamic Analysis – NDA (or NDP)[84]

So far, the non-linear time history analysis enables the most complete assessment by combining a detailed structural model *(considering the nonlinear characteristics of the individual components and variation in time-dependent parameters, like possible loss of strength and stiffness of plastic hinge under repeated cyclical strains)* and earthquake shaking represented by ground motion time-histories which the model is subjected to. The computer output shows estimates of component deformations for each degree of freedom in the model.

The results of this analysis can be directly compared with relatively low uncertainties to the acceptance criteria without any modification required.

However, the calculated response can be very sensitive to the characteristics of the individual ground motion used as site-specific seismic input; therefore, a minimum of seven ground motion analyses are required[85], before the results of the individual time-histories are averaged to achieve a reliable distribution of structural response and final analysis results.

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[81] cf. Ibid., p. 67-70 AND cf. Adams, Scott Michael: Performance-Based Analysis of Steel Buildings. Special Concentric Braced Frame. [Diss. San Luis Obispo 2010: California Polytechnic State University, p. 5

[82] Federal Emergency Management Agency (ed.): Prestandard and Commentary for the Seismic Rehabilitation of Buildings. Prepared by ASCE. Washington: FEMA 356 2000

[83] Applied Technology Council (ed.): ATC-40. Seismic Evaluation and Retrofit of Concrete Buildings. Volume 1, California: Applied Technology Council 1996

[84] cf. Booth/Key, p. 67-68

[85] Adams: Performance-Based Analysis of Steel Buildings. Special Concentric Braced Frame, p. 6

2.3.2.3 Analytical Structure Models[86]

1D or 2D structural modelling will not provide comparable results like the 3D structural modelling. However, there are cases which justify the use of reduced (1D and 2D) simulations.

The 1D structural modelling is a very quick and simple assessment method for linear static analysis, which does not requires a computer. The structure is simplified as a cantilevered oscillator with a single degree of freedom. Adopting this system to consider higher vibration modes or deriving appropriate stiffness will be impossible or a not reasonable effort in comparison to easy computer-generated 2-or 3D models.

The next step up are the 2D models, but the output is also limited, as biaxial bending in columns and torsional response, is not captured with this simulation.

Therefore 3D models are recommended. In addition to the information about the structural stiffness (by general structural models for static gravity load only), information must be added on mass distribution; without it would be impossible to calculate periods, mode shapes and in turn inertial forces. To apply 3D models for non-linear analyses, information about the yield properties of the elements and materials must be added too.

2.3.3 Conclusion

This section gives a brief summary of the previous mentioned methods and should help to clarify which analysis method is appropriate. The main methods are defined by four terms, which are summarized as follows:

- → <u>linear:</u> considering material behaviour until yield strength is reached and it COULD consider strain effects due to mass and stiffness irregularities (torsion) as long as a 3D model is used, but it will may underestimate the response after yielding, because the less stiff side tends to yield first and would hence add to the eccentricity
- → <u>non-linear:</u> considering post-yield behaviour (ductility); a 3D model is mandatory for non-linear analysis; strain effects due to irregularities are appropriately considered
- → <u>static:</u> the dynamic, time dependent earthquake impact is reduced to a pattern of lateral forces with vertical distribution, which should represent this impact; idealised as SDOF – it take only the fundamental vibration mode of the model into account
- → <u>dynamic:</u> the earthquake impact is calculated from a response spectrum, which is based on measured time history plots of past earthquakes, whether applied in time domain or not; MDOF – consider higher vibration modes too (at least those which activate 90% of the building's mass)

[86] cf. Booth/Key, p. 78

In comparison to other procedures the linear static analysis is easily and quickly applicable and provides satisfying results for short buildings (up to 15 stories) with regular horizontal and vertical planes, where higher vibration mode effects are not significant.

Several prescriptive codes are providing defined criteria for the regularity in design, to limit the use of linear static analysis. Especially in combination with capacity design, this analysis is good enough for the majority of residential buildings with normal importance.

For critical buildings or facilities with major importance or high hazard risk, more detailed considerations are required and dynamic procedures are mandatory.

For the analysis of already existing properties, dynamic procedures are recommended as well.

For complex buildings with significant irregularities in elevation (sudden change in mass or stiffness with height) or plan (separation between centres of stiffness and mass at any level) a non-linear analysis is required to guarantee a reliable assessment.

Due to the hazard risk in case of damages and the irregularities in the mass distribution of the structure, the non-linear dynamic analysis is recommended for industrial construction facilities.

3 Performance-Based Design

3.1 Two Design Approaches for Seismic Design

There are two relevant approaches for the seismic design process. On one hand there is the common design approach based on prescriptive codes, prescribed by national or international laws, on the other hand, the performance-based design (PBD) approach, which has been arisen in the need of a more flexible and innovative design approach.

At first the "standard approach", with prescriptive codes is defined in the following sections, followed by the PBD approach.

The following statement gives a good explanation of the main difference between these two approaches:

> *A prescriptive code is like having the address, a map of the entire area marked with the route and clearly written turn-by-turn directions. A performance-based code is like having a map of the city with a choice of locations marked on it*[87]. - *Statement 1*

3.1.1 Definition: Design based on Prescriptive Codes

3.1.1.1 Main Characteristics of Design based on Prescriptive Codes

According to *Statement 1* above, the main characteristics of the design process based on descriptive codes is, that the whole "route" is marked and the "address/the goal" will be reached by clearly given directions.

It means that the prescribed step-by-step application of the codes must be exactly followed to achieve the goal.

The "goal" is defined hereby as the minimum required performance level which prevents major structural failure and guarantees safety to life and property.

This is ensured by regulations for the acceptable materials of construction, approved structural and nonstructural systems, setting minimum levels of strength and stiffness requirements and by showing details of how a building is being put together. These prescriptive regulations are stated in terms of fixed values, like precise allowable area and height or specified values for dead loads, snow loads etc.[88]

Advantages and disadvantages regarding this design approach are stated in *Section 3.1.3 "Prescriptive Codes vs. PBD" on page 47*.

[87] http://plumbingcoalition.org/faq.html: FAQ's Plumbing Coalition for Public Health's and Safety. Bev A. Potts [20.11.2015]

[88] cf. FEMA (ed.): Next-Generation Performance-Based Seismic Design Guidelines. Program Plan for New and Existing Buildings. Prepared by ATC Council. Washington: FEMA 445/2006, p. 1

3.1.1.2 Development of Prescriptive Code-based Procedures[89]

Earthquakes in the early 20th century enforced governments all over the world to develop regulations to provide minimum levels of strength and ductility in order to avoid a repetition of hazards causing injury or death, with the result that the first seismic specific prescriptive codes had been introduced to the construction industry. These codes had been periodically updated in order to represent the current State of Art.

3.1.2 Definition: Performance-Based Design

3.1.2.1 Main Characteristics of Performance-Based Design (PBD)

According to *Statement 1 on page 42*, the initiator of this "journey" is responsible for defining the "location – the goal" and the way of how to achieve the selected goal.

That means that the owner has to set up individual performance requirements/objectives, which the building's performance should meet. The way for developing design and construction details is not prescribed and can be chosen by the owner or his engineer, as long as it can be guaranteed that the stated required performance can be met. This guaranty is given at the end of an iterative performance assessment process, which is another main characteristic of this performance-based approach.

Advantages and Disadvantages regarding this design approach are stated in Section *3.1.3 "Prescriptive Codes vs. PBD" on page 47.*

3.1.2.2 Development of Performance-Based Procedures (USA)

The first reference for a performance-based approach can be found in Hammurabi's Code, wherein the Babylonian King Hammurabi (1795 to 1750 BC) stated the first performance objective for buildings: "A house should not collapse and kill anybody[90]."

For the next 3.742 years, there weren't any further entries in history regarding the PBD.

Some major earthquakes in the end of the 20th century showed that the adherence to prescriptive seismic codes won't save structures from damage and loss. The design process needed to be improved somehow[91].

3.1.2.2.1 First Generation[92]:

The initial set of procedures for PBSD was published in 1992 from US agency FEMA (Federal Emergency Management Agency) in the form of a program to reduce seismic hazards, at first only for existing buildings. FEMA Report 273: "Guidelines for the Seismic Rehabilitation of Buildings", followed by a SEAOC (Structural Engineers Association of California) publication in 1995 – "Performance-based Seismic Engineering of Buildings".

These documents outlined a design framework, introduced the concept of structural and nonstructural components, provided guidelines for nonlinear analysis techniques and defined levels of performance (collapse, collapse prevention, life safety, immediate occupancy and

[89] cf. FEMA 445, p. 2
[90] cf. *https://en.wikipedia.org/wiki/Performance-based_building_design*: Performance-based building design. [17.11.2015]
[91] cf. FEMA 445, p. 3
[92] cf. Ibid., p. 6

operational performance) and linked them to specific levels of earthquake hazards to create performance objectives.

3.1.2.2.2 Second Generation[93]:

The FEMA Report No. 356 – "Prestandard and Commentary for the Seismic Rehabilitation of Buildings", includes technical updates to the analytical requirements and acceptance criteria of the first generation, although intended for existing buildings, procedures are being extrapolated for use in performance-based design of new buildings. - (**State of Art)**

3.1.2.2.3 Need for Next-Generation Performance-Based Procedures[94]

The following three main issues show the need for the next-generation performance-based procedures.

a) Clarity of Communication

Problems appeared by communicating PBD issues to the owner. The stated performance objectives are mostly descriptive, describing quality and function, whichs leave room for interpretation. Therefore it is very difficult to verify required objectives exactly, which in turn led to contractual problems in the past.

To avoid this problem the descriptive objectives must be transformed into quantitative performance parameters (e.g. repair costs, time of occupancy interruption) more related to the decision-making needs, so that the user/client would really know and understand why and what he is requiring[95].

For example:

- Descriptive Performance Objective:
  Little or no damage for small, frequently occurring events
- Quantitative Performance Objective:
  The repair costs will be 800.000 € or less if a magnitude-4.7 earthquake occurs

In order to implement this quantitative aspect, advance procedures for estimating costs must be created, for both new and existing buildings.

b) Accuracy and Reliability of the Performance Assessment

The building response simulation must be improved as well as the analytical techniques to assess losses must be refined to get reliable tools.

The current assessment procedure is based on many assumptions like:

- uncertainties in the level of earthquake hazard
- quality of building's construction and building's condition when an earthquake happens
- actual strength of various materials and connection
- building occupancy and tolerance to operating in less than ideal conditions
- availability of contractors to conduct repairs following the earthquake

[93] cf. FEMA 445, p. 7
[94] cf. Ibid., p. 7-8
[95] *https://en.wikipedia.org/wiki/Performance-based_building_design*: Performance-based building design. [17.11.2015]

c) Concept Extensions for the Industrial Construction Sector[96]

The current PBD concept needs to be improved to fit to industrial construction. Due to its variety of involved disciplines, functions and requirements a framework for industrial PBD procedures has to be developed to clearly guide through the interdisciplinary building processes and regulate the cooperation and communication between process-, mechanical-, electrical and civil engineering and owner.

Another issue is the structure modelling within the current assessment procedure - the entire system should be modelled (foundation, primary and secondary superstructure and nonstructural components) for an accurate performance-verification.

Modifying the structural procedures to assess the performance based more on global response parameters, so that the response of individual components do not unnecessarily control the prediction of overall structural performance, would improve this process[97].

[96] cf. Uzunoglu, Timur/Saragoni, H. Rodolfo/Ansal, Atilla: Structural performance objectives in seismic design of industrial constructions and equipments. Vienna: VESSD 2013, p.6

[97] cf. FEMA 445, p. 8

3.1.2.3 The State of Art of Performance-Based Design in Austria

The German term for the performance-based design approach is “Konzept der leistungsorientierten bautechnischen Vorschriften“ and came up as a part of the uniform building regulations[98] for the nine provinces of Austria, commissioned in 2000. Burgenland, Vienna, Tyrol and Vorarlberg accepted these regulations in 2008, followed by Styria in 2011. The rest of the provinces only adopted them partially. These uniform OIB guidelines propose a two-stage concept whereas at first qualitative functional requirements are defined followed by quantitative technical requirements and allow either applying the performance-based or prescriptive based approach[99].

Another attempt on the performance-based approach can be found in the construction management sphere within the tender design. Since the BVergG[100] 2002 the client can decide whether he wants a tender with detailed services descriptions or a functional tender. A detailed tender describes the construction work in detail prescribing construction method, materials, dimensions and quantities resulting in a bill of quantities. A functional tender describes the functional, quality-orientated performance of the construction without specifying construction method, material etc.

The performance-based design approach it is not a common way in Austria. Most of the designs are done based on prescriptive codes and standard design procedures.

[98] OIB - “Österreichisches Institut für Bautechnik“
[99] cf. Mikulits, Rainer/Vogler Franz: Handbuch Bautechnikverordnung 2014. Burgenland, Kärnten, Oberösterreich, Steiermark, Tirol, Vorarlberg, Wien etc. Wien: Linde Verlag 2014, p. 1-6
[100] „Bundesvergabegesetz“ 2002 idF BGBl I 99/2002 mentioned in Heid, Stephan: Die funktionale Leistungsbeschreibung im Baubereich. In: RPA Heft 2-(2011), p. 69

3.1.3 Prescriptive Codes vs. PBD[101]

The following chart *[Figure 31]* shows the respective steps of both design approaches. Differences are in the first step regarding the performance objective and in the last one, the verification of the performance.

The PBD allows setting an individual performance objective, in contrary to the design with prescriptive codes, with predefined performance objectives. The second step – developing the preliminary design, is equal in both approaches and should be done by observing the provisions in *Section 2.2.1 "Preliminary Design" on page 24*.

The third step – the assessment of the performance, is the most important for both approaches, but the ways to obtain values for the buildings response or performance differ. Both ways are based on the assessment methods described in *Section 2.3 " The Calculation of the Structural Response" on page 37*, but the design process with prescriptive codes provides defined values for the seismic force; prescribes a certain method for the assessment and provides limitation-values for the buildings response. In contrast, the PBD process contains no restrictions for the performance assessment.

In comparison to the PBD process, an extra performance-verification step becomes no longer necessary by the design process with prescriptive codes, as the verification of the prescribed performance objective is part of the assessment.

Further explanations about each step can be found in the related *Sections 3.2 "Process: Performance-Based design" on page 49 and 5.2.1.1 "Process: Prescriptive Design according Eurocode 8" on page 80.*

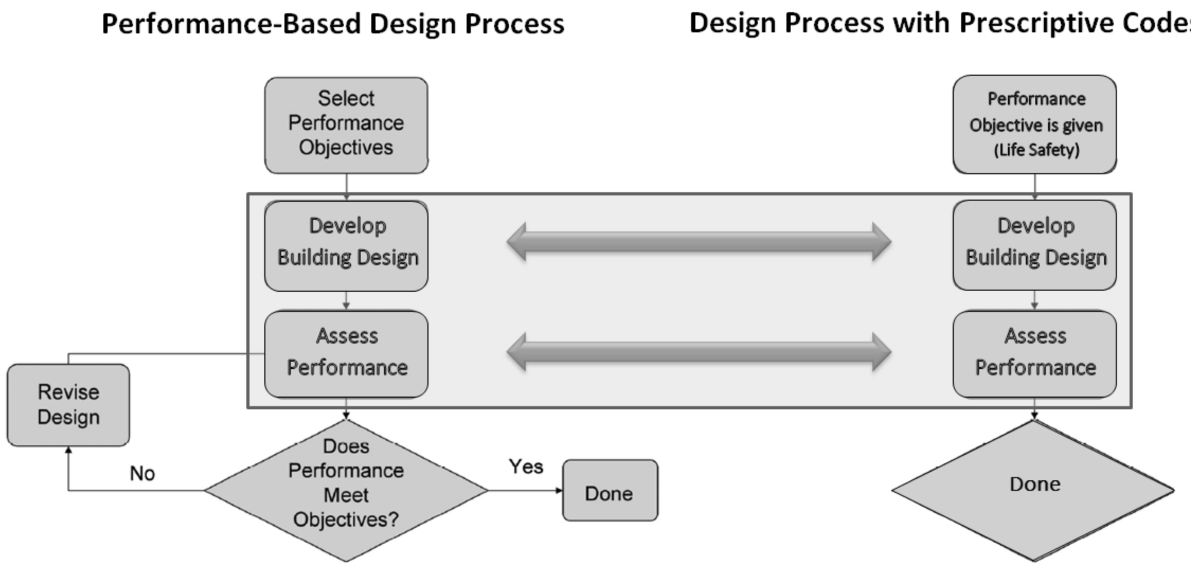

Figure 31 – performance-based design vs. design based on prescriptive codes

[101] cf. Spataro, Katie/Bjork, Marin/Masteller, Mark: Comparative Analysis of Prescriptive, Performancce-based and Outcome-based Energy Code Systems. Prepared by Cascadia Green Building Council. Alaska: AHFC 2011 AND FEMA 455 AND Uzunoglu/Saragoni/Ansal, Atilla: Structural performance objectives in seismic design of industrial constructions and equipments.

The advantages and disadvantages for both approaches are listed below.

3.1.3.1.1 Advantages - Design based on Prescriptive Codes

- ➔ familiar (common used framework, no specific skills needed as for modelling or analyses)
- ➔ simple (easy to follow, simple to verify adherence)
- ➔ clear (clearly lays out what is acceptable, clear description of measures)

3.1.3.1.2 Disadvantages – Design based on Prescriptive Codes

- ➔ hindering progress (detailed prescriptive codes to not allow alternative or flexible solutions, it hinders the exchange of building materials between countries, the design speed and the innovations in the building process)
- ➔ predefined performance level (objective is given by law - providing minimum performance level, no opportunity to reach a better performance level)
- ➔ actual performance is not assessed (does not utilize a whole building approach, no simulation or verification tools, therefore performance will most probably be in a noticeable range, better or worse than the minimum requirements anticipated by the code)
- ➔ optimistic (not required to test system once installed, assumes everything performs correctly, which is frequently not the case and it does not consider appropriately the uncertainties)

3.1.3.1.3 Advantages – Performance-Based Design

- ➔ More flexibility (not restricted to code-provided solutions to attain performance objectives, enhance innovations within the building process, allows application of new technologies)
- ➔ Individual performance level (possibility to define individual performance levels, provide guidance to make more informed decisions and clearly stated goals)
- ➔ Providing assessment methodology (Explicitly evaluates how a building is likely to perform and estimate potential losses and casualties)
- ➔ Verifying performance (verification tools are provided to evaluate various combinations of strategies, components and technologies and verify selected performance level – very important for critical facilities)
- ➔ Considering uncertainties (of the potential hazard and uncertainties in assessment of the actual building response)

3.1.3.1.4 Disadvantages – Performance-Based Design

- ➔ Incomplete (need for improvements – see Section 3.1.2.2.3 "Need for Next-Generation Performance-Based Procedures" on page 44)
- ➔ Expensive (requires special software, staff expertise in modelling and for the assessment)
- ➔ Optimistic (everything performs correctly which is frequently not the case)

3.1.4 Conclusion

The performance-based design approach is not meant to replace the design process based on prescriptive codes. As stated above, both approaches have advantages and disadvantages.

For each project it should be decided case-by-case whether the prescriptive or PBD procedures or a blending between both approaches is appropriate and more practical.

By designing simple buildings or using well proven technologies the use of prescriptive codes will probably result in a more effective, efficient, faster and less costly design process. Nevertheless, in some cases it may stifle changes and innovation.

Yet, it is not possible that facilities can be planned, procured, delivered, maintained, used and renovated only by using the current existing PBD procedure. Improvements and enhancements are needed to achieve that state of full applicability for the PBD approach[102].

3.2 Process: Performance-Based design

The previous section explains the principles, the development and some advantages and disadvantages of PBD. This section is a short abstract of the actual process, explained step-by-step, according *Figure 32*:

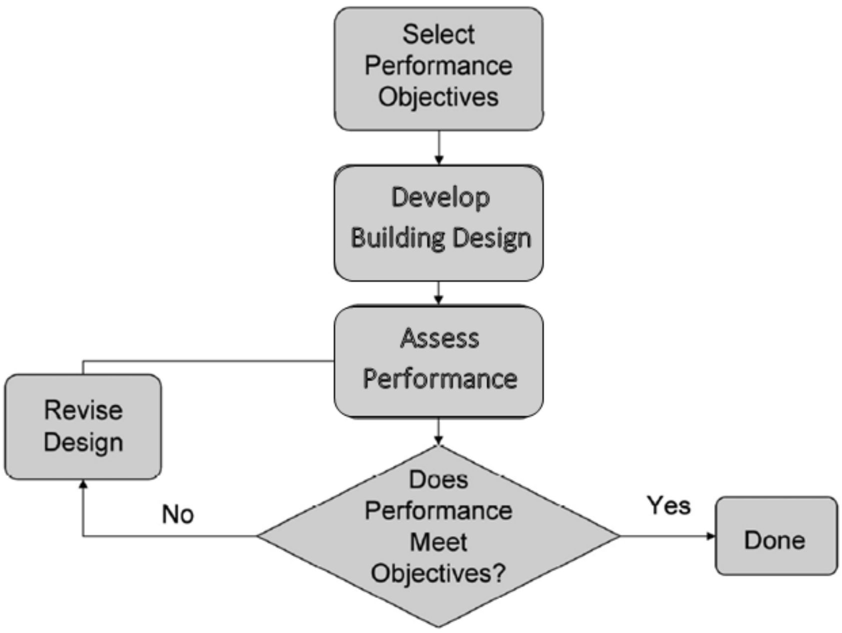

Figure 32 – performance-based design process[101]

According to *Section 3.1.2.1 "Main Characteristics of Performance-Based Design (PBD)" on page 43*, it is an iterative process starting by choosing specific and detailed performance objectives. After the requirements are defined the preliminary design development starts, and is followed by an assessment whether the design meets the required objectives. If that isn't the case, these last two steps are repeated until the required performance is achieved.

[102] cf. https://en.wikipedia.org/wiki/Performance-based_building_design: Performance-based building design. [17.11.2015]
[103] © FEMA 445, p 86

3.2.1 Select Performance Objectives

Setting individual and specific performance objectives is crucial for the PBD process. Beside, this step is the biggest advantage of the approach, due its ability to provide more flexibility and innovations within the design process; it is accompanied by a high risk of misinterpretations leading to serious communication and contractual problems, when stated imprecisely.

Definition of performance objectives: *"Performance objectives are statements of the acceptable risk of incurring specific levels of damage, and the consequential losses that occur as a result of this damage, at a specified level of seismic hazard, considering potential performance of both structural and nonstructural systems"*[104].

The damage and the consequential losses (repair-, downtime-, and replacements costs) need to be stated in quantitative terms not as single value, but as bands between upper and lower limits[105].

The performance objectives should be developed by a team of the decision making stakeholders, considering following questions[106]:

- ➔ What events are anticipated?
- ➔ What level of loss/damage is acceptable?
- ➔ How often might this happen?

Their functional user requirements, together with the determined earthquake hazard, form the acceptable level of performance. Like the user requirements will be more and more detailed as the project proceeds, the acceptable performance level should also considered as being dynamic, rather than static[107] *(might change during the design process)*.

But nevertheless the level should be carefully stated in precise quantitative and technical terms, at every stage of the project. Whereas carefully stated implies that it is easy and clearly measureable whether the required performance objectives can be met or not.

[104] cf. FEMA 445, p. 3
[105] cf. *https://en.wikipedia.org/wiki/Performance-based_building_design*
[106] cf. FEMA 445, p. 4
[107] cf. *https://en.wikipedia.org/wiki/Performance-based_building_design*

The FEMA next-generation performance-based seismic design guidelines recommend three different objective expressions[108]:

- Intensity-based performance objective:
  quantification of acceptable level of loss, given that a specific intensity of ground shaking is experienced (consequences if a shaking with a 475-year-mean-recurrence intensity occurs)
- Scenario-based performance objective:
  quantification of the acceptable level of loss, given that a specific earthquake event occurs (consequences if magnitude-7.0 earthquake occurs)
- Time-based performance objective:
  quantification of acceptable probability over a period of time that a given level of loss will be experienced or exceeded, considering all of the earthquakes that might affect the building in that time period and the probability of each (2% probability in 50 years that life loss will occur, annual damage repair not exceed 1% of replacement cost and one day of interruption by a return period of 100 years)

3.2.2 Develop Preliminary Building Design

When performance objectives are set, the preliminary design process starts. Provisions and recommendations for this task are given in Section 2.2.1 "Preliminary Design" on page 24.

3.2.3 Asses Performance

"Once performance objectives have been selected and a preliminary design is developed, it becomes necessary to assess the performance capability of the building design to determine if it meets the selected performance objectives"[109].

Therefore, the building's behaviour, the resulting damages and their accompanied financial or physical losses by a given seismic hazard, must be known. All these values must be linked to determine whether the performance objectives can be met or not.

More precisely, this is achieved by developing statistical relationships between four types of probability functions, termed: hazard functions, response functions, damage functions, and loss functions and by mathematically manipulating these functions to assess probable losses.[110]

[108] cf. FEMA 445, p. 18
[109] cf. Ibid., p. 86
[110] cf. Ibid., p. 20

Each step of that assessment process is described below: *[see Figure 33]*

1. Characterization of Ground Shaking Hazard:

The stakeholder determined the probable seismic hazard in first step – "Setting performance objectives". This, so far qualitatively described, seismic hazard needs to be transformed into realistic quantitative ground shaking functions and in turn in peak horizontal ground acceleration *see Section 2.1 "Representation of Ground Motion" and "Earthquake Response Spectra" on page 15 and 16*.

2. Analysis of the Structure:

The calculation of the building's response, due to different intensities of horizontal ground acceleration is determined by one of the described seismic analyses described *in Section 2.3 "* The Calculation of the Structural Response*" on page 37*.

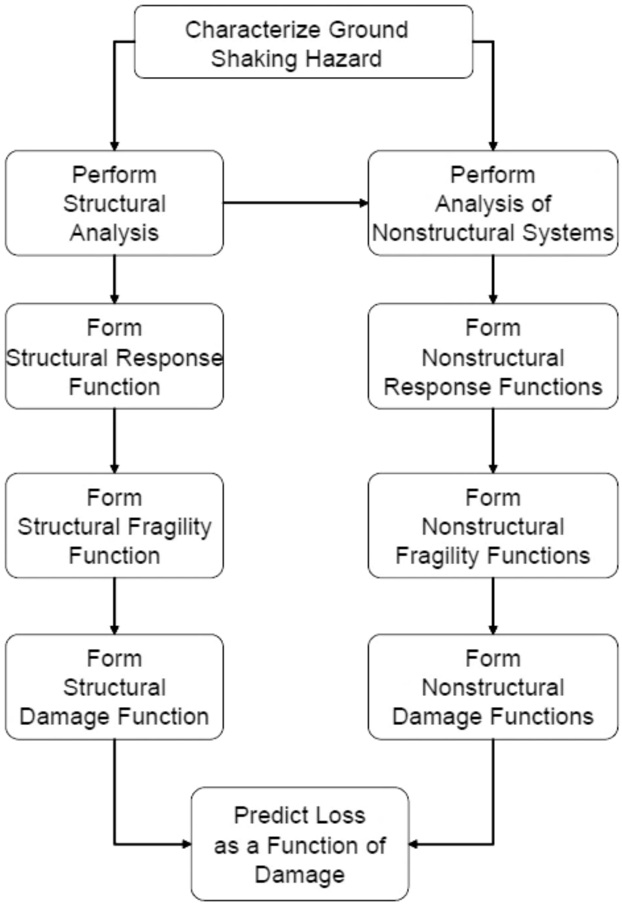

Figure 33 - performance assessment process[109]

The result is a response function showing the probabilities of various occurring building response for structure and nonstructural components, expressed in parameters obtained from structural analysis like storey drifts, member forces, deformations, joint plastic rotation demands, floor acceleration etc…[112]

3. Determine Probable Damage to Structure:

The next step is to assign probable damages to the given various levels of building's response as function of structural and nonstructural response, called fragility or damage function. Therefore, especially for the construction industry field, it is necessary to create logical system trees, considering the inter-relationship between the components and understand how failures of individual components will affect the system performance[113].

4. Determine Potential of financial and physical Losses:

The loss function shows the probability of various losses occurring, expressed in casualties, repair and replacement costs or occupancy interruption time, by a given damage level[114].

[111] FEMA 445, p. 87
[112] cf. Ibid., p. 21
[113] cf. Ibid., p. 101
[114] cf. Ibid.

5. Characterize Building Performance:

By evaluating the statistical relationships between these four functions it has become possible to verify if the stated objective is met, whether it is intensity-, scenario-, or time-based.

3.2.4 Check - Does Performance meet Objectives?

The design is completed, if the calculated performance meets the stated performance objectives; if not, the preliminary design has to be revised in an iterative process until the performance objectives are met. Nevertheless it is sometimes not possible to meet stated objectives at reasonable cost, which would make a downgrade modification of original objectives more appropriate[115].

[115] cf. FEMA 445, p. 21

4 Industrial Construction

This section focuses on the specific issues of the industrial construction.

In comparison to residential or office buildings the main characteristic of industrial construction is the capital-intensive equipment and special building contents. The equipment often represents more than 80 % of the total investment and in case of damage, besides this direct financial loss, expensive interruption of business costs may occur.[116].

Therefore special attention has to be paid on the equipment-protection against damages caused by seismic excitations. Furthermore, miscellaneous, hazardous materials are often handled or stored in such plants. Damages e.g. at a connection of a pipeline, containing these materials, would cause wide-ranging consequences for the surrounding environment. In such critical facilities it is essential to provide backup systems to ensure post-earthquake services and external supply of water, energy etc.[117]

Following section will give an overview of the characteristics of the industrial construction, point out the “State of Art and Practice” of the nonstructural-components-engineering-research and give recommendations to reach a higher level of nonstructural seismic performance.

4.1 Characteristics of the Industrial Construction

4.1.1 Definition

Industrial facilities often comprise various buildings like manufacturing halls, warehouses, assembling facilities, workshop buildings and logistics warehouses. Each of these buildings consists of a load-carrying primary structure and complex systems of nonstructural or non-building components.

This chapter is intended to address nonstructural issues relevant to industrial manufacturing, chemical factories, or power generation facilities.

[116] cf. Booth/Key, p. 225
[117] cf. Ibid.

4.1.2 Difference to General Building Construction

The industrial construction is different from the general building construction in the following aspects:

- **Separate Analyses of Primary Structure and Equipment**
  A distinction is made between the performance of the primary structural system and the performance of the nonstructural/equipment systems. The seismic performance of both must be analysed and certain levels of safety ensured.
- **Design Issues**
  The task of the primary structure; often steel frame constructions; is primarily to stabilize and protect the heart of the facility, the machineries needed for the production process. Normally there is no special architectural appearance required for the structure as it is often the case with residential and office buildings, but a lot of other special design issues must be taken into consideration for the industrial construction, like the effects of vibrating machineries on the structure or the severe irregularities in mass distribution (plan and elevation).
- **Focus on Equipment**
  In developed countries, like the USA and Austria, where seismic design has been implemented in local prescriptive codes, the seismic performance of structural systems reached a sufficient level so that catastrophic collapse can be prevented in almost every case. Now, the primary concerns of engineers have shifted to prevent expensive nonstructural damages. The nonstructural components often represent 80-90% of the overall construction costs[118] in industrial facilities and their damage inevitably results in significant economic losses for the owner. These losses are not limited to the cost of repairing the damaged components; they often include damage to other equipment and building contents plus the extensive loss-of-use costs associated with repair and restoration. In fact, these collateral losses accompanying equipment damages are often many times greater than the cost of repairing structural damages.[119]
- **High Risk Potential:**
  The impacts of damages in industrial facilities are associated with a high risk potential on the surrounding environment and society, especially when chemical factories and nuclear power plants are affected. The release of asbestos, toxic gases, chemical fluids, or other hazardous materials due to leaking of inadequately braced piping or damaged tanks can threaten the health of those located in a wide area around a damaged building[120]. Therefore the installation of redundant-, restraint-, and extinguishing systems is indispensable to restore safe conditions.

[118]cf. Applied Technology Council (ed.): ATC-69. Reducing the Risks of Nonstructural Earthquake Damage. State-of-the-Art and Practice Report. Cooperation with NEHRP, California: Applied Technology Council 2008, p. 2-1
[119]cf. ATC/SEAOC Jointventure: ATC-48. Part A: Overview of Component Types and Behavior. In: Build to resist Earthquakes. Briefing Paper 5 – Seismic Response of Nonstructural Components, Redwood City, California:1998
[120] cf. Ibid. ATC-48 AND Ibid. ATC-69

4.1.3 Categorisation of Equipment

The term equipment includes each component, its supports and attachments. Most of the equipment is permanently attached to the load-carrying primary structure. Beside its inability of carrying loads, equipment is required in every kind of structure to ensure comfort or/and function.

This section describes the categorisation of equipment.

4.1.3.1 Nonbuilding Structures, Nonstructural Components and Building Contents

It can be distinguished between nonbuilding structures (further distinction: nonbuilding structures similar to buildings and nonbuilding structures not similar to structures) nonstructural components (further distinction: architectural, mechanical, electrical components and the parts of the distribution systems/piping).

4.1.3.1.1 Nonbuilding Structures[121]

Nonbuilding structures are the exception of the definition above. Such structures are load-carrying, self-supporting structures other than buildings *[see Figure 34]*; further distinguished in nonbuilding structures with a dynamic response similar to buildings (pipe racks, electric power generation facilities and structural towers for tanks and vessels) and nonbuilding structures with a dynamic response not similar to buildings (detached tanks and vessels, stacks and chimneys). Nonbuilding structures are often large (>3m), heavy and field erected (except vessels and tanks). Their main function is to maintain structural stability.

[121] cf. Lindeburg/Baradar, p. 104

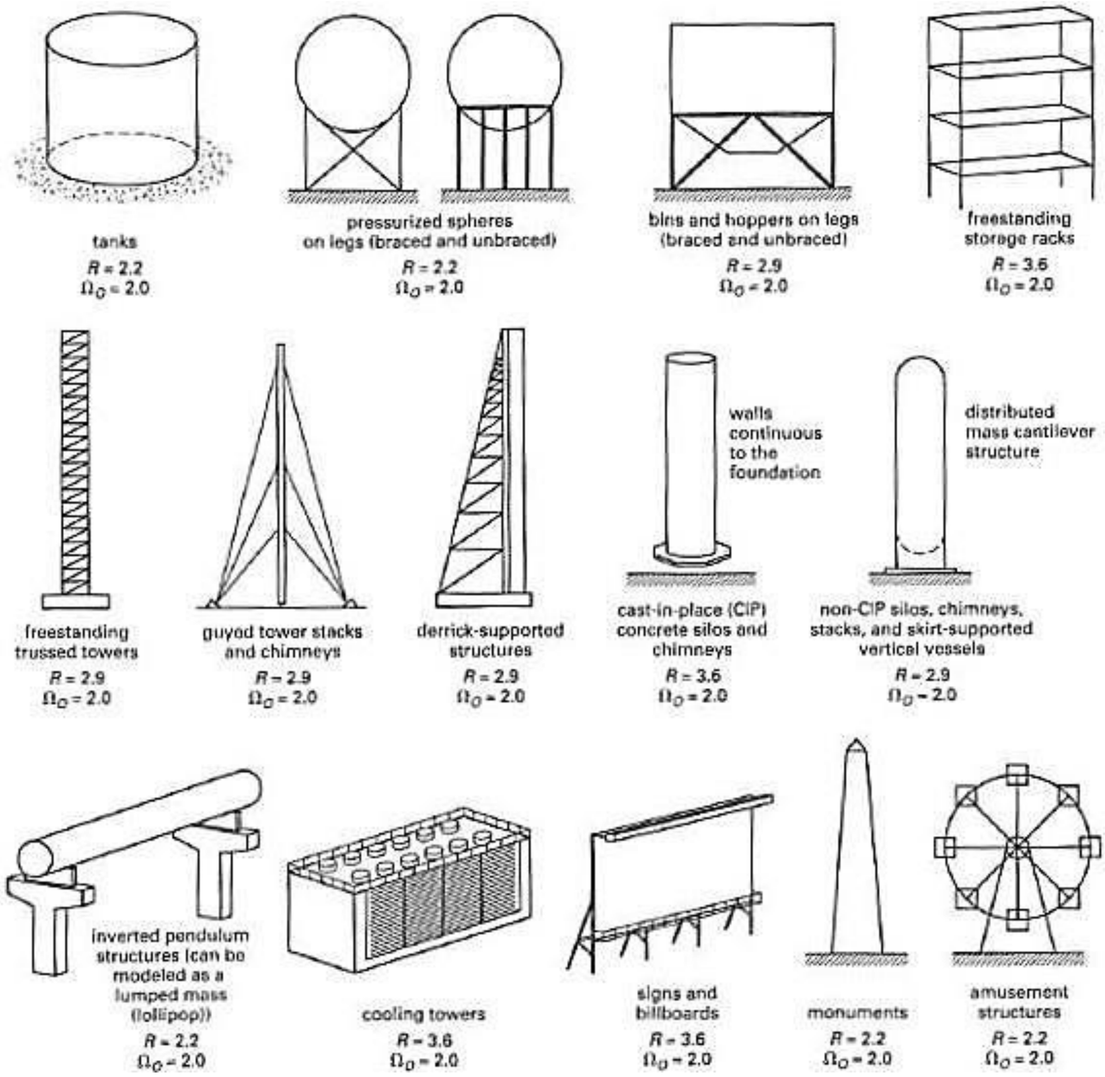

Figure 34 – nonbuilding stuctures[122]

4.1.3.1.2 Nonstructural Components[123]

Nonstructural components are normally small (<3m) and factory assembled, so they are often transported to site in one piece (except of cable trays, piping systems, ductwork). They are constructed to perform architectural, mechanical or electrical function purposes; e.g. a generator is designed to produce electric energy, without considering the load carrying task of these components due to probable lateral seismic forces. These components are considered as "black boxes" by the seismic engineer. Only anchorage and bracing can be designed to resist seismic loads.

4.1.3.1.2.1 Categorisation of Nonstructural Components based on Function

Based on function nonstructural components can be divided in architectural, mechanical, electrical components, the parts of the distribution systems/piping and building contents.

[122] Lindeburg/Baradar, p. 106
[123] cf. Ibid., p. 99

Examples of each category can be found below:

Architectural Components:

- suspended ceilings
- partitions or non-load carrying wall elements
- cantilever elements (unbraced or braced to structural frame below or above its center of mass)
- chimneys and stacks (roof-mounted or free standing)
- facade panels
- signs and billboards

Electrical Components:

- elevators
- generators
- batteries
- inverters
- transformers
- switch gear
- instrumentation cabinets
- lights and fixtures

Mechanical Components:

- HVAC systems parts
- manufacturing or process equipment
- engines
- turbines
- pumps
- compressors
- pressure vessels

Distribution System Components:

- piping and tubing
- ductwork
- electrical conduit
- mounted cable trays
- manufacturing or process conveyors
- plumbing

4.1.3.1.2.2 Categorisation of Nonstructural Components based on Reaction-Mechanism[124]

The earthquake excitation could cause damage to nonstructural components due to two distinct mechanisms, namely displacement and acceleration.

Displacement-sensitive components become damaged by distortions imposed on them by the structure, e.g. due to relative displacements on their supporting points. Examples for such components are cladding elements and line objects like pipes and ducts.

Two design strategies can be employed here. First option is to make the structure very stiff to limit the displacements (limits on storey drifts). Second option is to make the elements flexible enough to accommodate the imposed deflections themselves or at their points of attachment to the structure. In *Section 4.2.2 "Design of Displacement-Sensitive Elements" on page 63* a procedure is suggested to calculate the design relative displacement which the components need to withstand.

The other kind of components is rather compact and sensitive to the acceleration-impact and hence the inertia force, imposed on them by the structure. As result to this impact the components become detached from its support. Here, the design strategy is to make the anchorage sufficiently strong to prevent failure. Different methods to achieve this goal are described in *Section 4.2.3 "Design of Acceleration-Sensitive Elements" on page 64*.

[124] cf. Booth/Key, p. 226-227 AND cf. Holtshoppen, Britta: Beitrag zur Auslegung von Industrieanlagen auf seismische Belastungen. [Diss.] Aachen: 2009

4.1.3.1.3 Building Contents:

In comparison to the components mentioned above, building contents are typically movable rather than permanent build-in items. Each kind of furniture belongs to this description. Another difference between nonstructural components and building contents is that for nonstructural components the building owner is responsible, for building contents usually the building occupant[125].

4.2 Design of Nonstructural Components (State of Art)

Civil engineers do not design the nonstructural components. *(They will design nonbuilding structures, but not the nonstructural components!)* That is the task of mechanical engineers. The term „designing of nonstructural components“ means, from the civil engineering perspective, designing just the anchorage, mountings and supports for the nonstructural components.

In this section the main design principles and analysis methods for each of the two reaction-mechanisms (relative displacement and acceleration) of nonstructural components are described, starting with lessons learned from the past, important aspects engineers must consider within the design process and setting performance objectives or at least distinguish between different risk categories for nonstructural components.

The last two mentioned points together with the analysis task can be assumed to be an on-going engineering challenge as long as the development of new bracing and anchorage methods will be progressing and the complexity of restraining and interconnected systems will grow.

4.2.1 Design Principles

4.2.1.1 Data from Past Earthquakes[126]

"…generating statistics regarding the extent of losses due to nonstructural damage remains elusive[127]…"

There is too less information to generate statistical data regarding failures of nonstructurals leading in deaths, injuries, direct economic losses, repair costs or downtime.

But the direct damage to nonstructural items can be measured easily, which made it possible to determine certain components that have been repeatedly reported as damaged, like:

- small bore piping such as sprinkler distribution lines
- large bore piping
- pressure piping
- connections of piping to equipment
- ductwork
- suspended lighting
- roof mounted equipment

[125] cf. ATC-69, p. 2-1
[126] cf. Ibid., p. 3-4
[127] Ibid., p. 3-2

- spring isolated equipment
- elevators
- water heaters
- and vertical tanks

Future earthquakes might be able to provide the information necessary to validate the newest code requirements for nonstructural components to enhance the development of this research.

4.2.1.2 Failure Modes

This section deals with five principle failure modes of nonstructural components. The points engineers have to consider to avoid damages, stated in the following, are described in the *Section 4.2.1.3 "Engineering Considerations" on page 62.*

- internal damage to components due to shaking *(acceleration-sensitive components)*
- internal damage to components due to insufficient fixing *(acceleration-sensitive components)*
- interaction damages to other nonstructural components or the structure due to insufficient fixing *(acceleration-sensitive components)*
- damage to interconnected nonstructural components due to building deformations *(displacement-sensitive components)*
- damage nonstructural components crossing separations or joints between separate structures due to building deformations *(displacement-sensitive components)*

4.2.1.2.1 Internal Damage to Components due to Shaking

Sensitive nonstructural components can lose their functionality due to the earthquake excitation even though a proper anchorage is provided, which hold the components in place and no external damage is visible. The performance of the components itself under seismic loading must be considered within the design process. Especially critical components need post-earthquake operability to prevent further damages. To ensure post-earthquake operability components have to be tested on an earthquake shaking table for verification. *[See Section 4.2.3.2.3 "Testing of Components on a Shaking Table" on page 69.]*

4.2.1.2.2 Damage to Components due to Insufficient Fixing[128]

Components with insufficient fixing or anchorage are likely to change their position under seismic excitation. This position change lead to internal and external damages to the component and often in their loss of function. The inertia forces imposed to them exceed the holding capability of the fixing and lead to following position-changing-failure-modes: fall, slide, overturn or swing. The kind of mode occurring is depending on the location, size, shape, and orientation of component and the attachment to the building.

Components with insufficient mounting on ceilings can fall down and cause physical damage to the component itself or even hurt people inside the building.

Overturn endangered components are objects with a high center of gravity and a relatively small base such as electrical switch gear panels, storage racks, and interior partitions.

[128] cf. ATC-48 Briefing Paper 5

Items that are mounted on floors, roofs or platforms are primarily susceptible to sliding. However, that sliding movement can severe electrical and piping connections, causing fire hazards or water damage.

A suspended component with only vertical support, or less-than sufficient bracing, can swing like a pendulum, breaking piping or electrical connections and colliding with adjacent components. Due to the swinging motions the component can be disconnected from its own vertical fixings and could fall on occupants below.

4.2.1.2.3 Interaction Damages to other Nonstructural Components or the Structure due to Insufficient Fixings[129]

The construction space is limited. Various components are arranged close to each other on a relatively small area. In case of one of the position-changing-failure modes, mentioned in the previous section, it is most likely that adjacent components become damaged as well.

Most of the nonstructural components are to small and do not affect the seismic performance of the structural systems but some, very big and rigid components that are not isolated from the structural systems, can have an unintended influence on the structural system, often causing failure or collapse.

These interactions with other adjacent nonstructural components or the structural systems need to be avoided by ensuring proper fixings and checking for compatibility of deformations between the structural system and nonstructural components. Their effect on each other shall be considered within the design process.

4.2.1.2.4 Damage to Nonstructural Components due to Building Deformations

A sufficiently fixed component is subjected to deform the same amount as the structure does under seismic loads. These displacements can range from half a centimeter to several. Mountings will break, components will fall or slide and brittle materials like glass will crack under this load.

4.2.1.2.5 Damage Nonstructural Components crossing Separation or Joint between Separate Structures due to Building Deformations

Industrial plants comprise various buildings and structures. There are gaps between these buildings, which are important to allow the structures moving independently in case of a seismic event. Due to the interconnected production process supply pipelines or ducts often have to cross these building joints but their connections with the two different structures are often not flexible enough to compensate the occurring deformations.

[129] cf. ATC-69, p. 3-6

4.2.1.3 Engineering Considerations[130]

The previous sections described the possible failure modes to nonstructural components and the consequences of the interaction with the structural system. In this section points are stated, which have to be considered by the engineer in the design process to influence the extent or avoid damages to nonstructural components.

Considerations:

- the mass and the nonstructural components' dynamic characteristics
  The manufacture or mechanical engineer can provide this information.
- the location of the nonstructural components
  It makes a difference whether a heavy component is located at the basement or on the roof, as the acceleration increases with the building height. Furthermore their proximity to deforming structural or nonstructural components and joints, that may be crossed (flexible connection needed), need to be considered.
- the type of ground motion
  Engineers need to know the unique characteristics of the ground shaking at the site (e.g. high or low frequency motion, proximity to fault).
- the structural system of the building
  Nonstructural damages are caused by inter-story drift or floor-acceleration and these quantities depend on the structural response of the building (tall and flexible or short and stiff). Therefore attention must be paid to the selection of an appropriate structural system. *[See Section 2.2.3.2 " Lateral-Force-Resisting Structural* Systems*" on page 30]*.
- the anchorage of the nonstructural component
  The number, layout, type and location of bracing or anchorage must be carefully considered. The compatibility of the anchorage with the functional characteristics of the component being braced (e.g. rotating machinery - flexible anchorage) and the conditions of the structural elements used for anchorage (location of reinforcing bars in concrete used to anchor heavy items, condition of mortar etc…) are important aspects to be considered.
- the potential for secondary damage
  Special consideration must be paid to components containing hazardous materials. A damage, which cause the release of fluids, gases, toxins, asbestos, and other hazardous substances consequences in production downtime, plant evacuation or catastrophes like fire or explosions.

[130] cf. FEMA (ed.): FEMA E-74. Reducing the Risk of Nonstructural Earthquake Damage- A Practical Guide. 2012, p. 5-2 AND cf. ATC-48 Briefing Paper 5

4.2.2 Design of Displacement-Sensitive Elements[131]

4.2.2.1 Relevant Codes

4.2.2.1.1 For Analysis

- ÖNORM EN 1998-1: Eurocode 8: Design provisions for earthquake resistance of structures. General rules. Seismic actions and general requirements for structures. 2012
- ÖNORM EN 1998-4: Eurocode 8: Design provisions for earthquake resistance of structures. Silos, tanks and pipelines. 2008
- ASCE/SEI 31-03: Seismic Evaluation of Existing Buildings. American Society of Civil Engineers ASCE 2003
- ASCE 4-98: Seismic Analysis of Safety-Related Nuclear Structures. American Society of Civil Engineers ASCE 2000
- ASCE/SEI 7-10: Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers ASCE 2013, Chapter 13 and 15

4.2.2.1.2 For Execution

- FEMA 413: Installing Seismic Restraints for Electrical Equipment. Federal Emergency Management Agency 2004
- FEMA 414: Installing Seismic Restraints for Duct and Pipe. Federal Emergency Management Agency 2004
- FEMA E-74: Reducing the Risks of Nonstructural Earthquake Damage—A Practical Guide, Fourth Edition. Federal Emergency Management Agency 2012

4.2.2.2 Analysis and Assessment

A maximum relative displacement should be established between the points of attachment to limit distortion, which could damage the component. One way could be to ensure protection by abiding code-specified limits on storey drift like Eurocode 8, Part 1:

- brittle elements rigidly attached to structure → story drift < 0,5% of storey height
- ductile elements rigidly attached to structure → story drift < 0,75% of storey height
- elements with flexible fixing → story drift < 1% of storey height

But that way is permitted just for some standard architectural components, because it would underestimate the contribution of higher modes of vibrations to relative displacement, which may be significant at the upper levels of tall buildings.

Therefore, following procedure is suggested[132]:

a) derive deformations from response spectrum analysis, using Eurocode 8 design spectrum *[see Figure 45 "design spectrum for elastic analysis" on page 83]*

b) calculate design relative displacements by formula given in *Section 5.2.1.1.6 "Displacement Analysis" on page 85*

[131] cf. Booth /Key, p. 227

[132] Austrian Standards Institute (ed.): Eurocode 8: ÖNORM EN 1998-1. Teil 1: Grundlagen, Erbebeneinwirkung und Regeln für Hochbauten. Wien: Austrian Standards Institute 2013 p. 59, 4.3.4.

For extended nonstructural components (pipes, ducts and lifts) with multiple supports to the structure this practice is not capable of appropriately considering the more complex deformations. Explicit analysis of multiply-supported systems and guides to provide sufficiently flexibility is given in ASCE 4-98 (ASCE 2000). Useful design information, like introducing automatic shutdown valves for gas pipelines if ground acceleration exceeds a specific level, are given in ASCE/SEI 31-03 (ASCE 2003). Further practical provisions and limitations about the design of pipelines are given in EN 1998-4.

4.2.3 Design of Acceleration-Sensitive Elements[133]

4.2.3.1 Relevant Codes

4.2.3.1.1 For Analysis

- ÖNORM EN 1998-1: Eurocode 8: Design provisions for earthquake resistance of structures. General rules. Seismic actions and general requirements for structures. 2012
- ÖNORM EN 1998-4: Eurocode 8: Design provisions for earthquake resistance of structures. Silos, tanks and pipelines. 2008
- ASCE/SEI 7-10: Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers ASCE 2013, Chapter 13 and 15
- ASCE/SEI 31-03: Seismic Evaluation of Existing Buildings. American Society of Civil Engineers ASCE 2003

4.2.3.1.2 For Execution

- FEMA 412: Installing Seismic Restraints for Mechanical Equipment. Federal Emergency Management Agency 2004
- FEMA E-74: Reducing the Risks of Nonstructural Earthquake Damage—A Practical Guide, Fourth Edition. Federal Emergency Management Agency 2012

4.2.3.2 Analysis and Assessment

There are four methods for justification. The first method is appropriate solely for simple acceleration-sensitive components which can be approximated by a single degree of freedom system. For this purpose simplified formulas from several descriptive guidelines can be used to calculate the force required to attach nonstructural components. Some examples can be found in the following section.

More complex cases or critical components need more sophisticated methods like the analysis using the floor response spectra, testing of components directly on a shaking table or using an experience database for qualifying the components.

4.2.3.2.1 Analysis of Simple Acceleration-Sensitive Components

According previous section, several international prescriptive codes are providing formulas for the analysis of simple acceleration-sensitive components. This section refers to the doctoral thesis (Aachen 2009) of Dr.-Ing. Britta Holtschoppen, wherein she investigated the

[133] cf. Booth /Key, p. 228 AND cf. Holtshoppen: Beitrag zur Auslegung von Industrieanlagen auf seismische Belastungen.

development and compared the results of different international formulas. Some examples are given below:

ASCE 7-05 und IBC 2006:

$$F_P = \frac{0.4 * S_{DS} * a_p * W_p}{R_p/I_p} *)1 + 2 * \frac{z}{H}$$

Equation 3 – ASCE 7-05 nonstructural analysis[134]

F_p...........horizontal seismic force, acting at the centre of mass of the nonstructural

S_{DS} ....plateau value of the elastic response spectrum - 5% damping

a_p...........magnification factor acc. ASCE 7-05

W_p ....weight of the secondary element or equipment

I_p............Importance factor of the element acc. ASCE 7-05

R_p ....response modification factor of the element acc. ASCE 7-05

z ....height of the nonstructural element above the level of application of the seismic action

H ....building height from the foundation or from the top of a rigid basement

Eurocode 8:

$$F_a = \frac{S_a * W_a * \gamma_a}{q_a}$$

Equation 4 – EC8 nonstructural analysis[135]

F_p...........horizontal seismic force, acting at the centre of mass of the nonstructural

S_a...........seismic coefficient, defines difference between peak acceleration on the ground and the place of the component - determined by equation 5

W_a ....weight of the secondary element or equipment

γ_a ....importance factor of the element acc. EC 8, 4.3.5.3; 1 for most items, 1.5 for critical items

q_a...........behaviour factor of the element acc. EC8 table 4.4; 1 to 2

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

Equation 5 – EC 8 seismic coefficient for nonstructrual analysis [136]

α............ ratio of the design ground acceleration on type A ground, a_g, to the acceleration of gravity g

S ....soil factor; 1 = hard ground, 1.8 soft ground

T_a...........fundamental vibration period of the nonstructural element

T_1...........fundamental vibration period of the building in the relevant direction

[134]ASCE/SEI 7-10: Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers 2006, p. 144

[135]Eurocode 8: ÖNORM EN 1998-1, p. 60, 4.3.5.2

[136] Ibid.

z ....height of the nonstructural element above the level of application of the seismic action

H ....building height from the foundation or from the top of a rigid basement

NCh2369.Of2003[137]:

The Chilean Standard provides three different equations for following cases:

- secondary element is included in the modelling
- secondary element is not included in modelling, except for its mass
- no modal dynamic analysis has been carried out - secondary element is not included in modelling

Last one is presented below:

$$F_p = \frac{0.7 * a_k * K_p}{R_p} * P_p < P_p$$

Equation 6 – NCh2369 nonstructural analysis[138]

a_k...........acceleration at level *k* on which the secondary element or equipment is mounted - determined by equation 9

K_p...........must be defined by means of one of the two following equations *[7 and 8]*:

$$K_p = 2.2$$

Equation 7 – NCh2369 coefficient K_p value 1

$$K_p = 0.5 + \frac{0.5}{\sqrt{(1-\beta^2)^2 + (0.3 * \beta)^2}}$$

Equation 8 – NCh2369 coefficient K_p value 2

where:

β = 1 ....for $0.8\, T^* \leq T_p \leq 1.1 T^*$
β = 1.25 (T_p/T^*) ..for $T_p < 0.8\, T^*$
β = 0.91 (T_p/T^*)...for $T_p > 1.1\, T^*$

where:

T_p = Natural period of the fundamental vibration mode of the secondary element including its anchorage system and T is the period of the mode with the highest equivalent translational mass of the structure in the direction in which the secondary element may enter in resonance. The determination of β requires that the value of T* be over 0.06 s.*

R_p ....response modification factor of the element acc. NCh2369, table 7.1 ASCE 7-05

P_p...........weight of the secondary element or equipment

$$a_k = \frac{A_0}{g}(1 + 3 * \frac{Z_k}{H})$$

Equation 9 – NCh2369 acceleration at level of the equipment

A_0 ....maximum effective acceleration as defined in NCh2369 5.3.3.

Z_k...........height of level *k* above the base level

[137] The Chilean Code was not included in Holtshoppen's investigations
[138] NCh2369.Of2003, p. 51, 7.2.2

Britta Holtshoppen's Approach:

Beside this procedure of NCh2369 - these formulas have been developed for the general building constructions and its contents without considering the specific requirements and circumstances given in the industrial construction industry. Furthermore most of these formulas assume a linear increasing stress (with only the first mode of vibration considered) on the nonstructural component over the building height. Therefore and due to differences in the correction factors, the results varying a lot used in the different international formulas[139]. *[Figure 35 shows the differences in the height-depending extent of the ground acceleration in different formulas.]*

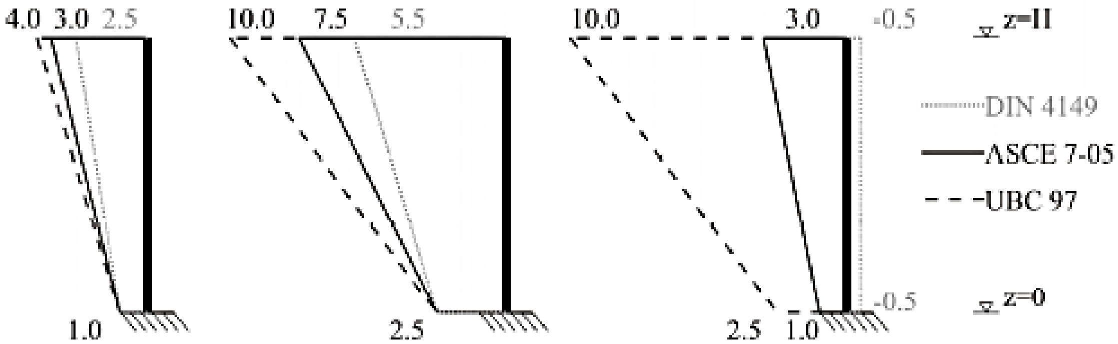

Figure 35 – differences in the height-depending extent of the ground acceleration in different formulas[140]

Dr.-Ing. Britta Holtschoppen analysed such formulas in her doctoral thesis and developed a new approach, by interpreting the characteristics of floor spectra to account also higher vibration modes, without increasing the analysing-effort.

Depending on the ratio of the fundamental vibration period of the nonstructural element T_a and the fundamental vibration period of the building in the relevant direction (first mode) T_1, she developed three different formulas[141]:

$T_a / T_1 > 2.5$:

$$F_a = m_a * 0.3 * S_{a,max} * \gamma_a$$

Equation 10 – "Mindestbemessungskraft" acc. Holtshoppen

m_a..........weight of the secondary element or equipment

$S_{a,max}$.......plateau value of the elastic response spectrum [m/s²]

γ_a...........importance factor of the element e.g. acc. EC 8 4.3.5.3; 1 for most items, 1.5 for critical items

$2.5 > T_a / T_1 > 0.6$:

$$F_a = m_a * S_{a,max} * \left(1 + A_{a.linear} * \frac{z}{H}\right) * \frac{1}{A_{1.linear} * T_1} * \gamma_a$$

Equation 11 – "Linearer Ansatz" acc. Holtshoppen

139 cf. Holtshoppen: Beitrag zur Auslegung von Industrieanlagen auf seismische Belastungen, p. 81
140 Ibid.
141 cf. Ibid., p. 89-92

$A_{a.linear}$ .....dynamic magnification factor, to take resonance effects with first mode into account, *acc. Figure 36* - x-axis shows the T_a/ T_1 ratio

$A_{1.linear}$ .....scaling factor for accounting the fundamental structural period acc. the structural systems type; 5.0 for standard steel frame constructions

z,H,γ_a ......already mentioned before

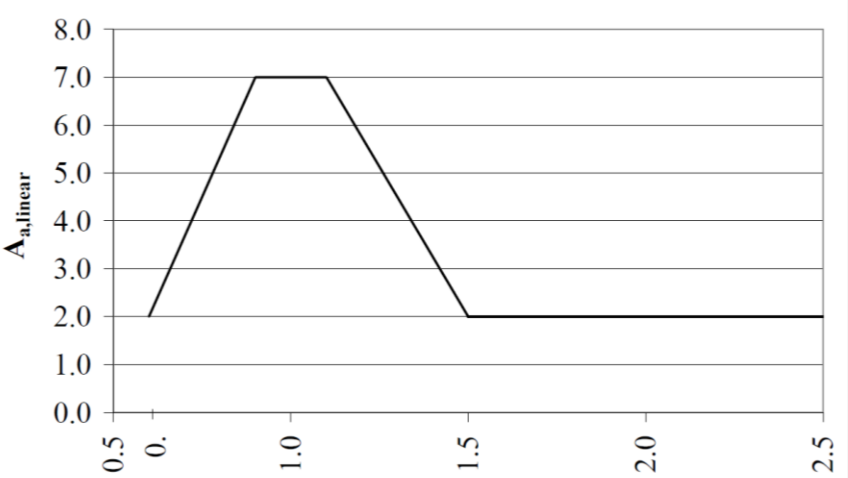

Figure 36 – dynamic magnification factor [142]

0.6 > T_a/ T_1:

$$F_a = m_a * a_i * \gamma_a * A_{a,modal}$$

Equation 12 – "Modaler Ansatz" acc. Holtshoppen

a_i ....floor acceleration at the point where nonstructural components are attached to the structure; determined by a foregoing floor-response-spectra-analysis of the structural system[143]

$A_{a.modal}$ ....dynamic magnification factor, to take resonance effects with higher mode into account; 2.5 for standard steel frame constructions

4.2.3.2.2 Analysis of Acceleration-Sensitive Components using 'Floor Response Spectra'[144]

Other ways to consider higher modes of vibration would be to include the nonstructural components in the structural design and assessment process, as well as in the 3D model of the whole structure. But in practice the attributes of the nonstructural components are often not known in detail at the time of the structural analysis. Therefore a method, by which the analysis of the main structure can be done separately from the analysis of the nonstructural components, is more reasonable.

"The solution here is to use the structural analysis to produce 'floor response spectra' at the points where the nonstructural components are expected to be attached".

This method corresponds to Britta Holtshoppen's procedure *(for 0.6 > T_a/ T_1)*, whereby a time history analysis is needed to be carried out on the structural system, which in turn provides the time history of motions (of course, amplitude as well as frequency content is included) at the considered point.

[142] Holtshoppen, p. 90
[143] Procedure described in Holtshoppen, p. 90-91
[144] cf. Booth /Key, p. 229

The disadvantage of this method is, that interaction of the nonstructural and structural response is not considered. Components with greater mass could influence the structural response (e.g. columns) which in turn again could have influence on nonstructurals. Or the natural periods of the structure and the nonstructural component are similar, which could consequence in the resonance-effect *[see Section 2.1.13.3 “Resonance Effect” on page 22*].

4.2.3.2.3 Testing of Components on a Shaking Table[145]

Previous mentioned methods ensure the appropriate fixing on the nonstructural to the structure. But that does not mean that the function of the component itself is provided during and after the earthquake. To ensure functionality the items must be placed on an earthquake shaking table, subjected to the suitable floor-response-spectra corresponding motions. This method is used for safety critical plant items.

4.2.3.2.4 Qualifying Components from an ‘Experience Database’[146]

With reason that the shaking test is too expensive, the nuclear power industry in the USA started to develop a database including the response experience of different components (specialised ones for the nuclear industry as well as pumps, generators and other standard items needed in every plant) in previous earthquakes. Unfortunately this database is not open for public, only for subscribing members. Similar information for the public can be found in ASCE/SEI 31-03 Seismic Evaluation of Existing Buildings.

[145] cf. Booth /Key, p. 230
[146] cf. Ibid.

4.2.4 Recommendations and on-going Research

The first recommendation is to show attention to the careful assignment of responsibilities for nonstructural components, because a lot of disciplines have to cooperate for ensuring a proper performance.

Owners, design professionals including architects, mechanical engineers, electrical engineers, structural engineers and other specialty engineers who may be specifying equipment on a project, general contractors, subcontractors including plumbing subcontractors, mechanical subcontractors, electrical subcontractors, and the range of subcontractors associated with ceilings, interior partitions and exterior cladding, material and equipment supplier, plan reviewers and construction inspectors – all these parties play a role in ensuring protection of the nonstructural components. The quality of their works, communication and agreements defines the level of protection and makes the difference between post-earthquake operability and the need to evacuate; or between protection of life safety and loss of life.

The main task is to coordinate the numerous parties with overlapping responsibilities and different own interests and working together for a common purpose. Therefore clear responsibility must be assigned for each component throughout the whole construction process (design, peer review, plan review, installation, observation, inspection). E.g. following questions must be answered:

- Who is responsible for the design of which types of components and determine which items require seismic bracing?
- Who provides oversight for the design of the many, potentially interconnected nonstructural items?
- Who controls if the design solutions are consistent with the chosen performance objectives?
- Who inspects proper bracing and anchorage of the components?

The use of tools like a project-specific list of all nonstructural components with associated responsibilities and the seismic code block (regulation for drawing review and permits) are recommended. These and other tools can be found on following website:

(http://www.stlouisco.com/YourGovernment/CountyDepartments/PublicWorks/Documents/PublicNotices/SeismicNotices) [02.02.2016]

In addition to the first recommendation (responsibilities for each components and its belonging tasks during the whole design and construction process) following actions should be taken to ensure protection of the nonstructural components:

- applying performance-based earthquake engineering methods – setting performance objectives for each components
- identification of elements or areas that are most critical to operation
- definition of damage states (intensity measures and demand parameters) for predicting damage to nonstructural components and systems
- determining what backup systems are required
- developing design of nonstructural components (engineering considerations)
- checking - is post-earthquake operability ensured, can performance objectives be met?

Hereby, the ideal procedure has been described. Point a, b, c, d, and f are not yet fully developed to be recommended as standard procedures. Until now the effort to fulfil these tasks properly is not economical (besides in critical facilities). Too less information about nonstructural performance from past earthquakes is available and therefore expensive tasks like pre-installation seismic qualification testing, special seismic design calculations and details, rigorous design review, construction inspection, and in-place testing need to be carried out to verify the nonstructural performance.

Because of the reasons mentioned above, recent and on-going research-projects like Phase 4 of the FEMA-445 Next-Generation of Performance-based Seismic Design Guidelines focus on following points[147]:

- providing standardized checklists, standard design specifications for seismic protection of nonstructurals: including weight, center of gravity, dimensions, recommended anchorage details, and fragilities for nonstructural components and contents
- develop performance-based procedure for the design of nonstructurals
- fragility functions for nonstructural components and systems relative to the damage states identified
- loss functions for nonstructural components and systems
- developing standard procedures for quantifying the performance capability (fragility and loss functions) for nonstructural components and systems, including testing protocols
- support the development of testing and performance data
- develop standard contract language, covering the responsibilities associated with the design, installation, and inspection of nonstructural components, should be incorporated into standard contracts templates of AIA or FIDIC

[147] cf. ATC-69, p. 9-1

5 Case Study A – International Differences in Seismic Design

5.1 Comparison of Risk

5.1.1 Risk in Austria[148]

In Austria, the seismic protection of buildings is not a big issue. The aspect must be considered just for public or critical buildings.

There are 30 – 60 earthquakes a year, perceived by the Austrian population. Most of them have magnitudes between 4.0 and 5.0, which correspond to the intensity-scale V or IV. For the definition of magnitudes and intensity scale see *Section 2.1.3 "Intensity and Magnitude Scale" on page 14.*

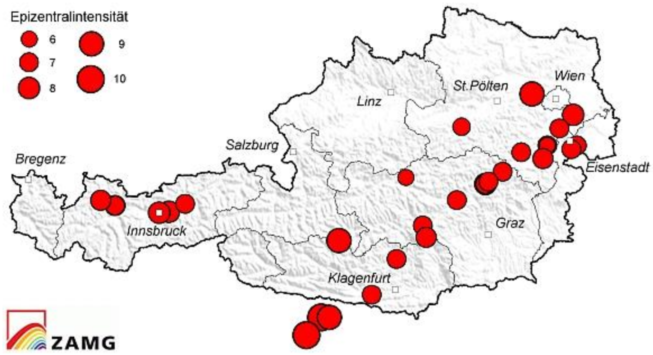

Figure 37 - maximum earthquake intensities in Austria

In a very general way, one could say that every three years an earthquake with fewer damages to buildings will likely happen, every 15-30 years one with moderate damages and every 75-100 years one with severe damages may emerge.

The seismic hazard in Austria is concentrated on four specific spots – the "Mur-Mürztal" fault, the "Inntal" fault and the "Lavantal" fault and the Vienna basin *[see Figure 37]*, where the last strong earthquake happened in 1972 resulted in more than 800 fire brigade operations in Vienna. Around 5 million inhabitants live in the affected area.

Beside the likelihood of other natural disasters (floods, avalanches or storms) occurring is much higher than earthquakes, but the cost risk, following a strong earthquake, is in no relation compared to the other disasters. That fact together with the 5 million affected persons, form the risk in Austria, which actually should not be underestimated.

Following *Figure 38* shows the seismic zones and related ground accelerations in Austria, according EC 8 (ÖNORM EN 1998-1).

[148] cf. https://www.zamg.ac.at/cms/de/geophysik/erdbeben/erdbeben-in-oesterreich/uebersicht_neu: Erdbeben in Österreich – Übersicht. Zentralanstalt für Meteorologie und Geodynamik. [19.11.2015]
[149] © ZAMG Geophysik

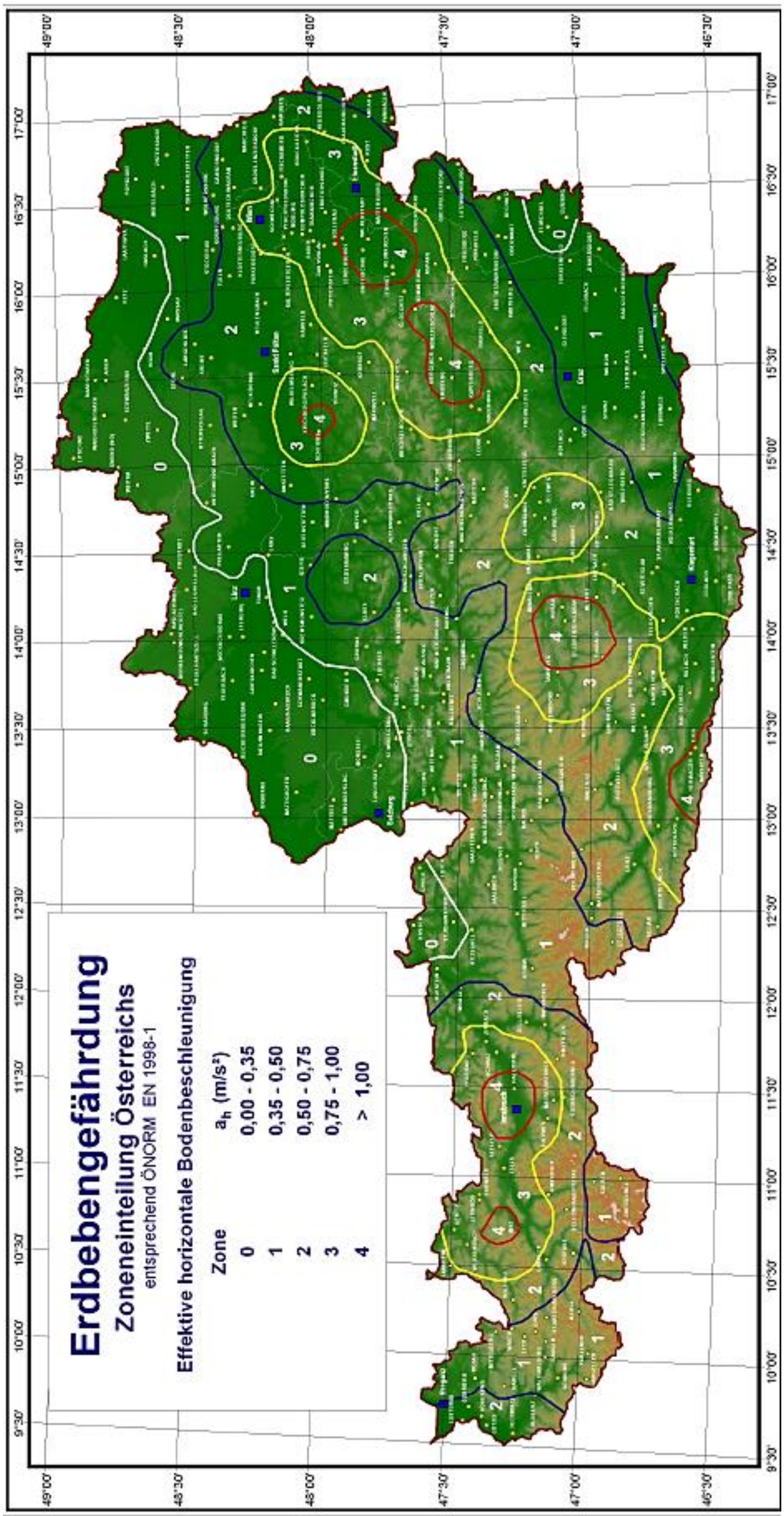

Figure 38 - hazard zones in Austria acc. ÖNORM EN 1998-1. © ZAMG Geophysik[150]

[150] https://www.zamg.ac.at/cms/de/images/geophysik/erdbebengefaehrdungszone-von-oesterreich-nach-oenorm-en-1998-1.-c-zamg-geophysik

5.1.2 Risk in the USA (emphasis on California)

The West Coast of the North American plate is part of the pacific Ring of Fire, which is a string of volcanoes and sites of seismic activity, or earthquakes, around the edges of the Pacific Ocean *[see Figure 39]*. Roughly 90% of all earthquakes occur along the Ring of Fire, and the ring is dotted with 75% of all active volcanoes on Earth[151]. In almost every part of the ring of fire major earthquakes occurred within the last 50 years, except for the fault line next to North America. That is one of the reasons why, the 37 million inhabitants of CA are waiting and preparing for the “Big One”. Geologists predict the probability of a severe earthquake (magnitude of 6.7 or higher) occurring in the next 20 years with more than 90%.[152]

Figure 39 – the pacific ring of fire[153]

The US Geological Survey (USGS) institution provides an updated earthquake map of recent events. Eleven earthquakes have been recorded in California so far this year[154] with magnitudes from 3.2 to 5.7[155] mostly along the San Andreas Fault system. *[see Figure 40]*

This fault forms the 1300 km (810 miles) long tectonic boundary between the Pacific and the North American Plate. These plates are moving slowly into opposite directions (northwest-southwest) for about 33-37 millimeters a year. Seismologists discovered that the San Andreas Fault consistently produces a magnitude 6.0 earthquake approximately once every 22 years. A study in 2006 confirmed that the stress level in the fault is constantly increasing. This regularity is another sign for the “Big One” in the next years[156].

[151] cf. *http://education.nationalgeographic.org/encyclopedia/ring-fire/*: Ring of Fire. National Geographic Society [24.11.2015]
[152] cf. *http://www.spiegel.de/wissenschaft/natur/kalifornien-banges-warten-auf-the-big-one-a-750946.html*: Banges Warten auf „The Big One“. SPIEGEL ONLINE. [15.03.2011]
[153] *http://news.bbcimg.co.uk/media/images/51640000/gif/_51640840_ring_of_fire2011.gif*
[154] This year = 2015
[155] cf. *http://earthquake.usgs.gov/earthquakes/eqinthenews/*: Significant Earthquake Archive. US Geological Survey [23.10.2013]
[156] cf. *https://en.wikipedia.org/wiki/San_Andreas_Fault*: San Andreas Fault. Wikipedia. [18.11.2015]

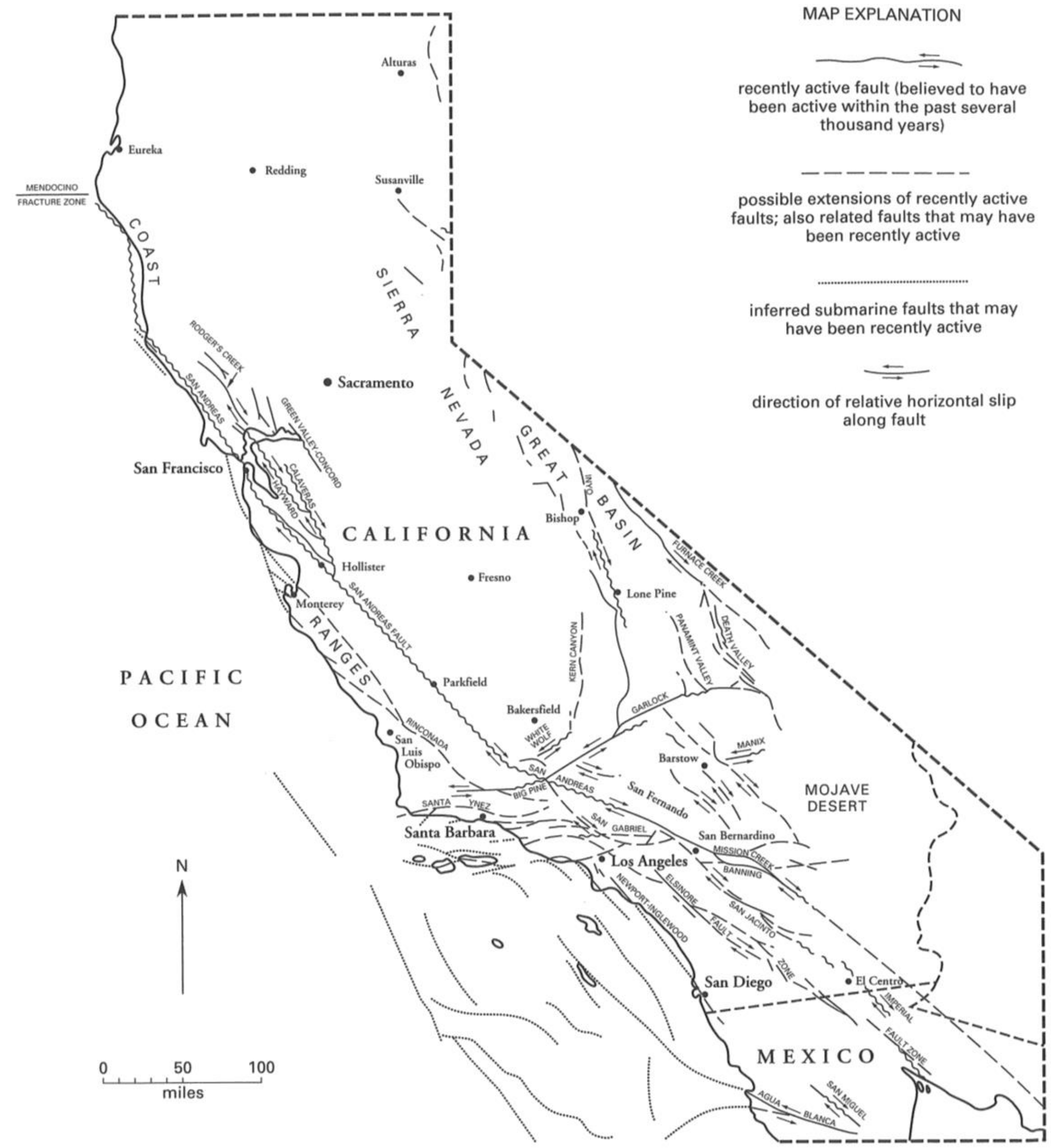

Figure 40 - active California faults[157]

David Schwartz, a seismologist of USGS, predicts an economic loss of more than 200 billion dollars for the US government, following a magnitude 7.0 earthquake. Ten thousands of older buildings are not built earthquake-resistant, a fifth of California's hospitals are structurally endangered and may collapse during such a seismic event and it is likely that the residents have to do without water and electricity for weeks[158].

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[157] Lindeburg/Baradar, p.6

[158] cf. *http://www.3sat.de/page/?source=/nano/umwelt/152833/index.html*: Auf unsicherem Boden – Kernkraftwerke stehen am San-Andreas-Graben. 3sat.online [17.03.2011]

The US government was forced to take action and the answer was the National Earthquake Hazards Reduction Program[159] with four basic goals[160], described as followed:

1. develop effective practices and policies for earthquake loss reduction and accelerate their implementation
2. improve techniques for reducing earthquake vulnerabilities of facilities and systems
3. improve earthquake hazards identification and risk assessment methods, and their use
4. improve the understanding of earthquakes and their effects

The USGS, one of the four agencies contributing the acomplishment of the program- goals, provides periodically updated national seismic hazard maps, available for building period of 0.2s, 1s or peak ground acceleration, either for 2 or 10% probability of exceedance.
[see Figures 41 and 42]

Figure 41 - 2014 Seismic Hazard Map - 2% probability of exceedance in 50 years of peak ground acceleration[158]

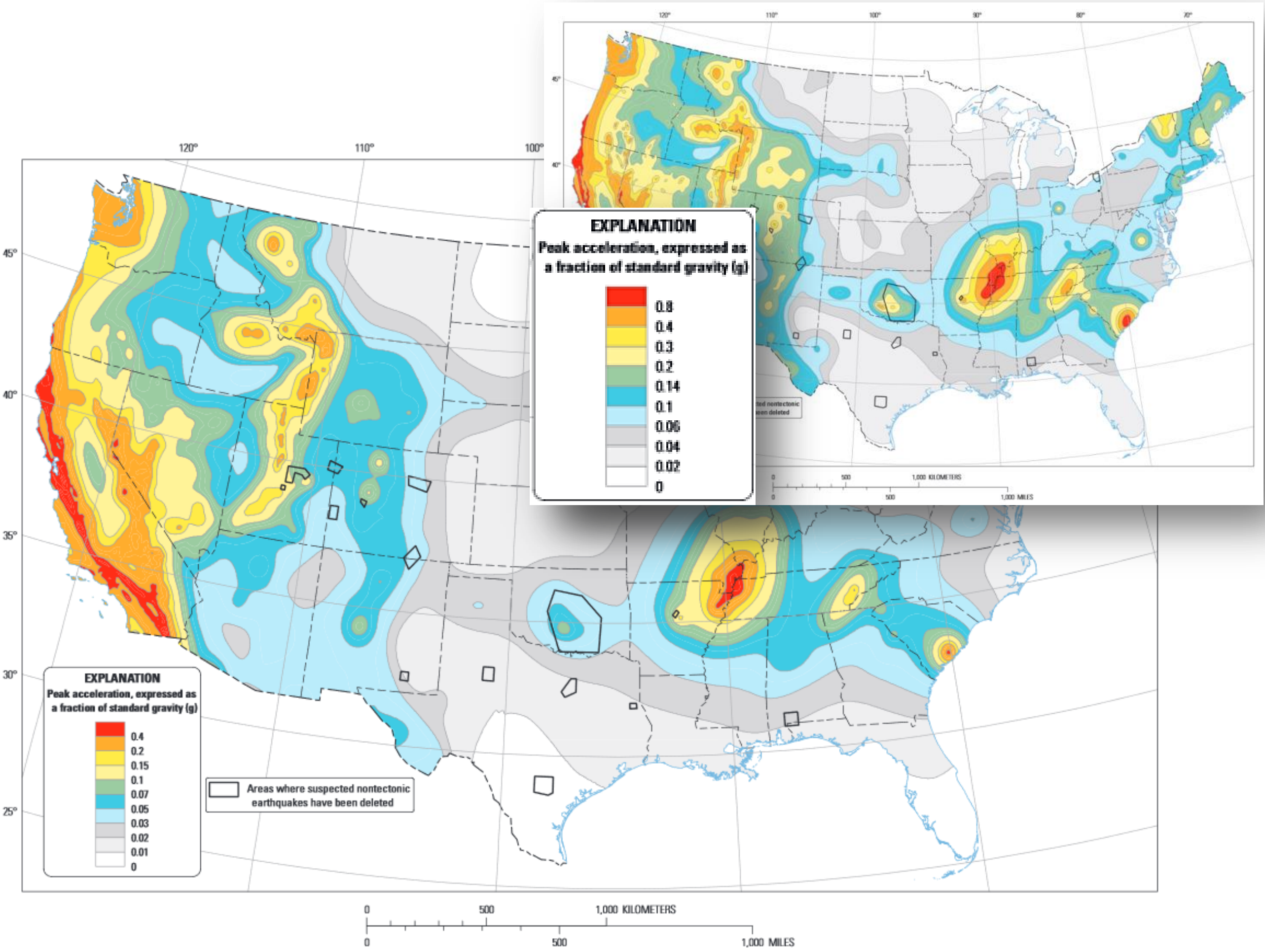

Figure 42 – 2014 Seismic Hazard Map - 10% probability of exceedance in 50 years of peak ground acceleration

[159] NEHRP started 1997, more information see *Section 5.2.2.2 "*US Seismic Design Codes*" on page 87*
[160] cf. *http://www.nehrp.gov/about/history.htm:* About us: Background&History. National Earthquake Hazards Reduction Program. [06.03.2009]
[161] *http://earthquake.usgs.gov/hazards/products/conterminous/* 17.02.2016

5.1.3 Risk in Chile[162]

Chile is one of the most seismically-endangered countries in the world. Some of the highest degrees of seismicity have been measured in Chile.

Nowadays, approximately 16 million people live in the narrow (4300 kilometers long and 175 kilometers wide) country bordered by Peru, Bolivia and Argentina.

Almost one third of the population lives in greater Santiago (capital city) situated in the middle of a central plain between coastal mountains on the west and the Andes Mountains to the east. The rest is, more or less, spread over urban areas within the country.

The entire length of the Chile lies along a major subduction zone – the Nazca subduction zone, which is like the San Andreas fault, part of the Pacific Ring of Fire. The Nazca Plate is being subducted beneath the South American plate resulting in the uplift and volcanism of the Andes Mountains and in turn in frequent, large-magnitude earthquakes. The two plates are converging, with a relatively high velocity, of approximately 7 meters per century.

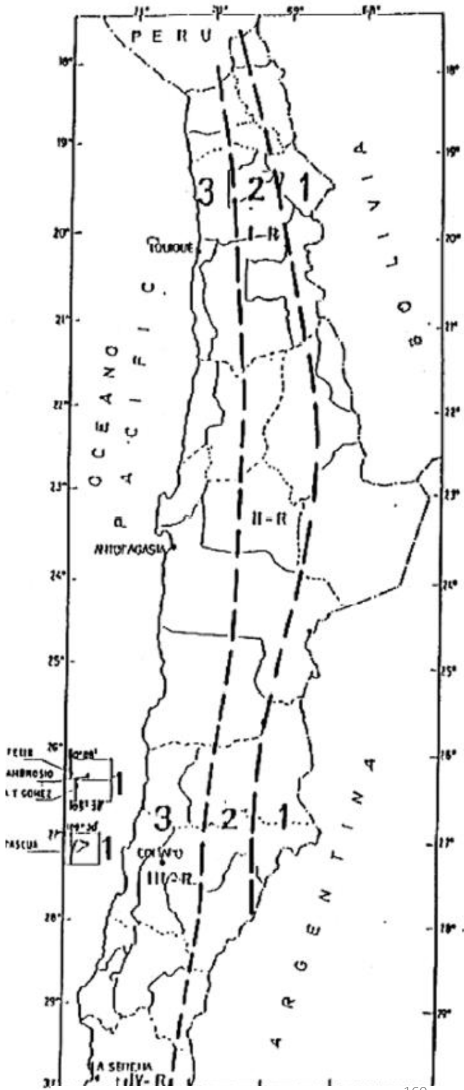

Figure 43 – seismic zones of Chile[160]

The United States Geological Survey (USGS) lists approximately 25 major earthquakes, with magnitude over 7.0, that have occurred within the country's borders since 1730.

On January 24th 1939, the earthquake with the greatest number of casualties happened in Chillán. It caused the loss of 30.000 human lives and 3500 building being collapsed at the initial shock. After the aftershocks, electrical power went down, the drinking water supply was damaged and 95% of the city destroyed.

The earthquake with the highest magnitude(Mw = 9.5) and the largest earthquake known in the 20th Century, occurred in Valdivia City on May 22, 1960.

The latest major seismic event (Maule Earthquake with Mw=8.8) occurred on February 27th 2010, 325 km away from Santiago de Chile and 125 km from Conceptión. The seismic motion of the initial shock lasted for approximately 3 minutes. During the following month, 257 aftershocks (until March 20th), 18 with magnitudes of 6.0 or greater occurred.

[162] cf. Ene, Diana/Craifaleanu Iolanda-Gabriela: Seismicity and Design Codes in Chile: Characteristic Features and a Comparison with some of the Provisions of the Romanina Seismic Code. In: Constructii – Nr. 2/2010, p. 69-78. AND cf. NEHRP Consultants Joint Venture (ed.): NIST GCR 12-917-18. Comparison of US and Chilean Building Code Requirements and Seismic Design Practice 1985-2010. Redwood City, CA: National Institute of Standards and Technology Oct. 2012

[163] Ene/Craifaleanu, p. 71

According to the Chilean code, the country is divided into three seismic zones *[see figure 43]*. The most vulnerable zone is at the pacific coast. It becomes lower as you move inland. The specific peak ground accelerations can be found in *Figure 44* below.

Seismic zone	A_0
1 (at the border with Argentina)	0.20 g
2	0.30 g
3 (shores)	0.40 g

Figure 44 – peak ground acceleration of Chilean's seismic zones[164]

5.1.4 Conclusion

See Table 1 for the comparison of the risk of the three countries. First thing compared is the population density. A look back to *equation 1 on page 12* →"risk is hazard times expected damage" shows that high density means high expected damages in a seismic event.

Due to the high population density of Austria, compared to the two others, the risk should not be underestimated in Austria, despite their relatively low hazard and low peak ground acceleration-values.

The population density of the USA is in the middle of Austria and Chile but the country experience more frequent moderate seismic events than the other two. Although there are areas in the US associated with the same maximum PGA as Chile, the experienced magnitudes cannot keep up with those record-breaking ones of Chile.

Chiles show the lowest population density, with huge sparsely populated areas. The problem hereby, which enhances the risk is the local concentration of the population. Almost one third of the whole population lives in the capital city, Santiago Metropolis. The country is only divided into three different seismic zones, all with relatively high values of peak ground accelerations. Note that the zone with the lowest PGA in Chile is equal to the second highest on in the US.

[164] Ibid., p. 72

Qualities	Austria	USA/California	Chile
Population Density[165]	101.4/km^2	95.0/km^2	24/km^2
Geological peculiarities	No tectonic boundaries *earthquakes may be triggered by mining activities*	San Andreas Fault Pacific and the North American Plate moving into opposite directionsfor about 33-37 millimeters a year	Nazca subduction zone The Nazca Plate is being subducted beneath the South American plate with a high velocity of approximately 800 millimeters a year
Seismic Hazard	magnitudes between 4.0 and 5.0, every 15-30 years one with moderate damages; every 75-100 years one with severe damages	Every year 10-20 earthquakes with magnitudes from 3.0 to 6.0 *The probability of a severe earthquake (magnitude of 6.7 or higher) occurring in the next 20 years is predicted with more than 90%.*	High magnitude-earthquakes - 25 major earthquakes one's, with magnitude over 7.0, since 1730 The latest event = Maule Earthquake with Mw=8.8 occurred on February 2010
PGA's	zone 0 → 0.00g - 0.03g (0.00 – 0.35 m/s²) zone 1 → 0.03g - 0.05g (0.35 – 0.50 m/s²) zone → 0.05g - 0.07g (0.50 – 0.75 m/s²) zone 3 → 0.07g - 0.1g (0.75 – 0.98 m/s²) zone 4 → > 0.10g (> 0.98 m/s²)	Zone 1 → 0.00g (0.00 m/s²) Zone 2 → 0.01g (0.09 m/s²) Zone 3→ 0.02g (0.19 m/s²) Zone 4 → 0.03g (0.29 m/s²) Zone 5 → 0.05g (0.49 m/s²) Zone 6 → 0.07g (0.68 m/s²) Zone 7 → 0.1g (0.98 m/s²) Zone 8 → 0.15g (1.47 m/s²) Zone 9 → 0.20g (1.96 m/s²) Zone 10 → 0.40g (3.92 m/s²)	Zone 1 → 0.20g (1.96 m/s²) Zone 2 → 0.30g (2.94m/s²) Zone 3→ 0.40g (3.92 m/s²)

Table 1 – comparison of risk

[165] Values acc. Wikipedia: https://en.wikipedia.org/wiki/California; https://en.wikipedia.org/wiki/Chile; https://en.wikipedia.org/wiki/Austria [23.02.2016]

5.2 Comparison of Relevant Standards and Regulations for Seismic Design

The relevant codes, standard and guidelines for seismic engineering are addressed in this chapter. Each country has its own prescriptive codes or recommendations for the design of structures. It is recommended to consider and follow those current design standards of countries with high earthquake hazards and advanced research of seismic impacts, like it is the case in China, Japan, India, Chile, Turkey or in the USA. Relevant Standards for the seismic design of industrial buildings of Austria and the USA are listed in the following.

5.2.1 Codes in Austria

The ÖNORM EN 1998 code in Austria contains the full text of the Eurocode 8 prescribed by CEN (the European Committee for Standardization) and additionally a national annex ÖNORM "B"1998 for special characteristics of Austria.

The Eurocode 8 – "Design provisions for earthquake resistance of structures" – is divided into 6 parts:

- ÖNORM EN 1998-1: Eurocode 8: Design provisions for earthquake resistance of structures. General rules. Seismic actions and general requirements for structures. 2012
- ÖNORM EN 1998-2: Eurocode 8: Design provisions for earthquake resistance of structures. Bridges.2012
- ÖNORM EN 1998-3: Eurocode 8: Design provisions for earthquake resistance of structures. General rules. Strengthening and repair of buildings 2013
- ÖNORM EN 1998-4: Eurocode 8: Design provisions for earthquake resistance of structures. Silos, tanks and pipelines. 2008
- ÖNORM EN 1998-5: Eurocode 8: Design provisions for earthquake resistance of structures. Foundations, retaining structures and geotechnical aspects. 2005
- ÖNORM EN 1998-6: Eurocode 8: Design provisions for earthquake resistance of structures. Towers, masts and chimneys 2005

5.2.1.1 Process: Prescriptive Design according Eurocode 8[166]

In the following text the design procedure of ÖNORM EN 1998-1 "Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings" is described.

Nuclear power plants, off-shore structures and large dams, are beyond the scope of EN 1998.

5.2.1.1.1 Performance Requirements and Compliance Criteria

The design objective this code pursued is, to:

- protect human lives
- limit damage
- provide operation of structures important for civil protection

[166] cf. Eurocode 8: ÖNORM EN 1998-1

There are two requirements buildings in seismic areas must meet:

- <u>no-collapse requirement *(ultimate limit states)*</u>
  withstand design seismic action *[defined acc. equation 1*4] without local or global collapse thus retaining its structural integrity and a residual load bearing capacity after the seismic events
- <u>damage limitation requirement *(damage limitation states)*</u>
  withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself

The design seismic action is expressed in terms of:

- the reference seismic action associated with a reference probability of exceedance, P_{NCR}, in 50 years or a reference return period, T_{NCR}, and
- the importance factor γ_I to take into account reliability differentiation. The recommended values are P_{NCR} =10% and T_{NCR} = 475 years.

The seismic action to be taken into account for the “damage limitation requirement” has a probability of exceedance, P_{DLR}, in 10 years and a return period, T_{DLR}. The recommended values are P_{DLR} =10% and T_{DLR} = 95 years.

In order to satisfy the fundamental requirements set forth the ultimate limit states and damage limitation states shall be checked.

ULS = the structural system shall be verified as having the resistance and energy dissipation capacity specified in the relevant parts of EN 1998. The balance between resistance and energy-dissipation capacity is characterised by the values of the behaviour factor q and the associated ductility classification.

Furthermore following points must be ensured:

- verification of overturning and sliding stability
- verification that both the foundation elements and the foundation-soil are able to resist the action effects resulting from the response of the superstructure without substantial permanent deformations.
- verification that under the design seismic action the behaviour of nonstructural elements does not present risks to persons and does not have a detrimental effect on the response of the structural elements

DLS = an adequate degree of reliability against unacceptable damage shall be ensured by satisfying the deformation limits

Furthermore following point must be ensured:

- verification to possess sufficient resistance and stiffness to maintain the function of the vital services in the facilities

5.2.1.1.2 General Design Considerations

The guiding principles governing the conceptual design against seismic hazard are:

- structural simplicity
- uniformity, symmetry and redundancy
- bi-directional resistance and stiffness
- continuous load path
- torsional resistance and stiffness
- diaphragmatic behaviour at storey level
- adequate foundation

The design principles of the preliminary design acc. *Section 2.2.1 "Preliminary Design" on page 24* must be observed. Furthermore, the application of the capacity design procedure, which is used to obtain the hierarchy of resistance of the various structural components and failure modes necessary for ensuring a suitable plastic mechanism and for avoiding brittle failure mode, is recommended.

Criteria describing regularity in plan and in elevation are given in the Eurocode 8 with following consequences on the seismic analysis and design process:

- the structural model can be either a simplified planar or a spatial one
- the method of analysis can be either a simplified response spectrum analysis (lateral force procedure) or a modal one
- the value of the behaviour factor q can be decreased depending on the type of non-regularity in elevation

5.2.1.1.3 Ground Conditions

According to Eurocode 8 a ground type and a seismic zone must be assigned for each specific construction site.

The ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters may be used to account for the influence of local ground conditions on the seismic action.
The seismic zones are needed to determine the local reference peak ground acceleration on type A ground, a_{gR}.

The reference peak ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to the reference return period TNCR of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years, P_{NCR})

For return periods other than the reference, the design ground acceleration on type A ground a_g is equal to a_{gR} times the importance factor γ_I ($a_g = \gamma_I * a_{gR}$).

5.2.1.1.4 Seismic Action

The earthquake motion at a given point of the surface is represented by an elastic ground acceleration response spectrum, called "elastic response spectrum".

The shape of the elastic response spectrum is taken the same for the two levels of seismic action for the no-collapse requirement and for the damage limitation requirement. The horizontal seismic action is described by two orthogonal components considered as independent and represented by the same response spectrum.

5.2.1.1.5 Method of Analysis

Acc. EC 8, depending on the structural characteristics of the building, one of the following two types of linear-elastic analysis may be used:

a) the lateral force method of analysis for buildings meeting regularity- criteria
b) the modal response spectrum analysis, which is applicable to all types of buildings

As alternative to a linear method, a non-linear method may also be used, such as:

c) non-linear static (pushover) analysis;
d) non-linear time history (dynamic) analysis

The lateral force method only considers the first fundamental vibration mode in contrast to the modal response spectrum analysis, where more vibrations modes are considered.

For a non-linear method the mathematical model used for elastic analysis shall be extended to include the strength of structural elements and their post-elastic behaviour.

The linear-elastic analysis may be performed using two planar models, one for each main horizontal direction, as mathematical model. This method is solely appropriate if buildings satisfying the regularity- criteria, otherwise they should be analysed using a spatial model.

Whenever a spatial model is used, the design seismic action shall be applied along all relevant horizontal directions (with regard to the structural layout of the building) and their orthogonal horizontal directions.

5.2.1.1.5.1 Design Spectrum for Elastic Analysis

The design spectrum, $S_d(T)$, is defined by the following expressions: *[see Figure 45]*

$$0 \le T \le T_B : \ S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3}\right)\right]$$

$$T_B \le T \le T_C : \ S_d(T) = a_g \cdot S \cdot \frac{2,5}{q}$$

$$T_C \le T \le T_D : \ S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C}{T}\right] \\ \ge \beta \cdot a_g \end{cases}$$

$$T_D \le T : \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C T_D}{T^2}\right] \\ \ge \beta \cdot a_g \end{cases}$$

Figure 45 –EC 8 design spectrum for elastic analysis

$S_d(T)$ ....design spectrum
T ....vibration period of a linear single-degree-of-freedom system
a_g ....design ground acceleration on type A ground
T_B, T_C ......limits of the constant spectral acceleration branch
T_D ....value defining the beginning of the constant displacement response range of the spectrum
S ....soil factor
q ....behaviour factor
β ....lower bound factor for the horizontal design spectrum

5.2.1.1.5.2 *Base Shear Force*

The seismic base shear force F_b for each horizontal direction in which the building is analysed, is determined as follows:

$$F_b = S_d(T_1) * m * \lambda$$

Equation 13 – EC 8 base shear force

$S_d(T)$ ....ordinate of the design spectrum at period T_1
T_1............fundamental period of vibration of the building for lateral motion in the direction considered
m...........total mass of the building, above the foundation or above the top of a rigid basement
λ............correction factor, the value of which is equal to: $\lambda = 0{,}85$ if $T1 < 2\ TC$ and the building has more than two storeys, or $\lambda = 1{,}0$ otherwise

For buildings with heights up to 40 m the value of T1 (in seconds) may be approximated by the following expression:

$$T_1 = C_t * H^{3/4}$$

Equation 14 – EC 8 approximation of fundamental (first) period of the building

C_t.............0,085 for moment resistant space steel frames
0,075 for moment resistant space concrete frames and for eccentrically braced steel frames
0,050 for all other structures
H ....height of the building, in m, from the foundation or from the top of a rigid basement

5.2.1.1.5.3 *Distribution of the Horizontal Seismic Forces*

The seismic action effects shall be determined by applying, to the two planar models, horizontal forces F_i to the lateral load resisting system (assuming rigid floors) to all storeys.

$$F_i = F_b * \frac{z_i * m_i}{\sum z_j * m_j}$$

Equation 15 – EC 8 distribution of the horizontal seismic force

z_i,z_j..........heights of the masses m_i,m_j above the level of application of the seismic action (foundation or top of a rigid basement)
m_i,m_j ......storey masses

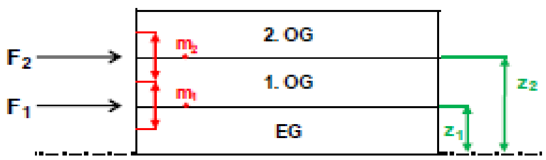

Figure 46 – EC 8 distribution of the horizontal seismic force

5.2.1.1.5.4 *Combination of the Effects of the Components of the Seismic Action*

If a_{vg} is greater than 0.25 g (2.5 m/s^2) the vertical component of the seismic action, as defined in 3.2.2.3, should be taken into account in the cases below:

- for horizontal or nearly horizontal structural members spanning 20 m or more
- for horizontal or nearly horizontal cantilever components longer than 5 m
- for horizontal or nearly horizontal prestressed components
- for beams supporting columns
- in base-isolated structures

All three of the following combinations may be used for the computation of the action effects. *[see Figure…]*

a) E_{Edx} "+" 0,30 E_{Edy} "+" 0,30 E_{Edz}

b) 0,30 E_{Edx} "+" E_{Edy} "+" 0,30 E_{Edz}

c) 0,30 E_{Edx} "+" 0,30 E_{Edy} "+" E_{Edz}

Figure 47 – EC 8 computation of seismic action effects

With this calculated force of the seismic action, the stress resultants are determined and the designed structural system can be verified. Special provisions for the most common structural systems are given in respective chapters of the Eurocode 8.

5.2.1.1.6 Displacement Analysis

If linear analysis is performed the displacements induced by the design seismic action shall be calculated on the basis of the elastic deformations of the structural system by means of the following simplified expression:

$$d_s = q_d * d_e$$

Equation 16 – EC 8 deformation of the structural system

d_s...........displacement of a point of the structural system induced by the design seismic action
q_d ....displacement behaviour factor, assumed equal to *q* unless otherwise specified
d_e...........displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum

When determining the displacements d_e, the torsional effects of the seismic action shall be taken into account. For non-linear analysis, static or dynamic, the displacements are those obtained from the analysis.

5.2.1.1.7 Provisions for Nonstructural Elements

The Eurocode 8 – part 1 contains a particular chapter regarding the design of nonstructural components. Together with the EC8 part 4 “Design provisions for earthquake resistance of structures. Silos, tanks and pipelines” provisions are given verify the nonstructural elements together with their supports, resisting the design seismic action.

For nonstructural elements of great importance or of a particularly dangerous nature, the seismic analysis shall be based on a realistic model of the relevant structures and on the use of appropriate response spectra derived from the response of the supporting structural elements of the main seismic resisting system. In all other cases properly justified simplifications of this procedure, like describe in *Section 4.2.3.2.1 “Analysis of Simple Acceleration-Sensitive Components” on page 64*, are appropriate.

5.2.2 Codes in the USA

5.2.2.1 General Development and Overview

This section provides an overview of the relevant seismic design guidelines in the US. The American building code organization is more comprehensible than it is in Austria. Many federal agencies or associations work hand in hand to develop structural codes and standards in order to constantly advance the State-of-Art and State-of-Practice[167]. The figure below names the most relevant institutions and shows their relationships.

In the early part of the last century three non-profit organizations developed three separate sets of model codes each used in different parts of the country. The Building Officials and Code Administrators International (BOCA) developed the National Building Code (NBC) used in the northeastern part, the International Conference of Building Officials (ICBO) developed the Uniform Building Code (UBC) applied in the West and the Southern Building Code Congress International (SBCCI) the Standard Building Code (SBC) which was, as the name implies, used in the southeastern part[168]. That inconsitency continued until the International Code Council (ICC) was established in 1994 with the intension to combine these three sets and develop a single set, named the International Building Codes (IBC) valid for the whole country. This look back is important because the IBC and the UBC are often mixed up in California and it should be noticed that a lot of current papers or guidelines still refer to the UBC only. *[see figure 21]*

In addition to the governmental IBC, three more institutions need to be mentioned because of their contribution to the code development: the Structural Engineers Association of California (SEAOC), the American Society of Civil Engineers (ASCE) and the Applied Technology Council (ATC).

SEAOC provided a seismic design manual to guide through the UBC application and established 1973 the non-profit Applied Technology Council (ATC). ATC is guided by SEAOC and ASCE.

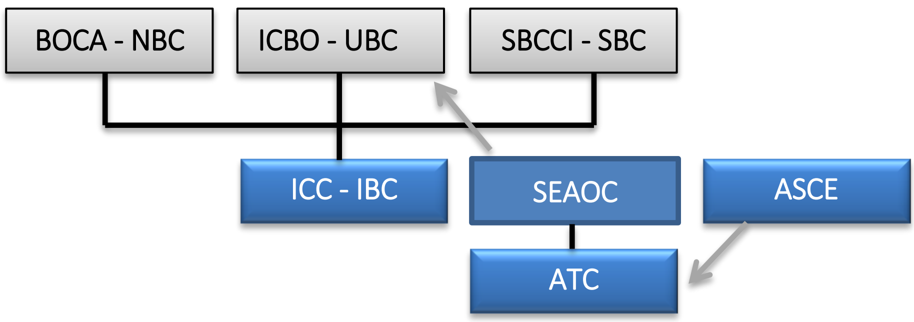

figure 48 – relevant seismic design guidelines in the US

[167] http://www.seaoc.org/mission
[168] http://skghoshassociates.com/sk_publication/PCI_Jan02_Seis_design_provi_in_US.pdf

5.2.2.2 US Seismic Design Codes

In order to reduce the seismic risk and limit the financial losses following a very probable future earthquake in the USA, the US Congress passed the Earthquake Hazard Reduction Act in 1977 and established the National Earthquake Hazards Reduction Program (NEHRP). The program was periodically reviewed and reauthorized[169] and involves four federal agencies[170]. *[see figure 44]*

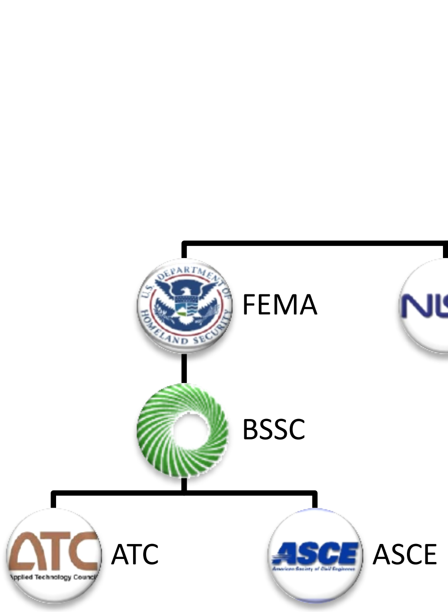

figure 49 – organisation of NEHRP

The important standards for seismic engineering are prepared by ATC and ASCE and funded by FEMA: ATC and ASCE are subcontracted by the Building Seismic Safety Council (BSSC) which is, in turn, under contract with FEMA.

[169] *<u>Reauthorized in years:</u> 1980, 1981, 1983, 1984, 1985, 1988, 1990, 1994, 1997, 2000, and 2004*

[170] *<u>The four federal agencies:</u> Federal Emergency Management Agency (FEMA) of the Department of Homeland Security; the National Institute of Standards and Technology (NIST) of the Department of Commerce (NIST is the lead NEHRP agency); the National Science Foundation (NSF) and the United States Geological Survey (USGS) of the Department of the Interior*

5.2.2.3 Relevant US Codes and Papers

Relevant US-codes and papers for the seismic design of industrial constructions are listed below:

- IBC International Building Code. International Code Council (ICC) 2015
- IBC Structural/Seismic Design Manuals. Volume 1-5. Structural Engineers Association of California (SEAOC) 2012

- SEAOC Blue Book. Recommended Lateral Force Requirements and Commentary. Structural Engineers Association of California (SEAOC) 2009
- SEAOC Vision 2000: Performance-Based Seismic Engineering of Buildings. Structural Engineers Association of California (SEAOC) 1995

- ASCE/SEI 31-03: Seismic Evaluation of Existing Buildings. American Society of Civil Engineers ASCE 2003
- ASCE/SEI 41-06: Seismic Rehabilitation of Existing Buildings. American Society of Civil Engineers ASCE 2006
- ASCE/SEI 43-05: Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities. American Society of Civil Engineers ASCE 2005
- ASCE/SEI 7-05: Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers ASCE 2006
- ASCE/SEI 7-10: Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers ASCE 2010

- FEMA P-749: Earthquake-Resistant Design Concepts: An Introduction to the NEHRP Recommended Seismic Provisions for New Buildings and Other Structures. Federal Emergency Management Agency 2009
- FEMA P-1050: NEHRP Recommended Seismic Provisions for New Buildings and Other Structures. Volume I: Part 1 Provisions, Part 2 Commentary. Federal Emergency Management Agency 2015
- FEMA P-1050-2: NEHRP Recommended Seismic Provisions for New Buildings and Other Structures. Volume II: Part 3 Resource Papers. Federal Emergency Management Agency 2015
- FEMA 450: NEHRP Recommended Provisions and Commentary for Seismic Regulations for New Buildings and Other Structures. Federal Emergency Management Agency 2003
- FEMA 451, 451B: NEHRP Recommended Provisions: Design Examples; Training and Instructional Materials. Federal Emergency Management Agency 2006

- FEMA 349: Action Plan for Performance-Based Seismic Design. Federal Emergency Management Agency 2000
- FEMA 445: Next-Generation Performance-Based Seismic Design Guidelines: Program Plan for New and Existing Buildings. Federal Emergency Management Agency 2006
- FEMA P-58-1: Seismic Performance Assessment of Buildings Volume 1—Methodology. Federal Emergency Management Agency 2012
- FEMA P-58-2: Seismic Performance Assessment of Buildings Volume 2—Implementation Guide. Federal Emergency Management Agency 2012

- FEMA 412: Installing Seismic Restraints for Mechanical Equipment. Federal Emergency Management Agency 2004
- FEMA 413: Installing Seismic Restraints for Electrical Equipment. Federal Emergency Management Agency 2004
- FEMA 414: Installing Seismic Restraints for Duct and Pipe. Federal Emergency Management Agency 2004
- FEMA E-74: Reducing the Risks of Nonstructural Earthquake Damage—A Practical Guide, Fourth Edition. Federal Emergency Management Agency 2012

- ATC-69: Reducing the Risks of Nonstructural Earthquake Damage, State-of-the-Art and Practice Report. Applied Technology Council 2008
- ATC-29-2: Proceedings of Seminar on Seismic Design, Performance, and Retrofit of Nonstructural Components in Critical Facilities. Applied Technology Council 2003

Due to the high number of relevant codes, the process of the prescriptive design in the US is not explained in detail, as it is the case for Austria in *Section 5.2.1.1 "Process: Prescriptive Design according Eurocode 8" on page 80* and for Chile in *Section 5.2.3.1 "Process: Prescriptive design acc. NCh433.Of96 and NCh2369.Of2003*below. The most important steps of the ASCE/SEI 7-10 are illustrated in *Table 2 "Comparison of design procedures" on page 99*.

5.2.3 Codes, Standards and Guidelines in Chile

After the Valdivia earthquake in 1960, the Chilean government financed the research work for a seismic design code (NCh433.Of96), which was implemented in 1996.

Due to this precautions, the number of casualties were relatively low in the Maule Earthquake (with Mw=8.8) of February 27th 2010, although it was near to Santiago de Chile (325 km) and Conceptión (125 km). Nowadays research is being carried out in order to update the NCh433.Of96 code.

In 2003 the NCh2369.Of2003 code was published that deals with the earthquake-resistant design of industrial structures and facilities.

In the following text the Chilean design procedure is described, based on the NCh433 and the NCh2369.

5.2.3.1 Process: Prescriptive design acc. NCh433.Of96 and NCh2369.Of2003[171]

5.2.3.1.1 Performance Requirements and Compliance Criteria

In the NCh433 code the performance objective is described as follows (in a very general way):

- resist moderate intensity seismic actions without damages
- limit damage to nonstructural elements during earthquakes of regular intensity
- prevent collapse during earthquakes of exceptionally severe intensity, even though they show some damage

In comparison the NCh2369.Of2003 states following two design objectives:

- Protection of life in industry
  prevent collapse in event of severe over-design-earthquakes, prevent fire, explosion or emission of toxic gases and liquids and assure operability of seismic emergency exits
- Continuity of operation in industry
  non-interruption of essential processes and services, minimize the standstill of operations and enable inspection and repair of damaged elements

These objectives are met by providing reserve of strength or capability of absorbing large quantities of energy beyond the elastic range, prior to failure.

The NCh2369.Of2003 is applicable to heavy and light industrial facilities (duct, pipe systems, mechanical and electrical equipment), but not to nuclear stations, electric power generation plants, dams, bridges and tunnels etc.

[171] cf. Instituto Nacional de Normalizacion (ed.): Official Chilean Standard. NCh 433.Of96. Earthquake resistant design of buildings. Santiago: INN 1996 AND NCh2369.Of2003

5.2.3.1.2 General Design Considerations

The principal design requirements, stated in both NCh433 and NCh2369, are:

- the systems shall be redundant and hyperstatic
- use of simple and clearly identifiable systems for transmission of earthquake forces to the foundation
- avoiding structures of high asymmetry and complexity
- obey requirements for diaphragms, building separations and deformation limits

NCh433 provide provisions of three general types of seismic force-resisting systems:

- shear wall and other braced systems
- moment-resisting space frame systems
- dual systems containing a combination of the above two systems

These systems are further classified according to their material of construction.

5.2.3.1.3 Ground Conditions and Seismic Action

Similar to the UBC in the United States and the EC8 in Austria, NCh433 uses seismic zonation to establish design shaking intensities. Each region and city is classified to one of the three Chilean seismic zones and its associated peak ground acceleration, A_0 (, in % of g).

The site effects on ground shaking intensity are accounted through assignment of spectral modification coefficients (S, T') based on soil type (I,II,III,IV).

For considering occupancy, importance and failure risk in the determination of seismic design forces, the Chilean Codes provide classifications for buildings. NCh433 defines four importance categories - A, B, C, and D.

NCh2369 defines three for structures and equipment (C1, C2 and C3), described in the following:

- Category C1. Critical structures and equipment based on any one of the following reasons: *(importance coefficient I = 1.20)*
  - Vital, must be kept in operation so to control fire, explosion and ecological damage, render health and first help services.
  - Dangerous, if their failure implies hazard of fire, explosion or air and water poisoning.
  - Essential, if their failure generates protracted standstills and serious production losses.
- Category C2. Normal structures and equipment, which may be affected by normal easily repairable failures, which do not cause protracted standstills or important production losses or hazard to other category C1 structures. *(importance coefficient I = 1.00)*
- Category C3. Minor or provisional structures and equipment, whose seismic failure does not cause protracted standstills nor exposes to hazard other category C1 and C2 structures. *(importance coefficient I = 0.80)*

5.2.3.1.3.1 *Methods of Analysis*

NCh433 recognizes two analytical procedures for determining seismic design forces: a static procedure and a modal response spectrum procedure. The modal response spectrum procedure can be used in the design of any building. The static analysis procedure is limited to specific applications, defined in the code.

According NCh2369 three procedures may be used:

- static analyses or analysis of equivalent static forces, which can only be applied to structures of up to 20 m height, provided their seismic response might be assimilated to a single-degree-of-freedom system.
- modal spectral analysis, which is applicable to any type of structure.
- special methods for structures featuring elastic behaviour

5.2.3.1.3.2 *Design Spectrum*

The design spectrum, $S_a(T)$, is defined by NCh433[172] the following equation:

$$S_a(T) = \frac{I * A_0 * \alpha}{R}$$

Equation 17 – NCh433 design spectrum

$S_a(T)$ ...... design spectrum
I............ importance factor
A_0 .... maximum effective acceleration defined according to the seismic zonification
α........... amplification factor *see Equation 19*
R........... response modification factor, based on system type reflects energy absorption and dissipation characteristics as well as seismic behaviour of different types of structures and materials used

$$\alpha = \frac{1 + 4.5 * (\frac{T_n}{T_o})^P}{1 + (\frac{T_n}{T_o})^3}$$

Equation 18 – NCh433 amplification factor for the design spectrum

T_n........... vibration period of mode *n*
T_o, P ...... parameters relative to the foundation soil type

5.2.3.1.3.3 *Base Shear Force*

The seismic base shear force Q_o for each horizontal direction in which the building is analysed, is determined (in NCh433 as well as NCh2369) as follows:

$$Q_o = C * I * P$$

Equation 19 – NCh433 base shear force [173]

C........... seismic coefficient, defined in *Equation 21 or 22*
I............ importance factor
P........... total weight of the building above the base level

[172] NCh 433.Of96, p. 26, 6.3.5
[173] Ibid., p. 23, 6.2.3

NCh433 provides following equation to determine the seismic coefficient:

$$C = \frac{2.75 * A_o}{gR} * (\frac{T'}{T^o})^n$$

Equation 20 – NCh433 seismic coefficient

T', n....... parameters relative to the foundation soil type
T^o .... period of mode with the highest translational equivalent mass in the direction of analysis,
calculated by a well-founded theoretic or empiric procedure

NCh2369 provides following equation to determine the seismic coefficient:

$$C = \frac{2.75 * A_o}{gR} * (\frac{T'}{T^*})^n * (\frac{0.05}{\xi})^{0.4}$$

Equation 21 – NCh2369 seismic coefficient[174]

T^*........... fundamental period of vibration in the direction of the analysis, calculated by a well-founded theoretic or empiric procedure
ξ .... damping ratio

5.2.3.1.3.4 Distribution of the Horizontal Seismic Forces

The seismic force shall be distributed along height according NCh2369 to the following expression:

$$F_k = \frac{A_k * P_k}{\sum_1^n A_j * P_j} * Q_o$$

Equation 22 – NCh2369 distribution of the horizontal seismic force[175]

$$A_k = \sqrt{1 - \frac{Z_{k-1}}{H}} - \sqrt{1 - \frac{Z_k}{H}}$$

Equation 23 – NCh2369 parameter a level *k*

F_k........... horizontal seismic force at level k
P_k, P_j...... seismic weight at levels k and j
A_k .... parameter at level k (k=1 is the lower level)
n............ number of levels
Qo .... base shear
Z_k, Z_{k-1} ... height above the base of k and k-1 levels
H .... highest height levels above the base level

[174] NCh2369.Of2003, p. 27, 5.3.3.
[175] Ibid., p. 28, 5.3.5

5.2.3.1.3.5 Combination of the Effects of the Components of the Seismic Action

According NCh2369 the structure shall be analysed considering the earthquake loads at least in two horizontal, approximately perpendicular directions.

The effect of vertical earthquake accelerations shall be considered in the following cases:

- hanging bars of suspended equipment and their supporting elements and beams of rolled, welded or bent plate steel, with or without concrete slab as composite beam, located within the seismic zone 3, where permanent loads represent over 75% of the total load.
- Structures and elements of prestressed concrete (pretension and post tension cable).
- Foundations and elements for anchorage and support of structures and equipment.
- Any other structure or element in which the variation of the vertical earthquake action significantly affects its detailing, as for instance, cantilever structures and elements.
- Structures with seismic isolation sensitive to the vertical effects.

5.2.3.1.4 Displacement Analysis

The deformations shall be determinded (in case the analysis considers R-factor reduced earthquake loads) acc. NCh2369 as follows:

$$d = d_0 + R_1 * d_d$$

Equation 24 – deformation of the structural system[176]

d............seismic deformation
d_0 ....deformation due to non-seismic service loads
d_d ....deformation calculated with R-factor reduced earthquake loads
R_1 ....factor resulting from multiplying the R factor by the quotient Q_0/Q_{min}

The separation between structures shall be bigger than the highest of the following values *[Equation 25 to 27]*, with the purpose of preventing impacts between adjoining structures:

$$S = \sqrt{(R_{li} * d_{di})^2 + \left(R_{lj} * d_{dj}\right)^2} + d_{0i} + d_{0j}$$

Equation 25 – NCh2369 minimum separation value 1

$$S = 0.002\ (h_i + h_j)$$

Equation 26 – NCh2369 minimum separation value 2

$$S = 30\ mm$$

Equation 27 – NCh2369 minimum separation value 3

d_{di}, d_{dj}....deformation of the structures i and j calculated as per *Equation 24*
R_{li}, R_{lj}.....response modification factor R_1 used for the design of the structures i and j
h_i, h_j......height at the considered level of the structure i and j measured from their respective base levels

The P-delta effect must be considered by using second order analysis in case the seismic deformation exceed 15 % of the building's height.

[176] NCh2369.Of2003, p. 47, 6.1

5.2.3.1.5 Provisions for Nonstructural Elements

NCh2369 provides following provisions for nonstructural elements:

5.2.3.1.5.1 Multistory Equipment

Rigid ducts or equipment vertically extended over more than one story shall be outfitted with bearing and connecting systems that prevent their participation in the strength or stiffness of the building. If this is not possible, the equipment shall be included in the model of the earthquake-resistant system.

5.2.3.1.5.2 Large Suspended Equipment

Hereby the most important point is to attach them by connectors that transmit the seismic force without restraining the horizontal and vertical thermal expansion.

5.2.3.1.5.3 Piping and Ducts

Expansion joints and supports that warrant seismic stability and simultaneously allow thermal expansion must be provided. If piping and ducts are light in relation to the buildings or structures they connect, the seismic analysis can be carried out introducing the deformations for the buildings or structures, at the points of connection.

The weight of tubes is mostly insubstantial as compared to the weight of buildings and structures; therefore it is enough that the seismic deformations are considered in the analysis of the piping system and in the design of the connections.

5.2.3.1.5.4 Elevated Tanks and Process Vessels

Elevated tanks shall be designed considering the mobility of water/sloshing effect in order to prevent secondary damages caused by the movement of the liquid.

Attention must be paid to the joint of the supports to the shell of the process vessels when it is not extended in the foundation.

For elevated stacks, the interaction between the duct and the external steel or concrete structure could be critical.

The shell of tanks and vessels shall be designed to prevent local buckling by ensuring that the shell compression stress does not exceed the lowest of the following values:

$$F_a = 135 * F_y * \frac{e}{\mathrm{D}} \qquad F_a \leq 0.8 * F_y$$

Equation 28 – NCh433 shell compression stress limitations[177]

F_a .... allowable tension in seismic condition
F_y .... yield stress
e .... thickness
D .... shell diameter

Furthermore it is prescribed to consider the interaction effects of connected or adjacent nonstructural components., The piping systems and their connection points to the tanks shall be designed with ample deformation capability in order to prevent the possible damages caused by eventual uplifts of the tank bottom or tank displacements.

[177] NCh2369.Of2003, p. 74, 11.7.4

5.2.3.1.5.5 Other International Standards

The general stated requirements are specified in different international codes, which the Chilean Codes refer to. For example:

- American Society for Mechanical Engineers – ASME for boilers and pressure vessels
- American National Standards Institute – ANSI/ASME for piping
- American Petroleum Institute – API for tanks for oil storage
- American Society for Testing Materials – ASTM for materials
- American Welding Society – AWS for welding
- American Waterworks Association – AWWA for water tanks
- Empresa Nacional de Electricidad – ENDESA General Technical Specifications – for electric equipment
- New Zealand National Society for Earthquake Engineers – NZ – tanks and vessels
- German DIN, British BS, French NF, Japanese JIS and Euro standards

5.2.4 Conclusion

This section provides a tabular comparison of three different international design procedures according the NCh433.Of96 of Chile, the ASCE/SEI 7-10 of the USA and the EC 8 (ÖNORM EN 1998) of Austria. *[see Table 2]*

The design procedures of Austria and Chile have been described more detailed in *Sections 5.2.1.1 "Process: Prescriptive Design according Eurocode 8" on page 80 and 5.2.3.1 "Process: Prescriptive design acc. NCh433.Of96 and NCh2369.Of200390* above. Additionally this section contains the conclusion of a paper[178] which compared US and Chilean Building Code Requirements and Seismic Design Practice from 1985–2010.

This paper contains a table, that compares the NCh433.Of96 and ASCE/SEI 7-10 seismic design requirements, which now has been adopted and extended in Table 2 to the Austrian seismic design requirements.

<u>General Differences in the Design Practice</u>

Differences in design practice are the result of the occurrence of major seismic events in the last century, the evolution in construction techniques, differences in labour costs as a portion of total construction costs, and differences in the roles that structural engineers play in the building design process.

Earthquake forces in NCh433.Of96 are allowable stress level forces (ASD[179]), and earthquake forces in ASCE/SEI 7-05 and EC8 are strength level forces (LFRD[180]).

Another peculiarity of the Chilean code is that the structural models typically include all structural elements, rather than just those comprising the seismic force-resisting system as it is the case in the Austrian and US design practice.

[178] NIST GCR 12-917-18. Comparison of US and Chilean Building Code Requirements and Seismic Design Practice 1985-2010.
[179] ASD = allowable stress design
[180] LFRD = load and resistance factor design

General Differences in the Construction Practice

In Chile, dual buildings with braced shear or concrete walls in combination with rigid ductile frames as second resistance line are recommended, because they performed best in practice, with acceptable deformations. In North America as well as in Austria the moment resisting frames, based on capacity design are recommended.

The Chilean tendency is towards the use of distributed structural systems in which many elements provide lateral resistance, in contrast to the United States, where engineers try to minimize the number of elements and reduce the amount of redundancy provided in structural systems.

Reasons for that may be the differences in cost of construction labour relative to the materials. The high labour cost in the US result in minimizing the working hours – hence not so many elements to connect or attach.

Traditional Chilean practice is to configure buildings with relatively short floor spans and many load-bearing walls providing gravity and seismic force resistance. As a result, typical Chilean buildings have highly redundant configurations. This practice likely contributed to the ability of many buildings to withstand severe damage without collapse. As a consequence of this redundancy, and past experience with typical building configurations, requirements for ductile detailing in Chile are relaxed relative to US requirements.

In contrast, US practice is to configure buildings with longer spans and fewer structural walls. As a result, walls are thicker, allowing for easier placement of confinement reinforcing, and increased ductility capacity. As a consequence, US designs have comparatively less redundancy than Chilean designs.

Requirement	NCh 433.Of96	ASCE/SEI 7-10	ÖNORM EN 1998-1	Comment
Scope	Minimum design requirements for buildings and components Procedures for repair of damaged structures	Minimum design loads for buildings and other structures including nonstructural components	Design and Construction Requirements and specific rules for various structural material and elements	
Base Shear Equations	$Q_0 = CIP$ $C = \frac{2.75 A_0}{gR}\left(\frac{T'}{T^*}\right)^n$ $C_{min} = A_0 / 6g$ C_{max} per NCh433 Table 6.4 (based on R)	$V = C_S W$ $C_S = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \leq \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$ $C_{Smin} = 0.044 S_{DS} I_e \geq 0.01$ $C_{Smin} = 0.5 S_1 / (R/I_e)$ near fault	$F_b = S_d(T_1) \cdot m \cdot \lambda$ *Factor λ accounts for the fact that in buildings with at least three storeys and translational degrees of freedom in each horizontal direction, the effective modal mass of the 1st (fundamental) mode is smaller – on average by 15% - than the total building mass*	*Although attributed to somewhat different sources, the estimated seismic weight is approximately the same in each code.*
General Design Sepctrum	$S_a(T) = \frac{I A_0 \alpha}{R^*}$ $\alpha = \frac{1+4.5\left(\frac{T_n}{T_o}\right)^p}{1+\left(\frac{T_n}{T_0}\right)^3}$	0 < T = 0.2 sec: $S_a(T) = \frac{S_{DS}}{2.5} + \frac{T}{0.2}(0.6 S_{DS})$ 0.2 sec < T < T_S: $S_a(T) = S_{DS}$ T_S < T < T_L: $S_a(T) = \frac{S_{D1}}{T}$ T_L < T: $S_a(T) = \frac{S_{D1} T_L}{T^2}$	$0 \leq T \leq T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3}\right)\right]$ $T_B \leq T \leq T_C : S_d(T) = a_g \cdot S \cdot \frac{2,5}{q}$ $T_C \leq T \leq T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C}{T}\right] \\ \geq \beta \cdot a_g \end{cases}$ $T_D \leq T : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C T_D}{T^2}\right] \\ \geq \beta \cdot a_g \end{cases}$	*Unreduced response spectra for NCh433.Of96 Soil Type III and ASCE/SEI 7-05 Site Class D, in regions of high seismicity, are similar in shape and magnitude, although the Chilean spectrum does not include a short period plateau.*
Performance Categories and Occupancy Importance Factors	A - Government, municipal, public service, police stations, power plants, /= 1.2 B - High and special occupancy, /= 1.2 C - Ordinary buildings, /= 1.0 D – Uninhabited buildings, /=0.6	IV – Essential structures, hospital, police and fire stations, /= 1.50 III – Important and high occupancy structures, /= 1.25 II – Ordinary structures, /= 1.0 I – Uninhabited structrues, /= 1.0	γ_1 ($a_g = \gamma_1 \cdot a_{gR}$) IV – Power stations, hospital, police and fire stations, /= 1.4 III – Important and high occupancy structures, /= 1.2 II – Ordinary structures, /= 1.0 I – minor importance, argricultural buildings, /= 0,8	*NCh 433.Of96 and ASCE/SEI 7-10 importance factors direct in base shear equation;* *EN 1998-1 - in case of other return reference period than 50 years – multiply ground acceleration and importance factor*
Seismic Zonation	3 geographic seismic zones – with associated ground accelerations	None	4 geographic seismic zones in national annex with associated a_{gR} for site class A	ASCE 7 used Seismic Design Category for some cirteria defined by seismic zone in NCh433

Design Ground Motion	Defined by zero period acceleration, A_0, for each zone: zone 1 – 0.2 g zone 2 – 0.3 g zone 3 – 0.4 g	Defined by MCER accerleration contour maps that include: S_S – short period spectral response accerleration parameter ranging to 2.0g S_1 – 1 secound spectral response acceleration parameter ranging to 0.8g and peak ground acceleration ranging to 1.0g	Reference ground acceleration associated for each city ranging from 0.28 -1.11 m/s^2	*The criterion of the 10% excedence in the course of a minimum 50-year return period has been adopted by the North American UBC and the SEOAC standards as well as by the Chilean NCh433 and the EC 8*
Soil Type and Site Class	*T′ and n – soil parameters in base shear equation* I – Rock with V_s > 900 m/s (3000 ft/s); uniaxial compressive strength > 10 MPa II – Firm soil with (a) V_s > 400 m/s; (b) dense gravel with unit weight > 20kN/m³; (c) dense sand with relative density > 75% or Modified Proctor Compaction > 95%; (d) stiff cohesive soil with S_u > 0.1MPa III – (a) unsaturated sand with relative density between 50% and 75%; (b) unsaturated gravel or sand with Modified Proctor Compaction < 95%; (d) saturated sand with 20 < N < 40 IV – saturated cohesive soil with S_u < 0.025 MPa Liquefiable soils require special study	included in SDS in base shear equation A – Hard rock with V_s > 5000 ft/s B – Rock with 2500 ft/s < V_s < 5000 ft/s C – Dense soil with 1200 ft/s < V_s < 2500 ft/s; N > 40 blows/ft, S_u > 2,000 psf D – Stiff soil with 600 ft/s < V_s < 1200 ft/s; 15 < N < 50; 1000 psf < S_u < 2000 psf E – Soft clay with V_s < 600 ft/s; N < 15; S_u < 1000 psf F – unstable, collapsible, liquefiable soils requiring site-specific study	*S* – soil parameter in base shear equation A – rock or rock-like geological formation > 800 m/s B – deposits of very dense sand, gravel or very stiff clay 360-800 m/s C – deep depsits of dense or medium dense sand, gravel or stiff clay 180-360 m/s D – deposits of loose-to-medium cohesionless soil <180 E – surface type C or D, underlain by stiff material > 800 m/s S_1 – soft clays/silts with high plasticity index and high water content < 100 m/s S_2 – desposits of liquefiable soils of sensitve clays, or any other soil profile not included so far	
Design Parametners	Response modification coefficient R or R′, for static and dynamic force analysis procedures - reflects energy absorption and dissipation characteristics as well as seismic behaviour of different types of structures and materials used Period-dependent coefficients for each mode: $R^* = 1 + \frac{T^*}{0.1T_0 + \frac{T^*}{R_0}}$ For shear wall buildings: $R^* = 1 + \frac{NR_0}{4T_0R_0 + N}$	*R* – response modifaction coefficient (mode and period independent) C_d – defleciton amplification factor Ω_0 – overstrength factor	q = The value of the behaviour factor *q*, which also accounts for the influence of the viscous damping being different from 5%, are given for the various materials and structural systems and according to the relevant ductility classes η = damping correction factor with reference value 1 for 5% viscous damping	

Drift Limits	0.002*h* at diaphram center of mass Not more than 0.001 *h* greater at any other point on the diaphragm	Varies from 0.01*h* to 0.25 *h* depending on structural system type and Risk Category	Nonstructural connected to structural system: brittle material → $d_r\,n < 0.005h$ ductile material → $d_r\,n < 0.0075h$ connection which allow same deformation → $d_r\,n < 0.01h$	
Vertical Distribution of Forces	$F_k = \frac{A_k P_k}{\sum A_j P_j} Q_0$ $A_k = \sqrt{1-\frac{Z_{k-1}}{H}} - \sqrt{1-\frac{Z_k}{H}}$	$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$ k = 1.0 for $T < 0.5$ seconds k = 2.0 for $T > 2.5$ seconds	$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j}$	NCh433 story forces are higher than ASCE 7 story forces in the upper stories
Accidential Torsion	$\pm 0.1 b_k \frac{Z_k}{H}$	Eccentricity taken as 5% of diaphragm dimension perpendicular in the direction of the seismic action	$e_{ai} = \pm 0.05 \cdot L_i$	Same for ASCE/SEI 7-10 and EC 8 *to cover uncertainties in the location of masses and in the spatial variation of the seismic motion, in each direction*
Orthogonal Effects	Each direction considered separately	100% X + 30% Y 30% X + 100% Y	a) E_{Edx} "+" $0{,}30E_{Edy}$ b) $0{,}30E_{Edx}$ "+" E_{Edy}	Same for ASCE/SEI 7-10 and EC 8

Table 2 – comparison of design procedures

6 Conclusion

The aim of this thesis was to investigate the development of "performance-based seismic design for industrial buildings". In order to do that each term has been investigated separately.

The first chapters of this thesis deal with the seismic design. The origin, hazard and risks of earthquakes are described in general, followed by an explanation of the seismological basics, which are important to understand the seismic design process. The seismic design process is explained in two chapters, the preliminary design and the calculation of the structural response.

The preliminary design chapter explains design principles of the structural concept and the building configuration and describes structural systems and materials, which are suitable to resist the lateral earthquake force. It is shown that basic principles in the building configuration are to be followed to ensure a proper resistance and seismic behaviour of the structure. This chapter concludes by giving an overview of energy-dissipating devices, which could reduce the earthquake impacts on structure and equipment.

Linear and non-linear methods of the structural response calculation have been explained and the differences between them presented in detail.

The next term investigated was the performance-based design (PBD) approach. The differences to the prescriptive code approach have been pointed out and the process of the PBD has been explained.

Afterwards characteristics of industrial buildings have been described, the equipment categorized and the design of nonstructural components (displacement-, or acceleration-sensitive) explained.

The thesis concludes with a comparison of earthquake-risk and prescriptive seismic design codes of my home country (Austria), the country where I did the research (USA) and the country with the highest earthquake risk in the world (Chile).

7 Commentary

In Austria the earthquake hazard is not big in contrast to Chile and the US. Nevertheless earthquakes are possible, triggered in neighbouring countries or due to mining activities and therefore seismic design issues should be considered in highly populated area. Also Austrian companies which build industrial structures abroad must be familiar with the seismic design and assessment procedure.

The task of seismic engineering is to reduce the seismic vulnerability of existing structures and to avoid vulnerable new constructions due to unawareness or a lack of knowledge.

Advanced software made it possible to generate realistic complex models, which visualize process workflows and the countless intersections/ interactions of nonstructural components and the structure and also to simulate the response of the structure to the seismic excitation without excessive effort.

Despite the use of computer-generated models and simulations, seismic engineering is accompanied with following uncertainties: the actual strength of the earthquake, the actual extent of the damage and inaccuracies in the analysis of the structural and nonstructural response.

To cover these uncertainties, applying the capacity design approach, a critical use of computer-generated results and providing redundant emergency units, back-up and extinguishing systems to quickly restore safe conditions are recommended.

The performance-based design approach is another step forward in the development of the seismic design. By deviating from existing prescriptive codes, it allows the owner of an industrial facility to set higher performance objectives and meeting those by the application of individual, flexible and innovative design solutions.

The next few years will show if the performance-base design approach becomes an integral part of the seismic design practice. To achieve that, the clarity (of the stated performance objectives), the transparency (standardized procedures for quantifying the performance) and the applicability (for the industrial construction) must be improved.

The US organisation FEMA[181] is working on the “Next Generation of Performance-Based Seismic Design Guidelines”[182]. The research project budget is reckoned with $20 million and a period of 10-14 years is estimated.

Furthermore future earthquakes will provide information necessary to validate the newest code requirements for nonstructurals to enhance the research of how nonstrucutral components influence the overall performance of industrial facilities.

[181] Federal Emergency Management Agency
[182] FEMA 445

The research for this topic was challenging but also very interesting for me. Filtering the numerous relevant information and compress it on 100 pages was not easy.

Through the development process of this thesis, I developed myself professionally (became familiar with the seismic design topic, got insights in foreign standards and prescriptive codes) and personally (improved my time management and made lots of new experiences, while doing my research in California).

I hope this thesis will serve as introduction into the seismic design topic and provides an insight on the specific requirements of the industrial construction, easily understandable for students or civil engineers, who have not dealt with this topic so far.

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References

Books:

- Booth, Edmund/Key, David: Earthquake design practice for buildings. Second edition. London: Thomas Telford Publishing 2006
- Lindeburg, Michael R./Baradar, Majid: Seismic Design of Building Structures. A Professional's Introduction to Earthquake Forces and Design Details. Eight edition. Belmont CA: Professional Publications 2001
- Bachmann, Hugo: Erdbebensicherung von Bauwerken. 2., überarbeitete Auflage. Basel: Springer Basel AG 2002

Articles:

- Spataro, Katie/Bjork, Marin/Masteller, Mark: Comparative Analysis of Prescriptive, Performancce-based and Outcome-based Energy Code Systems. Prepared by Cascadia Green Building Council. Alaska: AHFC 2011
- Uzunoglu, Timur/Saragoni, H. Rodolfo/Ansal, Atilla: Structural performance objectives in seismic design of industrial constructions and equipments. Prepared for Vienna Congress on Recent Advances in Earthquake Engineering and Structural Dynamics 2013. Paper No. 125. Vienna: VESSD 2013
- Shihada, M. Samir: Earthquake-Resistant Systems. Gaza: Handout 3-12. 2012
- Heid, Stephan: Die funktionale Leistungsbeschreibung im Baubereich. In: RPA Heft 2-2011
- Mikulits, Rainer/Vogler Franz: Handbuch Bautechnikverordnung 2014. Burgenland, Kärnten, Oberösterreich, Steiermark, Tirol, Vorarlberg, Wien etc. Wien: Linde Verlag 2014
- Bachman, Robert/Dowty, Susan: Nonstructural Component or nonbuilding Structure. In: Building Safty Journal April-May 2008
- ATC/SEAOC Jointventure: ATC-48. Part A: Overview of Component Types and Behavior. In: Build to resist Earthquakes. Briefing Paper 5 – Seismic Response of Nonstructural Components, Redwood City, California: 1998
- Ene, Diana/Craifaleanu Iolanda-Gabriela: Seismicity and Design Codes in Chile: Characteristic Features and a Comparison with some of the Provisions of the Romanina Seismic Code. In: Constructii – Nr. 2/2010, p. 69-78.

Dissertations:

- Williams, Matthew Joseph: Performance Based Analysis of Steel Buildings. [Diss. San Luis Obispo 2009: California Polytechnic State University]
- Adams, Scott Michael: Performance-Based Analysis of Steel Buildings. Special Concentric Braced Frame. [Diss. San Luis Obispo 2010: California Polytechnic State University]
- Holtschoppen, Britta: Beitrag zur Auslegung von Industrieanlagen auf seismische Belastungen [Diss. Aachen 2009: Rheinisch-Westfälische Technische Hochschule Aachen]
- Tafreshi, Kamyar Tavoussi: Zum Tragverhalten von mehrgeschossigen Holzbauten unter Erdbebenbeanspruchung. Übertragung von Versuchsergebnissen an teilbiegesteifen Rahmenecken in praxisnahe Rechenmodelle. [Diss. Wien 2004] Wien: TU Wien 2004; A4-1

Guidelines and Codes:

- Bachmann, Hugo: Erdbebengerechter Entwurf von Hochbauten – Grundsätze für Ingenieur, Architekten, Bauherrn und Behörden. Richtlinien des BWG – Directives de l'OFEG – Direttive dell'UFAEG. Biel: Bundesamt für Wasser und Geologie 2002
- FEMA (ed.): Next-Generation Performance-Based Seismic Design Guidelines. Program Plan for New and Existing Buildings. Prepared by ATC Council. Washington: FEMA 445 2006
- FEMA (ed.): Reducing the Risk of Nonstructural Earthquake Damage- A Practical Guide. Washington: FEMA E-74 2012
- American Society of Civil Engineers (ed.): Minimum Design Loads for Buildings and Other Structures. (ASCE) 7-05. Reston, VA: 2006

- American Society of Civil Engineers (ed.): Minimum Design Loads for Buildings and Other Structures. (ASCE) 7-10. Reston, VA: 2013
- Applied Technology Council (ed.): ATC-69. Reducing the Risks of Nonstructural Earthquake Damage. State-of-the-Art and Practice Report. Cooperation with NEHRP, California: Applied Technology Council 2008
- NEHRP Consultants Joint Venture (ed.): NIST GCR 12-917-18. Comparison of US and Chilean Building Code Requirements and Seismic Design Practice 1985-2010. Redwood City, CA: National Institute of Standards and Technology Oct. 2012
- Austrian Standards Institute (Ed.): Eurocode 8: ÖNORM EN 1998-1. Auslegung von Bauwerken gegen Erdbeben. Teil 1: Grundlagen, Erbebeneinwirkung und Regeln für Hochbauten. Wien: Austrian Standards Institute 2013
- Austrian Standards Institute (ed.): Eurocode 8: ÖNORM EN 1998-4. Auslegung von Bauwerken gegen Erdbeben. Teil 4: Silos, Tankbauwerke und Rohrleitungen. Wien: Austrian Standards Institute 2007
- Instituto Nacional de Normalizacion (ed.): Official Chilean Standard. NCh 433.Of96. Earthquake resistant design of buildings. Santiago: INN 1996
- Instituto Nacional de Normalizacion (ed.): Official Chilean Standard. NCh2369.Of2003. Earthquake-resistant design of industrial structures and facilities. Santiago: INN 2003

Internet Resources:

- http://www.wbdg.org/resources/seismic_design.php: Seismic Design Principles. Gabor Lorant [15.03.2012]
- http://www.tulane.edu/~sanelson/Natural_Disasters/eqhazards%26risks.htm: Earthquake Hazards and Risks. Nelson, Stephen. [09.09.2014]
- http://www.fema.gov/your-earthquake-risk: Your Earthquake Risk. Official website of the Department of Homeland Security. [24.04.2015]
- http://earthquake.usgs.gov/earthquakes/eqinthenews/: Significant Earthquake Archive. US Geological Survey [23.10.2013]
- http://www.nehrp.gov/about/history.htm: About us: Background&History. National Earthquake Hazards Reduction Program. [06.03.2009]
- http://earthquake.usgs.gov/hazards/products/conterminous/ [17.02.2016]
- http://www.seaoc.org/mission [03.11.2015]
- http://skghoshassociates.com/sk_publication/PCI_Jan02_Seis_design_provi_in_US.pdf [04.11.2015]
- https://www.zamg.ac.at/cms/de/geophysik/erdbeben/erdbeben-in-oesterreich/uebersicht_neu: Erdbeben in Österreich – Übersicht. Zentralanstalt für Meteorologie und Geodynamik. [19.11.2015]
- http://education.nationalgeographic.org/encyclopedia/ring-fire/: Ring of Fire. National Geographic Society [24.11.2015]
- http://www.3sat.de/page/?source=/nano/umwelt/152833/index.html: Auf unsicherem Boden – Kernkraftwerke stehen am San-Andreas-Graben. 3sat.online [17.03.2011]
- http://www.spiegel.de/wissenschaft/natur/kalifornien-banges-warten-auf-the-big-one-a-750946.html: Banges Warten auf „The Big One“. SPIEGEL ONLINE. [15.03.2011]