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Portland, Oregon, June 15th 2009,

Kristofer Tonning

Table of Contents

Abstract

The field of earthquake engineering and seismology is of great importance to structural engineers around the world. Only by studying past seismic activity can we predict, with a level of uncertainty, the occurrence of future earthquakes. The effects of previous earthquakes are also of importance when studying and improving seismic restraint systems in structures.

The location, size and consequences of an earthquake are variable depending on several conditions. Surface conditions, boundary/fault type and distance from the boundary and hypocenter are all elements that dictate the outcome of a seismic event. Describing the effects of an earthquake can be difficult. Early records of earthquakes date back to ancient civilizations. Studies of seismic activity were based on descriptive observations. With the introduction of sensitive instruments, the science of seismology has become much more accurate and it is easier to compare seismicity globally.

The seismic design criteria specify the minimum seismic design requirements that are necessary to meet the performance goals established for a specific structure. These minimum requirements are generally outlined in the codes that are in effect at a particular location. In the US, the earthquake design criteria are to conform to a local code in each state, which is usually based on 2006 IBC[1] and ASCE7-05[2]. Throughout the European countries, Eurocode 8 is being implemented as the standard for seismic design.

A key step in developing the design criteria is to determine the peak ground acceleration (PGA). This is easily measured by a seismometer or accelerometer. The ground acceleration will decrease as the distance from the epicenter increases. For this reason attenuation relationships describe the actual ground acceleration at any site, based on the magnitude and distance from the source. This is incorporated

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[1] International Code Council, 2006 *International Building Code*

[2] American Society of Civil Engineers, *ASCE 7-05 Minimum Design Loads for Buildings and Structures*

into the seismic section of building codes, and is generally not addressed in the design process. ASCE 7-05 uses mapped acceleration parameters that are obtained from the 0.2 and 1.0 s spectral response accelerations shown on maps prepared by the US Geological Survey. The Eurocode uses the peak ground acceleration as the basis for the design spectrum, and these values are given on maps in the National Annex of the code.

The International Building Code (IBC) is the authority of structural provisions used in the United States. Due to the comprehensiveness of this code, most of the seismic provisions are given in a publication by the American Society of Civil Engineers (ASCE 7-05). The European code is reviewed with emphasis on the provisions for Norway given in the National Annex. The Norwegian Standard (NS 3491-12) will not be discussed here, because it is no longer the most current code used in design and it is also largely based on the Eurocode.

The seismic criteria adopted by current codes involve a two-level approach to seismic hazard. The basic criterion in Eurocode 8 is a level of ground shaking that has a 10% probability of being exceeded in 50 years (475 year return period).[3] This return period has also been used to define design basis earthquake in several of the primary building codes in the United States that preceded the new International Building Code (IBC). The 2006 IBC, through reference to the ASCE 7–05, uses two-thirds of the maximum considered earthquake (MCE) as the design earthquake. In the United States, the MCE is defined as an event with an approximate 2,500-year return period (2% probability of exceedance in 50 years).[4]

Although the two codes have certain differences, it is clear that they are both based on a common understanding of earthquake behavior. The science behind the provisions are founded on common scientific ground, and even though the analysis approach differ in context, the results achieved closely correlate.

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[3] Naeim, F., *The Seismic Design Handbook*. Section 14.7

[4] http://www.irmi.com/Expert/Articles/2007/Gould03.aspx

Preface

With very little seismic activity in Norway, the requirements for Norwegian engineers to master seismic design have in the past been limited. The introduction of the Eurocode to the Norwegian standardization system, has presented a need to investigate the contents of this code in order for it to be properly implemented.

The purpose of this thesis is to give an overview of the field of earthquake engineering and seismic design, and a detailed study of the codes. The Eurocode will be compared to the code used in the United States, in order to investigate the different approaches to earthquake engineering. This will be done to uncover the background of the design criteria used in Europe and America.

The content of this thesis is based on literary studies, a comparative analysis of European and American codes, and case studies where the different codes are applied. An evaluation and comparison of the results will be provided to uncover any discrepancies in the methods.

When presented with the opportunity of writing a thesis in the United States, the topic of seismic engineering stood out as a field of interest. Working with a consulting engineering company on the west coast, where problems of seismic design are commonplace, has given valuable experience that has been applied in the process of writing this thesis.

This document is also meant to give graduate students of structural engineering an entry-level understanding of the design and detailing of steel and concrete structures for earthquake resistance.

1 Introduction to Seismology

1.1 Background

The field of earthquake engineering and seismology is of great importance to structural engineers around the world. Only by studying past seismic activity can we predict, with a level of uncertainty, the occurrence of future earthquakes. The effects of previous earthquakes are also of importance when studying and improving seismic restraint systems in structures. This section will give an introduction to important terms and concepts that define the science of earthquake engineering.

The location, size and consequences of an earthquake are variable depending on several conditions. Surface conditions, boundary/fault type and distance from the boundary and hypocenter are all elements that dictate the outcome of a seismic event. Describing the effects of an earthquake can be difficult. Early records of earthquakes date back to ancient civilizations. Studies of seismic activity were based on descriptive observations. With the introduction of sensitive instruments, the science of seismology has become much more accurate and it is easier to compare seismicity globally. Both qualitative and quantitative reports are now used to describe ground motions and their effects.[5]

1.2 Seismic Hazards

An earthquake is one of few naturally occurring events that can have devastating and tragic results. The most important hazards relating to seismic activity can be identified and sectioned as follows:

Ground shaking is caused by seismic waves that radiate from the source and travel through the crust of the earth. When the waves reach the surface, they produce shaking that can cause severe damage. Ground shaking can be considered the most important hazard, because it is the cause of all the other seismic hazards.

Structural hazards are those we most commonly associate with earthquakes. The damage and collapse of buildings and other structures is the leading cause of death

[5] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 1.2

and economic loss in earthquakes. In the last few years, advances in seismic design has improved the seismic restraint systems and moved the focus of design from purely strength to a combination of strength and ductility. This has led to a need for more accurate ground motion predictions and codes have been issued in frequent and thorough revisions to accommodate this.

Liquefaction is a phenomenon in which the strength of the soil is drastically reduced, to a point where it is unable to support structures. These events only occur in loose, saturated sand, near river, lakes or other bodies of water.

Earthquake induced landslides can occur as a result of liquefaction. The soil on slopes can also fail due to ground shaking even when the soil is stable under static conditions. The landslides are often relatively small, but in some cases entire towns and villages have been buried by the rogue masses. A majority of destructive landslides cause damage by destroying buildings, bridge sections and other structures in their path.

A tsunami is a long period wave produced by a rapid vertical seafloor movement. These movements are caused by a fault rupture during an earthquake. Even though these waves usually have a height of less than a meter in the open sea, their height drastically increases as the waves approach shore. The geometry of the seafloor in some areas can amplify the wave and devastating damage can occur when the wave strikes land.

In enclosed bodies of water, earthquake induced waves can cause a phenomenon known as a seiche. The effect is caused by the resonance that occurs when long period waves match the natural period of oscillation of the water in the basin. A standing wave causes the water level to significantly drop in one area of the reservoir and drastically increase in another.[6]

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[6] http://en.wikipedia.org/wiki/seiche

The science of earthquake engineering involves the mitigation of seismic hazards. This is embedded in the process of earthquake resistant design. However, only a few of these hazards can be accounted for in the design of buildings. Only the effects of ground shaking on structures are dealt with when designing for earthquake resistance. Damages to buildings that are not caused by the direct effects of ground motion, i.e. damages due to earthquake-induced phenomena, are difficult to predict.[7]

Fig. 1.1: *1906 San Francisco Earthquake. Ruins in vicinity of Post and Grant Avenue. (Wikipedia)*[i]

1.3 Significant Historical Earthquakes

The recorded earthquakes of the past are of significant importance to us for several reasons. It has furthered our understanding of the phenomenon, both in terms of the natural science of the earth and the social ramifications of affected communities. As a result, the devastating consequences of seismic events have been recognized, and measures of moderating these effects have been promoted.

The 1906 San Francisco earthquake (see Fig. 1.1) is perhaps the most well known and is recognized as the first great earthquake to strike a densely populated area in

[7] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 1.3,1.4

the US. Although ground shaking caused significant damage, most of the harm was caused by subsequent fire initiated by ruptured gas mains.[8]

The 1985 Mexico City earthquake has left us with pictures of disfigured reinforced concrete components, which many now associate with earthquake damages. Only buildings of a certain height (see Fig. 1.2), and hence stiffness, were affected. This illustrates the importance of understanding the effects of a building's natural period and subsequent danger of resonance in a seismic event.

Fig. 1.2: *Close-up of failed member of Juarez Hospital, 1985 Mexico City earthquake. (*Western Washington University/ *USGS)*[ii]

Earthquakes in Japan, China, Iran and Pakistan with devastating damage have been observed over the last few years. Every event teaches scientists more about earthquake effects. Engineers can use the acquired information to better understand the lateral loads imposed on buildings (see Fig. 1.3), and to further the design of earthquake resistant systems.

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[8] http://en.wikipedia.org/wiki/san_fransico_earthquake

Fig. 1.3: *Diagonal cracking beams and pier columns. 2008 Sichuan earthquake. (Wikipedia)*[iii]

1.4 Internal Structure of the Earth

The earth has a layered structure. The inner core is surrounded by the outer core, which in turn is enclosed by the mantle. The crust is the outermost layer that covers it all, and is the surface on which we live. The temperature of each layer increases with depth. The temperature gradient in the mantle causes the semi-molten rock to move slowly by convection.

In a seismic event, two different types of seismic waves are produced. Body waves travel through the interior of the earth and are categorized by two types of waves, p-waves and s-waves. The p-waves are longitudinal waves that involve successive compression and rarefaction of the materials they travel through. The s-waves are transverse waves that cause shear deformations in the materials they pass through. The p-waves travel faster than any other seismic waves and are therefore the first waves to arrive at a particular site. The s-waves cannot travel through fluids because they have no shear stiffness, and can subsequently not travel through the core.

Surface waves result from the interaction of body waves and the surface layers of the earth. These waves are more common at distances farther from the source of the earthquake and will produce peak ground motions if the distance is great enough.

The most important surface waves for engineering purposes are the Rayleigh waves and the Love waves. Whereas Rayleigh waves involve both vertical and horizontal particle motion, the Love waves have no vertical component.[9]

1.5 Plate Tectonics

The crust is broken into a number of large plates and smaller platelets. Lateral movement of the mantle causes shear stresses on the bottoms of the plates. Together with gravitational forces, the stresses cause the plates to move with respect to each other.

Relative movement of the plates causes stresses to build up on their boundaries. As movement occurs, strain energy accumulates near the boundaries. This energy is eventually released either smoothly and continuously, or in a stick-slip manner that produces earthquakes.

There are three different types of plate boundaries (see Fig. 1.4) and their nature influence the amount of strain energy that can build up in their vicinity. As a result, the different types of boundaries have different earthquake characteristics. Subduction zone boundaries have the potential of producing the largest earthquakes, followed by transform fault boundaries and spreading ridge boundaries.

A subduction zone boundary is one where two plates move toward each other, and their respective movements cause one plate to ride over the other. If one plate is oceanic, it will sink by its own weight beneath the lighter continental plate. Two colliding continental plates lead to the formation of mountain ranges along the interface. Earthquakes are generated at this interface between the two plates.[10]

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[9] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 2.2

[10] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 2.3

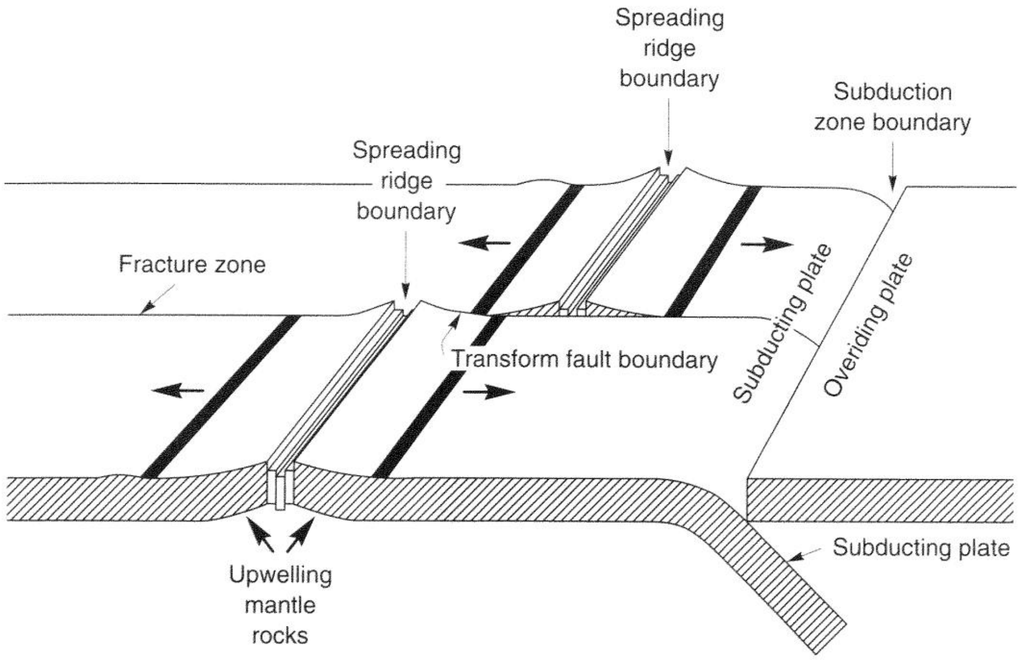

Fig. 1.4: *Spreading ridge, subduction zone, and transform fault boundaries. (Kramer)*[iv]

1.6 Faults

The surfaces on which relative movements occur are called faults. Faults may range in length from several meters to hundreds of kilometers. Locations of faults can be obvious to observers or they can be very difficult to detect. At a particular location, a fault is assumed to be planar with an orientation described by its strike and dip. The presence of a fault does not necessarily mean that an earthquake is to be expected, because movements can occur aseismically or the fault can be inactive.

The orientation of fault movement is described by dip-slip and strike-slip components, indicating the normal and reverse faulting and left lateral and right lateral faulting. It has been suggested that earthquakes should most likely occur along portions of a fault for which little seismic activity has been observed unless movements have occurred aseismically.[11]

[11] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 2.4

1.7 Elastic Rebound Theory

The plates of the earth are in constant motion, and the majority of their relative movement occurs near their boundaries. This movement causes elastic strain energy to be stored in the material near the fault as a result of increased shear stresses on the fault planes. When the level of shear stresses along a fault reaches the shear capacity of the rock, the accumulated strain energy is released as the rock fails. Depending on the nature of the rock, the outcome of this release has different effects. If the rock has weak and ductile properties, only a small amount of energy will build up. The stored energy will then be released slowly and movement will occur without the event of an earthquake. However, if the rock is strong and brittle, vast amounts of energy can build up leading to a rapid release. This type of rupture will form the characteristic waves of an earthquake. The process of buildup and subsequent release of strain energy in the rock near faults is described by the elastic rebound theory.

The material properties of the rock along a fault are not uniform, and the surface of a fault can have both weak and strong zones. Various models describe the mechanisms of a rupture. It is presumed by the *asperity model,* that stresses are not uniformly distributed across a fault. This is because some stresses will be released by the weaker zones prior to stress release by the stronger zones. The *barrier model*, on the other hand, assumes that the stresses are uniform. In a seismic event, only the weaker zones release the stresses. The stresses in the fault plane then redistribute, and the rock adjusts to accommodate a new uniform stress level. In reality it appears that some strong zones behave as asperities and some as barriers. From an engineering perspective, the importance of the strong zone behavior lies in the influence it has on ground shaking characteristics close to the fault.

The elastic rebound theory indicates that the occurrence of earthquakes will relieve some stresses along a portion of a fault. The segment will then need time to build up sufficient energy for another earthquake. The probability of a seismic event should therefore be related to the time that has passed since the last earthquake. This means that an earthquake in a particular portion of a fault is considered not to be a random event, and it would be more likely for an earthquake to occur in portions of a fault with little or no recorded seismic activity.

The seismic moment can be developed from the concepts of elastic rebound theory. It is a measure of the work done by an earthquake and correlates well with the energy released by it. The seismic moment is a good indication of the magnitude of an earthquake and is the basis of the moment magnitude scale, which corresponds closely to the Richter scale.[12]

1.8 Earthquake location and size

In order to accurately describe the location of an earthquake, there are certain terms that must be defined. The hypocenter is the location where the rupture initiates. From the hypocenter, the rupture spreads along the fault and can involve thousands of square kilometers of fault plane surface. The epicenter is the point on the ground surface directly above the hypocenter.

The location of an earthquake is usually specified by the location of the epicenter. In order to pinpoint the location of the epicenter with a certain degree of accuracy, three different seismographs must determine the epicentral distance to the earthquake. The seismographs can determine the distance, but not the direction of the earthquake. When measurements from the three seismographs are recorded on a map, the three circles, representing the radial distance from the seismograph, will intersect at the point of the epicenter.

The size of an earthquake can be measured by its intensity, magnitude or energy. The intensity is the oldest measure, and is a qualitative description of the observed damage and human reactions as a result of a seismic event. Since the measure does not rely on instrumental records, this can be used to describe historical earthquakes that took place before the development of modern technology. In this manner, ancient accounts of earthquakes can be compared to more recent earthquakes and an estimate of the earthquake size can be determined. The most

[12] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 2.5

common scales are the Rossi-Forel, the modified Mercalli intersity (MMI), the Japanese Meteorological Agency (Shindo) and the Medvedev-Spoonheuer-Karnik (MSK).[13]

As modern technological advances led to the development of seismic instruments, a more objective, quantitative measure of earthquake size was made available. There are several important magnitude scales. Most famous is the local magnitude scale, which is also known as the Richter scale. Other scales include the surface wave magnitude, the body wave magnitude, and the moment magnitude. The former scales, however, have some weaknesses, as they do not accurately reflect the size of very large earthquakes. They are closely related, but experience a phenomenon known as saturation, where the scales become imprecise as amplitudes of the described waves tend to reach limiting values (see Fig. 1.5). The moment magnitude, which is not obtained from ground motion characteristics, is able to describe the size of any earthquake. The energy released during an earthquake can be described by a relationship that is closely related to the moment magnitude.[14]

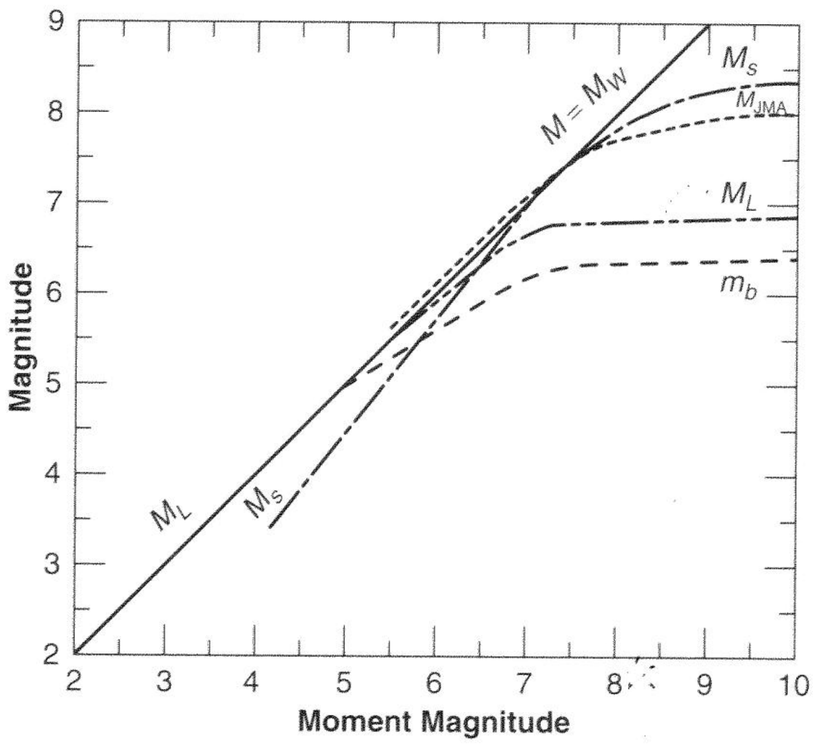

Fig. 1.5: *Correlation of the various magnitude scales with saturation at higher values.* M_W *(moment magnitude),* M_L *(Richter local magnitude),* M_S *(surface wave magnitude),* m_b *(short-period body wave),* m_B *(long-period body wave), and* M_{JMA} *(Shindo). (Kramer)*[v]

[13] http://en.wikipedia.org/wiki/seismic_scale

[14] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 2.7, 2.8, 2.9, 2.10

1.9 Strong Ground Motion

1.9.1 Strong Motion Measurement

At any given point, the motion produced by an earthquake can be described by three components of rotational motion and three components of translational motion. Orthogonal, translational components are most commonly measured, and the three rotational components are usually neglected.

Strong-motion measurements can be made using a number of different instruments. The dynamic response characteristics of each instrument determine the conditions for which they are best suited. Seismographs are used for measurements involving relatively weak ground motion, whereas strong ground motion is recorded using accelerographs. The latter is of more interest to structural engineers, since strong ground motion is more relevant in seismic design.

In recent year, digital seismographs and accelerographs have been used for field measurements of earthquakes. The raw strong motion data, measured by the sensitive instruments, may include background noise from several different sources. These errors can be caused by anything from traffic to wind, and correction of the data is required to produce accurate strong motion records. Strong motion processing is often required to minimize background noise and to correct for other measurement errors.[15]

1.9.2 Strong Motion Parameters

The complete description of strong ground motion can be quite complicated and involves a large amount of data. For engineering purposes, three characteristics of earthquake motion are of importance. The amplitude, frequency content and duration of the motion all play major roles in the effects of ground motion on structures under consideration. These essential characteristics of a strong ground motion can be described in much more compact form using ground motion parameters. Some parameters describe one of the characteristics, while others can describe two or three of them.

[15] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 3.1, 3.2

The amplitude is often measured by peak acceleration, peak velocity and peak displacement. The peak acceleration gives a good indication of the high-frequency components of ground motion. The amplitudes of the intermediate- and low-frequency components are described by the peak velocity and peak displacement. The vertical component of the ground motion have received less attention in structural engineering, because the design of structures for gravity loads usually gives adequate resistance for the vertical dynamic forces induced by earthquakes. For this reason the horizontal components are more interesting, and most seismic design involves lateral resistance.

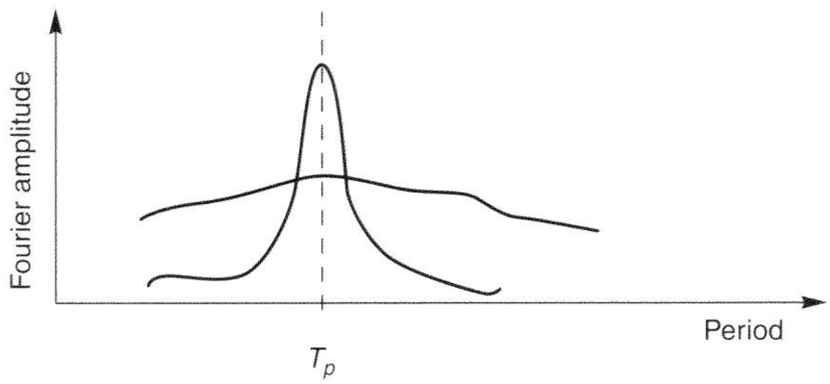

Fig. 1.6: *Two Fourier amplitude spectra with the same predominant period, but very different frequency content. (Kramer)*[vi]

The frequency content of strong ground motion is described by using different types of spectra. Fourier spectra (see Fig. 1.6 and Fig 1.7) and power spectra directly illustrate the frequency content of the ground motion itself. Response spectra, on the other hand, reflect the influence of the ground motion on structures of different natural periods. A variety of spectral parameters are available to describe the frequency content of strong ground motion. Among these parameters are the predominant period, bandwidth, central frequency, shape factor and Kanai-Tajimi parameters (see Fig. 1.6 and Fig. 1.8).

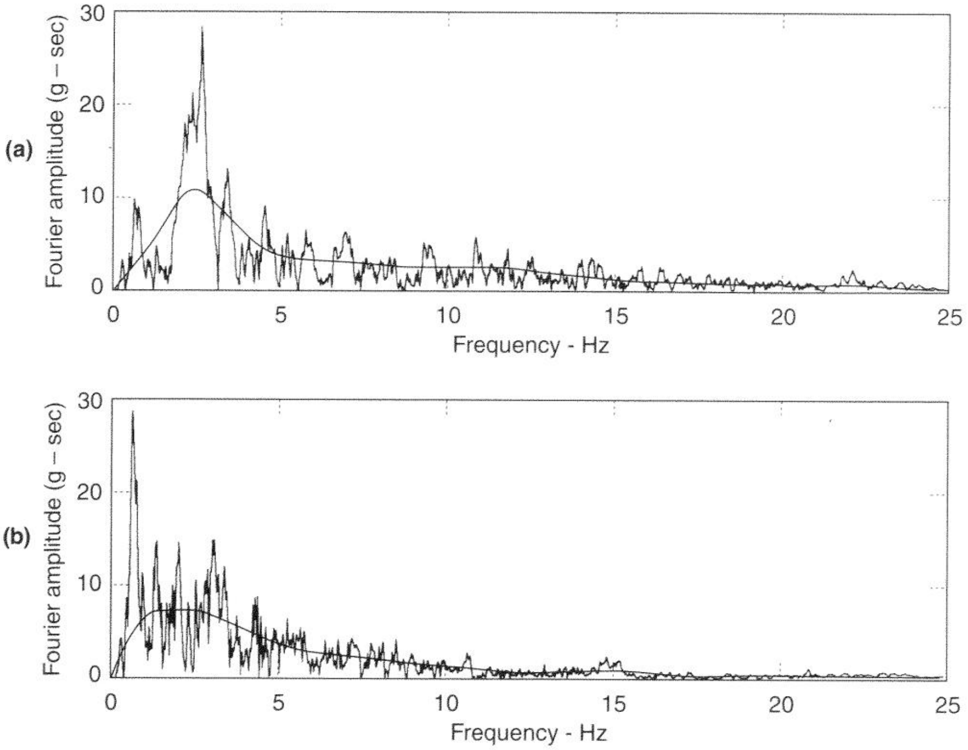

Fig. 1.7: *Raw and smoothed Fourier amplitude spectra for two different ground motions. (Kramer)*[vii]

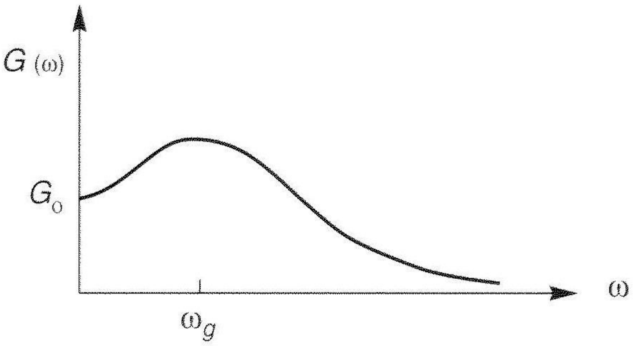

Fig. 1.8: *Shape of the Kanai-Tajimi power spectral density function. (Kramer)*[viii]

The duration of strong ground motion has a significant effect on the degree of damage caused by an earthquake, because the number of load or stress reversals is critical to the degradation of a structure's stiffness and strength. A motion of short duration may not produce enough load reversals for damaging response to build up, even if the amplitude of the motion is high. On the other hand, motion of moderate amplitude but with long duration can produce enough load reversals to cause significant damage.

Time between the first and last exceedances of threshold acceleration is known as bracketed duration, and is based on an absolute measure of the acceleration. The bracketed duration is the measure most commonly used for engineering purposes, because it reflects the strength of shaking.[16]

1.9.3 Estimation of Ground Motion Parameters

Design of earthquake resistant structures requires estimation of the level of ground shaking they will be exposed to. The level of shaking is most conveniently described in terms of the ground motion parameters mentioned above, and methods for estimating these parameters are required. So-called predictive relationships express a particular ground motion parameter in terms of the quantities that affect it, such as magnitude and distance. These relationships are used to estimate the ground motion parameters, and they therefore play an important role in seismic hazard analyses.

Peak ground acceleration is the most commonly used ground motion parameter, and is, as mentioned earlier, a measure of the amplitude. Of course, the peak acceleration will decrease with increasing distance, and the approximate predictive relationships for parameters such as these are often recognized as attenuation equations. Many such equations have been developed over the years, and refined as more strong motion data has become available.[17]

The following equation for peak horizontal acceleration was developed by Campbell in 1981, and is a good example of what equations for predictive relationships look like. This is a relatively simple relationship that takes into account the local or surface wave magnitude, M, and the closest distance, R, to the fault rupture of the earthquake.

$$\ln PHA\,(g) = -4.141 + 0.868\,M - 1.09\,\ln\,[R + 0.0606\,\exp\,(0.7\,M)]$$

[16] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 3.3

[17] Lindeburg, M.R., *Seismic Design of Building Structures*, Section 23

The frequency content of a ground motion is related to the earthquake magnitude. Large earthquakes produce larger and longer-period ground motions than smaller magnitude earthquakes. When seismic waves travel away from a fault, their higher-frequency components are scattered and absorbed more rapidly than their lower-frequency components. Consequently, the frequency content also changes with increasing distance.

With response spectra extensively being used in earthquake engineering, their importance has led to the development of methods for predicting them directly. Previously, the shapes of all response spectra were assumed to be identical. Design spectra were developed by scaling average spectral shapes upward or downward by some ground motion parameter depending on the magnitude of the earthquake. As more recorded data was made available, the magnitude dependence of spectral shapes was recognized and accelerograms were introduced as a tool for computing response spectra more accurately.

The duration of strong ground motion increases with increasing earthquake magnitude. Furthermore, strong motion duration based on absolute acceleration levels, such as bracketed duration, would be expected to decrease with distance. Since acceleration amplitudes decrease with distance, all accelerations will drop below the threshold acceleration at some point and the bracketed duration will be zero. For engineering purposes, the bracketed duration appears to provide the most reasonable indication of the influence of duration on potential damage.[18]

More importantly, for a longer duration of strong ground motion, more energy will be transferred to a structure. Since a structure can absorb only a limited amount of elastic strain energy, a longer duration earthquake has a greater chance of driving structural performance into inelastic behavior.[19]

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[18] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 3.4

[19] Lindeburg, M.R., *Seismic Design of Building Structures*, Section 33

1.10 Seismic Hazard Analysis

Seismic hazard analysis is a tool that is used to determine the design ground motion, which describes the level of shaking that occurs during an earthquake. Its importance is significant in earthquake resistant design, where the goal is to produce structures that can withstand a certain level of shaking without excessive damage.

Seismic hazard analyses involve estimation of ground motion characteristics at a particular site, and require the identification and characterization of all potential seismic sources that could produce significant ground motions. The analyses may be conducted deterministically, where a particular earthquake scenario is assumed, or probabilistically, where uncertainties in earthquake size, location and time of occurrence are taken into account.[20]

1.10.1 Deterministic seismic hazard analysis (DSHA)

For an earthquake event where ground motion characteristics are determined, the use of deterministic seismic hazard analyses is commonplace. DSHAs often assume that earthquakes of the largest possible magnitude occur at the shortest possible distance to the site. The earthquake that produces the most severe site motion is then used to compute site-specific ground motion parameters. In areas with relatively frequent occurrence of earthquakes, such as on the coast of California, deterministic values for the design earthquake are used.

The DSHA approach provides a simple framework for evaluation of worst case ground motions when applied to structures where failure could have catastrophic consequences. However, it provides no information on the probability of occurrence of the design earthquake, the likelihood of it occurring where it is assumed to occur, the level of shaking that might be expected during a finite period of time, or the effects of uncertainties in the various steps required to compute the resulting ground motion characteristics.[21]

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[20] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 4.1

[21] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 4.3

1.10.2 Probabilistic seismic hazard analysis (PSHA)

In the evaluation of seismic hazards, probabilistic seismic hazard analyses explicitly consider uncertainties in the size, location, rate of recurrence, and effects of earthquakes. A PSHA requires quantified uncertainties in earthquake location, size, recurrence, and ground shaking effects. For each source zone, uncertainty in earthquake location is characterized by a probability density function of source-to-site distance. Evaluation of the probability density function requires estimation of the geometry of the source zone and of the distribution of earthquakes within it.

Various recurrence laws can describe the uncertainty in the size of earthquakes produced by each source zone. The Gutenberg-Richter recurrence law, which assumes an exponential distribution of magnitude, is commonly used with modifications to account for minimum and maximum magnitudes. The law is described by the relationship:

$$\log \lambda_m = a - bM$$

where λ_m is the mean annual rate of exceedance, M is the earthquake magnitude, and a and b are certain probabilistic values. The return period of an earthquake is consequently given by:

$$T_R = 1/\lambda_m.$$

The probabilities of earthquakes of various sizes occurring in finite periods of time are usually computed assuming that earthquakes occur as Poisson processes. The model is expressed by the equation:

$$P = 1 - \exp(-\lambda t)$$

where P is the probability of exceedance, λ is the annual rate of exceedance, and t is a certain time period. Although the Poisson model assumes an independence of events that is not consistent with elastic rebound theory, it remains the most commonly used model in modern PSHA.[22]

To compute ground motion levels with various probabilities of exceedance in different periods of time, standard methods of probability analysis can be used to combine the uncertainties in earthquake size, location, occurrence, and effects. Because of the complex and empirical nature of the probability density functions, exceedance probabilities are usually computed by numerical, rather than analytical methods. Seismic hazard curves show the mean annual rate of exceedance of a particular ground motion parameter and are the ultimate result of a PSHA. A hazard curve can be used to calculate the probability of exceedance of some peak ground acceleration in a certain time period, and the associated return period can hence be determined. In the same manner, the peak acceleration with a certain probability of being exceeded in a given time period can be found.[23]

[22] http://www.ce.washington.edu/~geotech/courses/cee526/arduino/chapter4c.pdf

[23] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 4.4

2 Translating Ground Motions into Seismic Loads

2.1 Design Criteria for Response Analysis

2.1.1 Selection of Seismic Design Criteria

The seismic design criteria specify the minimum seismic design requirements that are necessary to meet the performance goals established for a specific structure. These minimum requirements are generally outlined in the codes that are in effect at a particular location. In the US, the earthquake design criteria are to conform to a local code in each state, which is usually based on 2006 IBC[24] and ASCE7-05[25]. This will be covered in more detail in Section 3.

A key step in developing the design criteria is to determine the peak ground acceleration (PGA). This is easily measured by a seismometer or accelerometer. The PGA values are most commonly specified as a fraction of the gravitational acceleration, g. As mentioned in the previous section, the ground acceleration will decrease as the distance from the epicenter increases. For this reason attenuation relationships describe the actual ground acceleration at any site, based on the magnitude and distance from the source. This is incorporated into the seismic section of building codes, and is generally not addressed in the design process. ASCE 7-05 uses mapped acceleration parameters that are obtained from the 0.2 and 1.0 s spectral response accelerations shown on maps prepared by the US Geological Survey (see Fig 2.1).[26]

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[24] International Code Council, 2006 *International Building Code*

[25] American Society of Civil Engineers, *ASCE 7-05 Minimum Design Loads for Buildings and Structures*

[26] ASCE 7-05, Chapter 22

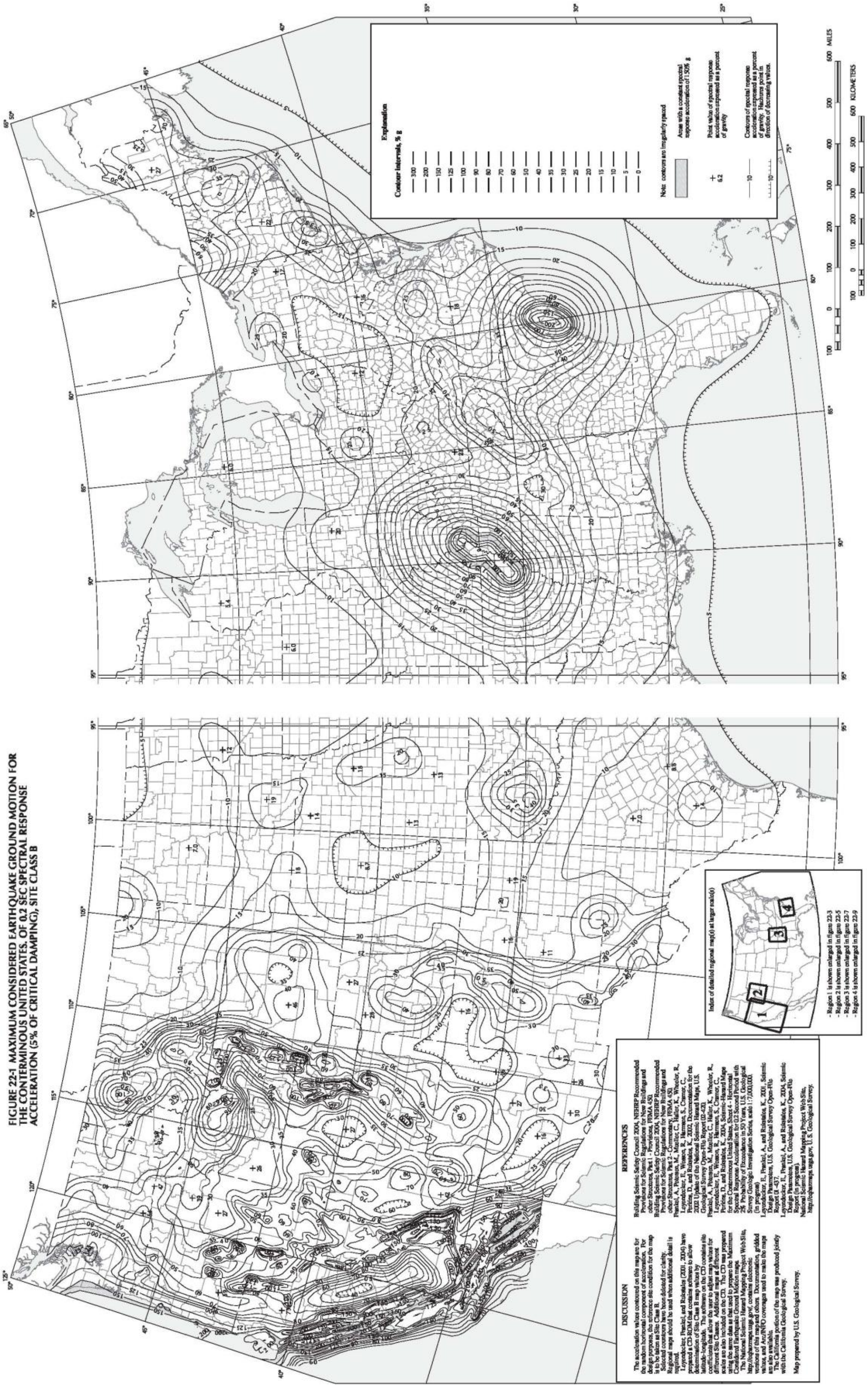

Fig. 2.1: *Maximum considered earthquake ground motion at 0.2s spectral response acceleration (5% of critical damping). (USGS/ASCE 7-05)*[ix]

2.1.1.1 Selection of Safety Level (Return period for the Earthquake)

The seismic criteria adopted by current codes involve a two-level approach to seismic hazard. The Design Basis Earthquake (DBE) describes the level of ground shaking that has a 10% probability of being exceeded in 50 years (475 year return period).[27] This return period has long been used to define design basis earthquake in several of the primary building codes in the United States that preceded the new International Building Code (IBC). The 475 year return period is also used as a basic criterion in Eurocode 8[28], and in the Norwegian Standard NS 3491-12. The 2006 IBC, through reference to the ASCE 7–05, uses two-thirds of the maximum considered earthquake (MCE) as the design earthquake. In the United States, the MCE is defined as an event with an approximate 2,500-year return period (2% probability of exceedance in 50 years).[29]

The redefinition of the design earthquake in the IBC is intended to provide a more uniform level of safety across the country. This makes the spectral accelerations corresponding to the two safety levels quite different for Eastern and Western United States. The MCE is only 50 percent larger than the DBE in coastal California, but it can be four or five times as large as the DBE in the Eastern United States. This means that for the West Coast, the two safety levels give accelerations that are closely related, and the design values for the MCE are only slightly more conservative. However, in Eastern United States, the 2,500 year return period account for more severe ground shaking, which give much more conservative design values than the previous use of the return period of 475 years. [30] The decision to increase the level of safety is a result of shifting the design focus from only being concerned with life safety to also incorporate collapse prevention.

In Easter United States, the local values of ground motion at a return period of 475 years are so small that they usually do not control the lateral design. However, this

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[27] Naeim, F., *The Seismic Design Handbook*. Section 14.7

[28] CEN, *Eurocode 8*, Section NA.2.1

[29] http://www.irmi.com/Expert/Articles/2007/Gould03.aspx

[30] ICBO Staff, UBC-IBC Structural (1997-2000), Section 1613-1623

region has a potential for very severe ground motion, which was not accounted for when the 475 year return period was used. When the return period used for seismic design was increased to 2500 years, significantly larger earthquakes where incorporated in the seismic hazard. Using some simple calculations, the effect of this change on the design values can be presented in a more understandable form.

DE = design earthquake for 2500 year return period
MCE = maximum considered earthquake for 2500 year return period
DBE = design basis earthquake for 475 year return period
$DE_{(2500)}$ = 2/3 MCE, per definition

In Coastal California:
MCE is approx. 50% larger than DBE:
$DE_{(2500)} = 2/3\ MCE = 2/3 \times 1.5 \times DBE_{(475)} = DBE_{(475)}$
(More or less the same)

In Eastern United States:
MCE is approx. 4 or 5 times larger than DBE:
$DE_{(2500)} = 2/3\ MCE = 2/3 \times 5 \times DBE_{(475)} = 3.3 \times DBE_{(475)}$
(More than 3 times larger than before)

The new design values used in California are about the same as before. This is partially because earthquakes occur relatively frequently, and there is therefore a lot of available data describing the local ground motion. In Eastern United States, the new values for ground acceleration are much higher, and seismic loads must now be accounted for in the design of buildings. This does not mean that the design earthquake in Easter United States is 3 times higher than in California. The design earthquake for the 2500 year return period in California is still higher than anywhere else in the US, but the difference is not as great as it once was.

2.1.1.2 Selection of Importance Factors for Structural Design

The process of seismic design using the 2006 IBC involves determining a series of factors and parameters that will be applied in the final analysis. One essential factor is the Seismic Importance Factor, which represents an attempt to control the seismic performance capabilities of buildings in different occupancy categories by assigning a higher safety level to buildings that hold a large number of people or that are essential for the community in an emergency situation. This factor modifies the minimum base shear forces and reflects the relative importance assigned to the occupancy during and following an earthquake.

The seismic importance factor is assigned to each structure based on the Occupancy Category, which is described in the codes.[31] Most structures fall into Occupancy Category II and are assigned I = 1.0. The same importance factor pertains to buildings in Occupancy Category I, which represent a low hazard to human life, such as agricultural buildings and minor storage facilities. Occupancy Category III includes buildings that hold a large number of people and are assigned I = 1.25. Also included in this category are power plants, water treatment and sewage facilities, as well as telecommunication centers and other structures that have a potential to cause a substantial disruption in civilian life. Structures in Occupancy Category IV are hospitals, emergency care units, emergency response stations, and other essential facilities. Due to their significance in an emergency situation, these structures are assigned I = 1.5.

As a result of the use of these factors, the design seismic force will increase by 25% when using I = 1.25 and 50% when using I = 1.5. Both the 2006 IBC and Eurocode 8 define the Occupancy Categories in much the same way. Values for the importance factors differ from country to country, but the overall classifications remain the same. The determination and use of the different factors and parameters in the codes will be covered in more detail in Section 3.[32]

2.1.2 Local Site Effects and Design Ground Motions

Local site effects play an important role in earthquake resistant design and must be specifically accounted for in each design situation. This is usually accomplished by

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[31] *ASCE 7-05*, Table 1-1 / CEN, *Eurocode 8*, Table 4.3

[32] *CodeMaster: Seismic Design/ASCE 7-05, Section 11.5*

developing one or more design ground motion time histories. These motions reflect the levels of strong motion amplitude, frequency content, and duration that a structure at a particular site should be design for.[33]

2.1.2.1 Effects of Local Site Conditions on Ground Motion

Local site conditions can significantly influence amplitude, frequency content, and duration, which are all important characteristics of strong ground motion. The geometry and material properties of the subsurface materials, as well as on site topography, affect the extent of the influence of the conditions on these characteristics. The nature of local site effects can be illustrated in several ways, using either a theoretical approach or measured surface and subsurface motion time histories.

There are several theoretical reasons why ground surface motions are influenced by local site conditions. Since the density and surface wave velocity varies in different materials, it is obvious that ground motions are site dependent. The characteristics of local soil deposits can also influence the extent of ground motion amplification that will occur at a particular site. A more realistic description of local site conditions should therefore include the density and stiffness of the soil and the bedrock.

[33] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 8.1

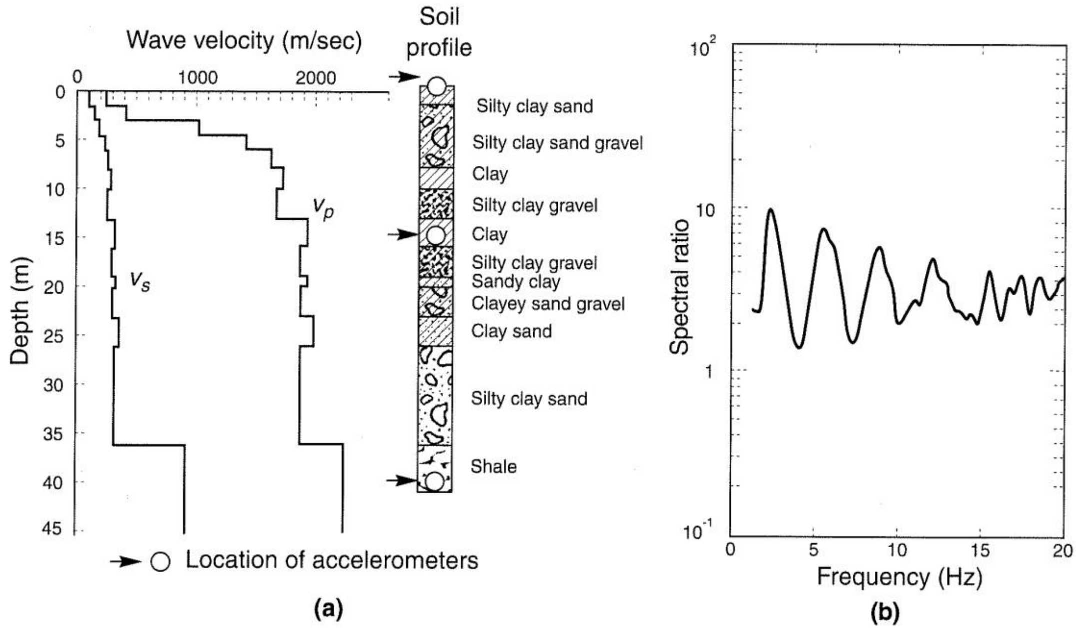

Fig. 2.2: *(a) Subsurface soil profile (b) Surface-bedrock amplification function. (Kramer)*[x]

Actual amplification functions can be computed by interpreting strong motion data from sites where both surface and subsurface instruments have been installed. The importance of local soil conditions on ground response is clearly illustrated by the strong amplification at the natural frequencies of soil deposits shown in Fig. 2.2. The frequency dependence of the actual amplification function is qualitatively similar to that predicted by the simple analyses of the theoretical approach.

The importance of local site conditions is underlined when comparing ground surface motions measured at different sites. Variations in ground motion, expressed in terms of peak horizontal acceleration and response spectra, are shown in Fig. 2.3 along with variations in soil conditions along a 4-mile section through San Francisco during an earthquake in 1957.

Similar effects have been observed in many other earthquakes, one of which being the 1985 Mexico City earthquake. This earthquake, which was of magnitude M_S = 8.1, caused only moderate damage near its epicenter. However, the damage in Mexico City, which was 350 km away from the epicenter, was extensive. Studies of ground motion records at different sites in Mexico City illustrated the significant relationship between local soil conditions and damaging ground motions.

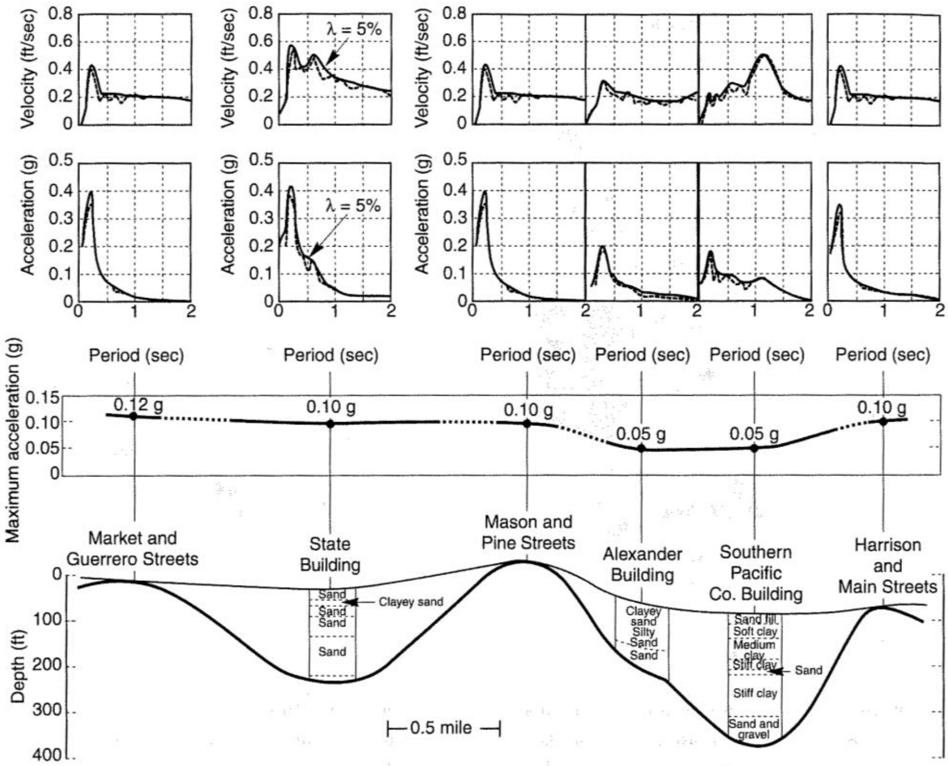

Fig. 2.3: *Variation of spectral velocity, spectral acceleration, and peak horizontal acceleration along a 4-mile section of through San Francisco in the 1957 San Francisco earthquake. (Kramer)*[xi]

The structural damage in Mexico City during the 1985 Michoacan earthquake was highly selective. Large parts of the city experienced no damage while other areas suffered major damage. The greatest damage occurred in certain zones that consisted of 38 to 50 m of soft soil, where the characteristic site periods were estimated at 1.9 to 2.8 s. Even within this area, damage to buildings of less than five stories and modern buildings greater than 30 stories was minor. Most buildings in the five- to 20-story range, on the other hand, either completely collapsed or were badly damaged. Using the rough rule of thumb stating that the fundamental period of an N-story building is approximately N/10 s., it can be estimated that most of the damaged buildings had a fundamental period equal to or slightly less than the characteristic site period. It seems likely that the damaged structures were subjected to many cycles of large dynamic forces at periods near their fundamental periods. This resonance condition, combined with structural design and construction deficiencies, caused locally devastating damage.

Local site conditions strongly influence peak acceleration amplitudes and the amplitudes and shapes of response spectra. This has clearly been shown by the case histories of ground response in Mexico City, the San Francisco Bay area, and many other locations. Furthermore, local site conditions influence the frequency content of surface motions and therefore also the response spectra they produce.[34]

2.1.2.2 Design Parameters

Designing new structures for earthquake resistance and evaluating the safety of existing structures, involves prediction of their response to earthquake induced shaking. A design level of shaking is defined based on the acceptable performance of a structure, and is described by a design ground motion. The design ground motion is found by using design parameters that have been developed from a design earthquake or by the means of seismic hazard analysis. The design ground motions are most commonly specified by parameters such as peak horizontal acceleration, peak horizontal velocity, predominant period, and duration.

The seismic loading for the dynamic analysis of structures is often represented by the use of response spectra. As a result, design spectra are often used to express the design ground motions. The design spectra and the response spectra of earthquakes are not the same. Response spectra of selected time histories contain detailed shapes that reflect the specific frequency content and phasing. As a contrast, design spectra are generally smooth, and are determined by averaging the response spectra of several motions. Using the smooth design spectra underlines the uncertainty of the soil and structural materials by avoiding the sharp fluctuations in spectral accelerations with small changes in period.[35]

2.1.2.3 Development of Design Parameters

The characteristics of the design ground motion at a particular site are influenced by several factors. The location of the site relative to potential seismic sources, the seismicity of those sources, the nature of rupture of the source, local site effects, and the importance of the structure for which the ground motion is to be used, all play a part in the determination of the characteristics. Design ground motions are usually

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[34] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 8.2

[35] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 8.3

developed either from site-specific analysis or from the provisions of building codes and standards.

The detailed effects of the particular subsurface conditions at the site of interest can be determined to reflect the site-specific design ground motions. The typical process for developing site-specific ground motion involves the use of seismic hazard analyses and ground response analyses.

As an alternative, design ground motions can be developed on the basis of building code provisions. Consideration of earthquake and other actions in the design of new structures is controlled by building codes, which are to be adopted as law by various governments. The building codes are developed by consensus of a broad group of experienced professionals and researchers. Even though current codes consider local site effects, they usually do so by lumping groups of similar soil profiles together. Hence, the provisions apply to broad ranges of soil conditions into which any local conditions of a particular site are expected to fall. Because of this, design ground motions developed from code provisions are usually more conservative than those developed from site-specific analysis.[36]

2.2 Dynamics of structures

2.2.1 Earthquake Response of a Linear System

Analyzing the response of structures to ground shaking caused by earthquakes, is one of the most important applications of the theory of structural dynamics. A study of earthquake response of linear single-degree-of-freedom systems to earthquake motions is required in developing a basis for understanding seismic loads.[37]

2.2.1.1 Response Spectrum Concept

Ground motion and their effects on structures are characterized by the concept of the earthquake response spectrum. The response spectrum provides a convenient way to summarize the peak response of all possible systems to a particular component of ground motion. It also provides a practical approach of applying

[36] Kramer, S.L., *Geotechnical Earthquake Engineering*, Section 8.4

[37] Chopra, A.K., *Dynamics of Structures,* Section 6

structural dynamics to the design of structures and development of lateral force requirements in the building codes.[38]

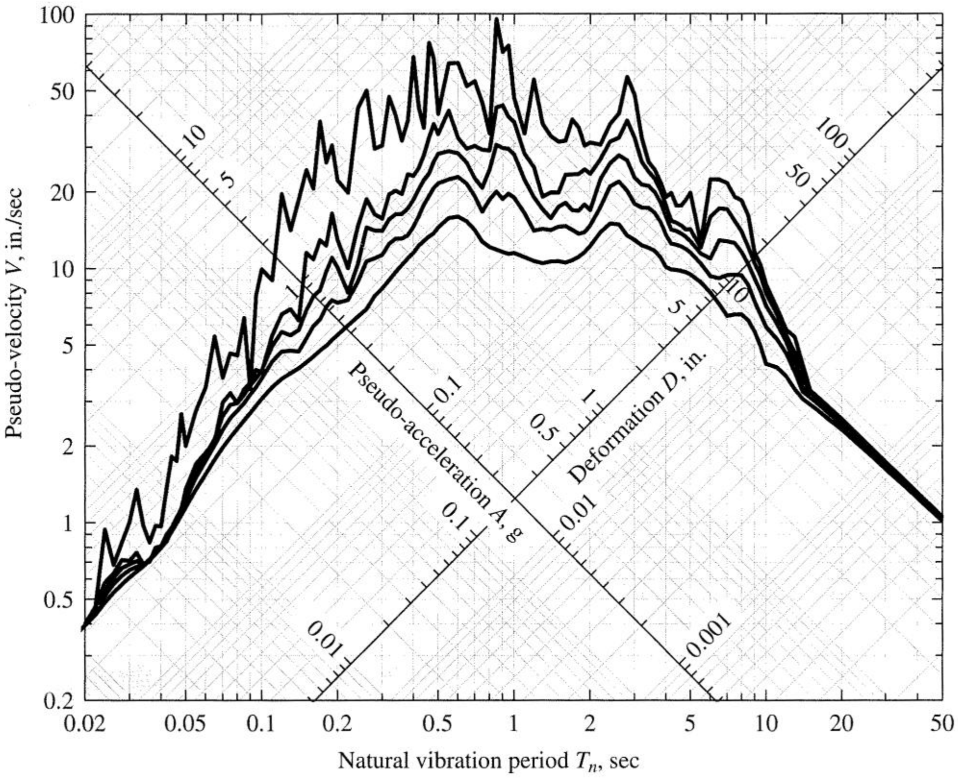

Fig. 2.4: Example of a *combined DVA response spectrum. Damping values ξ = 0, 2, 5, 10, and 20%. (Chopra)*[xii]

2.2.1.2 Peak Structural Response

The peak value of the deformation or the peak value of an internal force in any linear system can easily be determined if the response spectrum for a given ground motion component is known. This is the case because the complex dynamic analyses have already been completed in generating the response spectrum. Corresponding to the natural vibration period T_n and damping ratio ξ of the system, the values of deformation, D, pseudo-velocity, V, or pseudo-acceleration, A, are read from the spectrum (see Fig. 2.4). All response quantities of interest can be expressed in terms of D, V, or A, and the mass or stiffness properties of the system.[39]

[38] Chopra, A.K., *Dynamics of Structures,* Section 6.5

[39] Chopra, A.K., *Dynamics of Structures,* Section 6.7

2.2.1.3 Elastic Response Spectra

In designing structures for earthquake resistance, the ultimate goal is to resist the earthquake response of structures. The acceleration experienced by a building depends on its dynamic characteristics. The natural period and damping ratio are assumed to have greater effect on the acceleration than other factors. For a given damping ratio a curve known as a response spectrum of spectral acceleration can be drawn for various building periods. There will be a region on the response spectrum where the acceleration is highest. This occurs when the natural period matches the period of the earthquake, and the building experiences resonance. Theoretically, infinite resonant response is possible, but highly unlikely since all real structures are damped. Also, a properly designed and constructed building rarely experience true resonance. Planned and unplanned yielding occurs before true resonant response is achieved, and this yielding damps out the resonance.

The shape of the response spectra is often quite jagged, and it is not practical to use such a historical record for design. The spectrum reflects the occurrence of an actual earthquake, and it will never be matched perfectly by another one. At least three response spectra would have to be applied and the average values could then be used for design. For this reason the response spectrum is used to make an idealized average design spectrum based on the performances of several earthquakes, which has curves that are much smoother.

The relationship below shows that the spectral displacement, velocity and acceleration can be derived from one another if the natural frequency of vibration is known. Since these parameters are all related, the three spectral quantities can be shown by a single curve on a graph with three different scales (see Fig. 2.4). Such graphs are known as log tripartite, and are widely used to represent response spectra.

$$|S_d| = \left|\frac{S_v}{\omega}\right| = \left|\frac{S_a}{\omega^2}\right|$$

S_d, S_v, and S_a is the spectral displacement, velocity and acceleration, respectively, and ω is the natural frequency of vibration in rad/s. This expression is exact for the case of an undamped, single degree-of-freedom system in simple harmonic motion, but is approximate otherwise. [40]

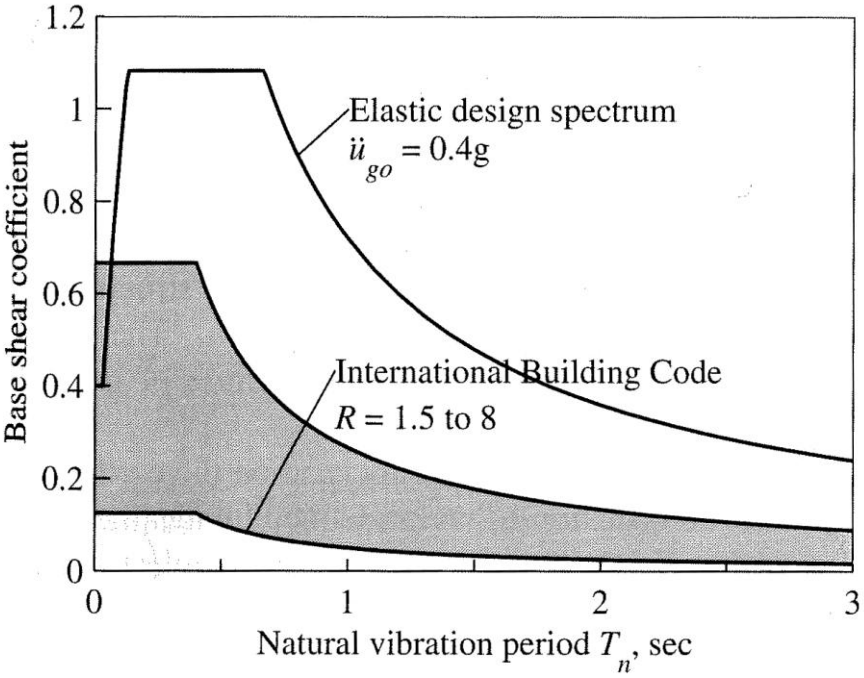

Fig. 2.5: *Comparison of base shear coefficients from elastic design spectrum and the IBC. 5% damping. (R is the response modification factor given in the 2006 IBC.) (Chopra)*[xiii]

2.2.2 Earthquake Response of Inelastic Systems

Most buildings are designed for base shear that is smaller than the elastic base shear associated with the strongest shaking that can occur at a particular site. This is clearly shown in Fig. 2.5, where the base shear coefficient A / g from the scaled design spectrum of Fig. 2.6 is compared with the base shear coefficient of the 2006 IBC[41]. This difference implies that buildings designed according to the code would be deformed beyond the limit of linearly elastic behavior when subjected to the presented ground motions. The response of structures deforming into their inelastic range during intense ground shaking is of vital importance in the design of structures. The objective of the engineers is to make sure the damage is controlled to an acceptable degree.[42]

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[40] Lindeburg, M.R., *Seismic Design of Building Structures*, Sections 44, 65, 66 and 68

[41] ICC, *2006 International Building Code*

[42] Chopra, A.K., *Dynamics of Structures,* Section 7

2.2.2.1 Force-Deformation Relations

Hundreds of laboratory tests have been conducted to determine the force-deformation behavior of structural components for earthquake conditions. During an earthquake, structures experience oscillatory motion with reversal of deformation. The experimental test results indicate that the cyclic force-deformation behavior of a structure depends on the structural material and on the structural system. The force deformation relation is often conveniently idealized by an elastoplastic relation, because this approximation allows the development of response spectra in a way that is similar to linear elastic systems. The peak deformation of an elastoplastic system due to earthquake ground motion is evaluated and the deformation is compared to the peak deformation caused by the same excitation in the corresponding linear system.[43]

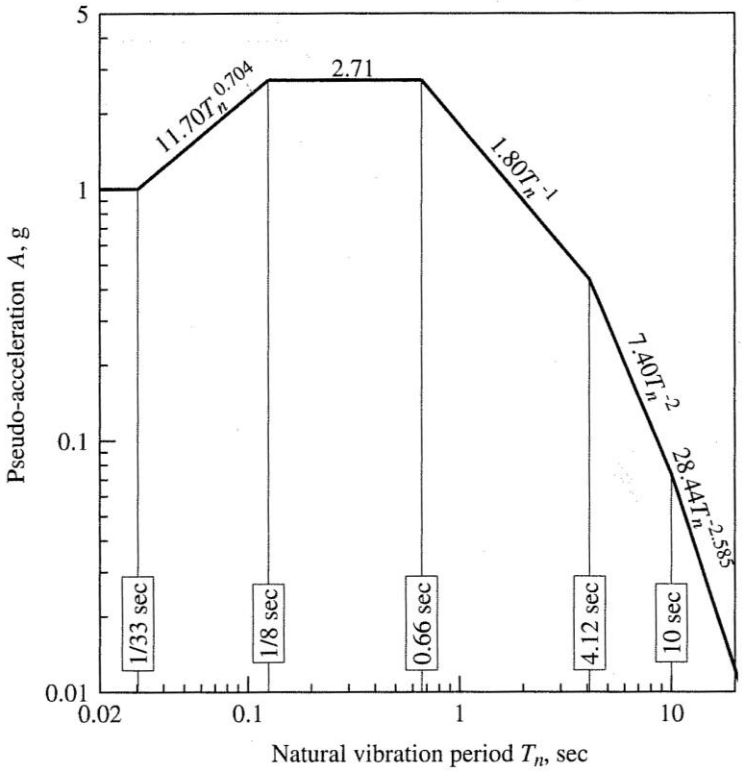

Fig. 2.6: *Elastic design spectrum. 5% damping. (Chopra)*[xiv]

2.2.2.2 Response Spectrum for Yield Deformation and Yield Strength

In order to limit the ductility demand imposed by the ground motion to a specified value, the necessary yield strength, f_y, of the system needs to be determined. An

[43] Chopra, A.K., *Dynamics of Structures,* Section 7.1

interpolative procedure is necessary to obtain the yield strength of an elastoplastic system for a specified ductility factor, μ. This factor is defined in Section 2.3.2.1. The procedure of constructing the response spectrum for elastoplastic systems corresponding to specified levels of ductility factor is a fairly straightforward sequence of steps.[44]

2.2.2.3 Inelastic Design Spectrum

By establishing the constant-ductility response spectrum for many possible ground motions, the design spectrum for elastoplastic systems for specified ductility factors can be constructed. Based on these data, the design spectrum associated with an exceedance probability can be established. Another approach is to develop a constant-ductility design spectrum from the elastic design spectrum, multiplying it by the normalized strength, f_y, or dividing it by the yield strength reduction factor, R_y. The yield strength reduction factor, R_y, is determined from the following expression:

$$R_y = \begin{cases} 1 & T_n < T_a \\ \sqrt{2\mu - 1} & T_b < T_n < T_{c'} \\ \mu & T_n > T_c \end{cases}$$

where μ is the ductility factor, and T_a, T_b, …, T_n are the periods separating the spectral regions.[45]

The inelastic design spectrum shows what the acceleration will be when some of the seismic energy is removed inelastically. When the response of a building to a major earthquake is being determined, it is important to consider the inelastic effects. The design yield strength and the design deformation for a system can be determined using allowable ductility, which is based on allowable deformation and on the ductility capacity. The inelastic design spectrum is also useful for direct displacement-based design of structures. The goal is to determine the initial stiffness and yield strength of the structure necessary to limit the deformation to some acceptable level. The

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[44] Chopra, A.K., *Dynamics of Structures,* Section 7.5

[45] Chopra, A.K., *Dynamics of Structures,* Section 7.11

inelastic response spectra are usually derived from the elastic response spectra. A simplified approach involves scaling down the elastic curves by the ductility factor, μ, as shown below.

$$S_{a,inelastic} = \frac{S_{a,elastic}}{\mu}$$

where $S_{a,\ inelastic}$ and $S_{a,\ elastic}$ are the spectral accelerations of the inelastic and elastic response spectra.[46]

2.3 Earthquake Response and Design of Multistory Buildings

2.3.1 Earthquake Response of Linearly Elastic Buildings

2.3.1.1 System Analysis

When analyzing multistory buildings it is common to idealize them as lumped mass systems. One way of doing such an analysis is by modeling the systems using single bay frames for all levels of the structure. When the dimensions and properties of the beams and columns are known, only a couple of other parameters are required. These parameters are the fundamental natural period, T_1, and the beam to column stiffness ratio, ρ, defined in the expression:

$$\rho = \frac{\sum_{beams} EI_b/L_b}{\sum_{columns} EI_c/L_c}$$

where EI_b and EI_c are the beam and column rigidities, and L_b and L_c are the lengths of the beams and columns.

The stiffness ratio is a measure of the relative beam to column stiffness and indicates how much the system may be expected to behave as a frame. Shown in Fig. 2.7, different values of ρ give various degrees of joint rotation. For ρ = 0, shown in (a), the joints rotate freely, and the frame behaves as a flexural beam. For ρ = ∞, shown in (c), there is no rotation of the joints, and the frame behaves as a stiff moment frame. Fig 2.7(b) shows an intermediate value of ρ, where beams and

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[46] Lindeburg, M.R., *Seismic Design of Building Structures*, Section 73

columns experience bending deformation with joint rotation. Typical earthquake resistant structures consist of frames with columns that are stiffer than the beams. The stiffness ratio is of great importance in determining the dynamic behavior of the frame.[47]

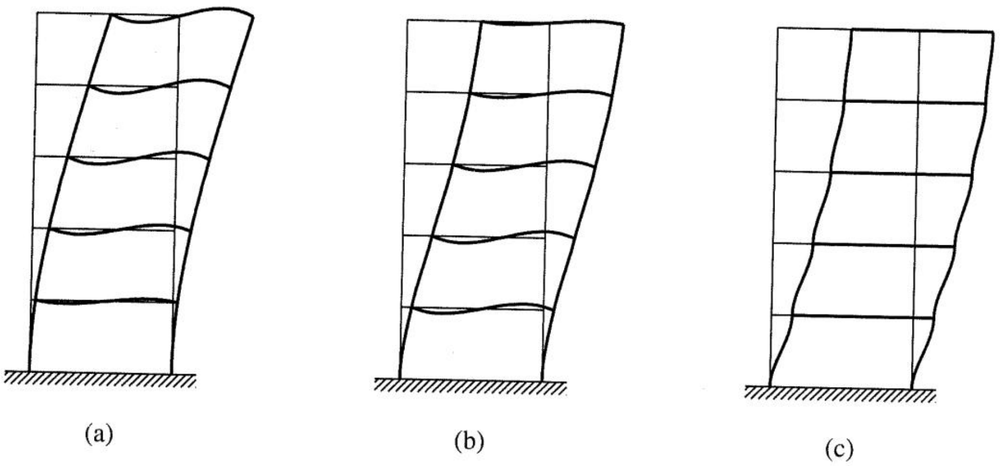

Fig. 2.7: *Deflected shapes: (a) ρ = 0, (b) ρ = 1/8, (c) ρ = ∞. (Chopra)[xv]*

2.3.1.2 Modes

The response contributions of all the natural modes of vibration must be included if the exact value of the structural response to earthquake excitation is desired. However, it is commonly recognized that sufficiently accurate results can be provided by the first few modes. In order to obtain the same desired accuracy, more modes should be included in the analysis of buildings with smaller ρ compared to the number of modes necessary for buildings with larger ρ. In other words, more modes should be included in the analysis of flexural frames than for stiff moment frames.[48]

2.3.2 Earthquake Analysis and Response of Inelastic Buildings

2.3.2.1 Ductility

When designing buildings it is necessary to accept some yielding during large earthquakes. Ductility in design is the capability of a structural member or building to yield without collapsing. During an earthquake, a ductile structure can dissipate large

[47] Chopra, A.K., *Dynamics of Structures,* Section 18.1

[48] Chopra, A.K., *Dynamics of Structures,* Section 18.7

amounts of seismic energy even after local yielding of connections, joints and other members has begun.

The actual ductility of a structural member is specified by its ductility factor, μ. There are a number of definitions of the ductility factor, all of which represent the ratio of some property at failure to that same property at yielding. The area under the stress-strain curve represents the strain energy absorbed. One definition of the ductility factor, μ, is shown in the expression below and is the ratio of the maximum energy that can be absorbed without failure, U_T, to the maximum energy that can be absorbed without yielding, U_R (see Fig 2.8).[49]

$$\mu = \frac{U_T}{U_R}$$

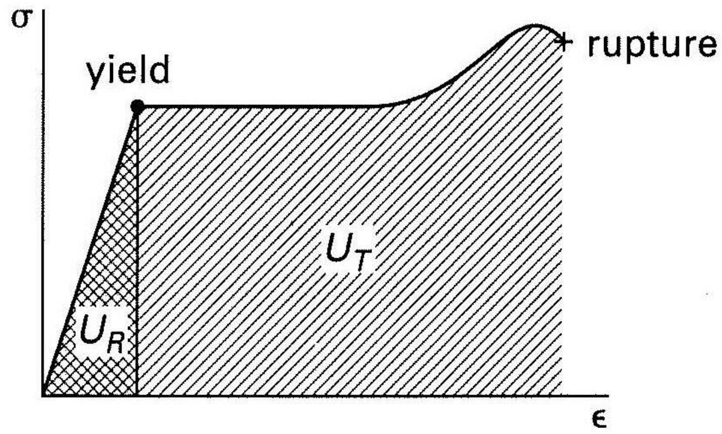

Fig 2.8: *Example of a stress strain curve for a ductile material. (Lindeburg)*[xvi]

2.3.2.2 P-Δ effects

One of the specific effects of ground motion on structures is known as the P-Δ effect (see Fig. 2.9a). Under normal conditions, column members in a building are concentrically loaded by vertical gravity loads. However, when a lateral seismic load acts upon the building, the vertical loads are eccentric with respect to the base. The overturning moment adds an eccentric bending stress to the columns, with a

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[49] Lindeburg, M.R., *Seismic Design of Building Structures*, Section 69

magnitude of P·Δ. P is a function of the building weight, and Δ is the story drift. Protection again these effects can be provided by diagonal bracings or shear walls.[50]

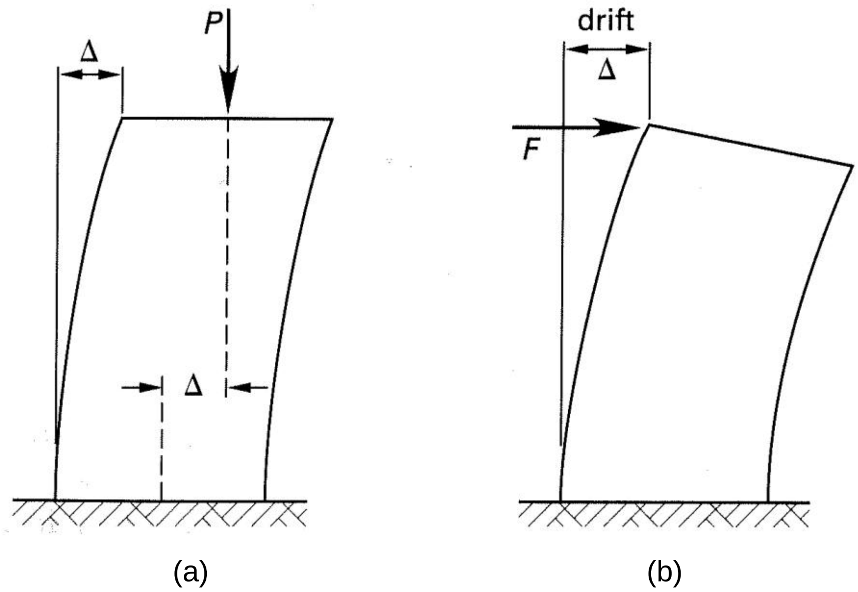

Fig. 2.9: *(a) P-Δ effect (b) Drift. (Lindeburg)*[xvii]

2.3.2.3 Story Drift

Another effect that must be accounted for is story drift, which is the deflection of one floor relative to the floor below (see Fig. 2.9b). Excessive drift can be accompanied by large secondary bending moments and inelastic behavior (see Figs 2.10 and 2.11). In a severe earthquake where yielding is experienced, a modern high-rise building can be expected to experience drift of approximately 1.5% of its total height at the top level.[51]

[50] Lindeburg, M.R., *Seismic Design of Building Structures*, Section 75

[51] Lindeburg, M.R., *Seismic Design of Building Structures*, Section 74

Fig. 2.10: *Olive View Hospital after the San Fernando earthquake in 1971. Large deformations occurred in the first story columns. (U of C)*[xviii]

Fig 2.11: *Fractured column of the Olive View Hospital building. (EERI)*[xix]

2.3.2.4 Influence of inelastic behavior

The extent to which a frame deforms into the inelastic range controls the distribution of story drift over the height of a multistory building. The story drift demands and their variation with height for inelastic systems are different from those of elastic systems and depend significantly on the ductility factor. The response contributions from higher vibration modes are known to be significant at the upper stories of the elastic frame, which cause an increase in story drifts. The largest drift occurs near the base of the structure as the ductility factor increases and the drifts in upper stories decrease.

2.3.3 Earthquake Dynamics of Base-Isolated Buildings

Base isolation is the concept of protecting a building from the damaging effects of an earthquake by using some type of support that isolates the building from the ground shaking. Early proposals for isolations systems go back over 100 years, but it is only in recent years that base isolation has become a practical strategy for earthquake resistant design.[52]

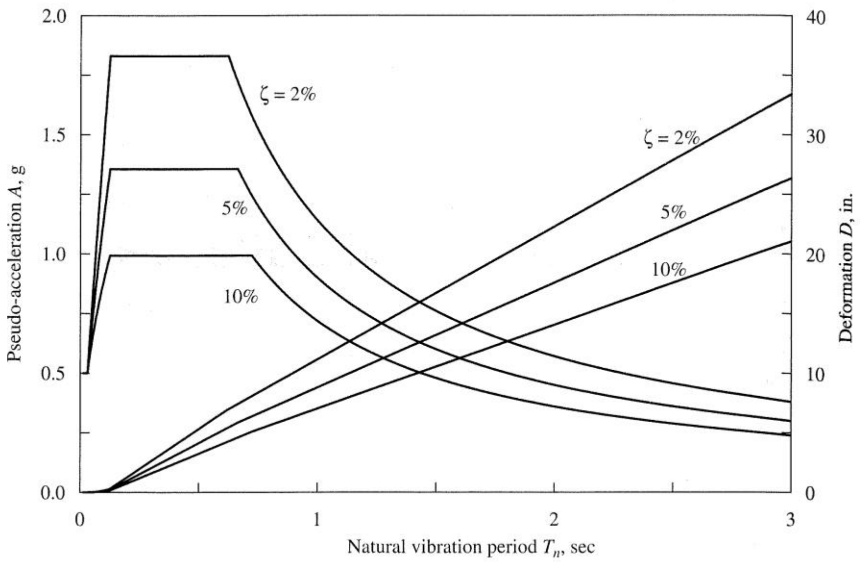

Fig. 2.12: *Example of an elastic design spectrum for different damping values. PGA value 0.5g. (Chopra)[xx]*

2.3.3.1 Isolation Systems

Base isolation systems follow two basic approaches with certain common features. In the first approach the isolation system consists of a layer of low lateral stiffness

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[52] Chopra, A.K., *Dynamics of Structures,* Section 20

between the structure and the foundation. With this isolation layer the structure has a natural period that is much longer than its fixed based natural period. As shown by the elastic design spectrum in Fig. 2.12, this increase in period can reduce the pseudo-acceleration and therefore also the earthquake-induced forces in the structure. The most common system of this type uses short, cylindrical bearings with alternating layers of steel plates and hard rubber. Placed between the base of the structure and the foundation, these laminated bearings are strong and stiff under vertical loads, yet very flexible under lateral forces (see Fig 2.13).

The second most common type of isolation system uses sliding elements between the foundation and the base of the structure. The shear force transmitted to the structure across the isolation interface is limited by keeping the coefficient of friction as low as practically possible. The friction must be sufficiently high, however, to sustain strong winds without sliding. [53]

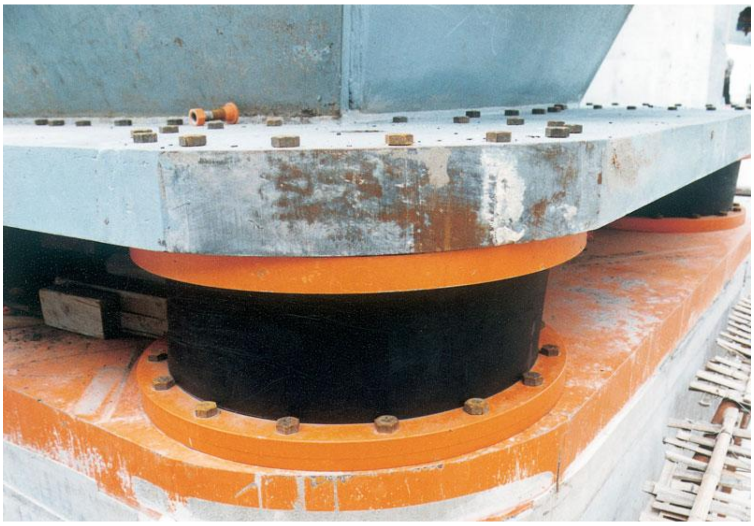

Fig 2.13: *Laminated rubber bearing.* [xxi]

2.3.3.2 Effectiveness of Base Isolation

The effectiveness of base isolation in reducing structural forces is clearly related to the increase in the natural period of the structure. The ratio between the isolated period and the fixed-base period, T_b/T_f, should therefore be as large as possible. To what extent the forces in a structure are reduced because of the period shift mainly

[53] Chopra, A.K., *Dynamics of Structures,* Section 20.1

depends on the natural period of the fixed-base structure and on the shape of the earthquake design spectrum.

The benefits obtained by base isolation are much greater for buildings with shorter fixed base vibration periods than for buildings with longer fixed base periods. For this reason, base isolation is rarely used for structures with natural periods in the velocity sensitive region of the spectrum.[54]

2.3.3.3 Applications of Base Isolation

Base isolation provides an alternative to the conventional, fixed base design of structures and may be cost effective for some new buildings in locations where very strong ground shaking is likely. Its application should be seriously considered for buildings that must remain functional after major earthquake such as hospitals, emergency response centers, and other essential facilities.

Both types of isolation systems have also been used to retrofit older existing buildings that where designed with limited understanding or consideration of earthquake hazards. Conventional design for seismic strengthening requires adding new structural members, such as shear walls, moment frames, and bracings. By reducing the earthquake forces transferred to the building, base isolation systems minimizes the need for such strengthening measures.

[54] Chopra, A.K., *Dynamics of Structures,* Section 20.3

Fig. 2.14: *The International Terminal at the San Francisco Airport.*[xxii]

Being in a region of high seismicity, the San Francisco Airport chose to use base isolation when they built their new International Terminal, which was completed in 2000 (see Fig. 2.14). It was designed to remain operational after an earthquake of magnitude 8. To achieve this performance goal, the superstructure was isolated using 267 isolators, one at each column base (see Fig. 2.15). Each isolator is a friction pendulum sliding bearing (FPB). The earthquake forces on the superstructure were reduced to 30% of the demands for the fixed base structure when using the base isolation. With this reduction in force, the superstructure was designed to remain essentially elastic and undamaged under the selected design earthquake with peak ground acceleration of 0.6g.[55]

[55] Chopra, A.K., *Dynamics of Structures,* Section 20.5

Fig. 2.15: *San Francisco Airport: Friction Pendulum Bearing at base of column.*[xxiii]

2.4 Structural Dynamics in Building Codes

The use of a static equivalent lateral force (ELF) procedure for many regular structures with relatively short periods is permitted by most seismic building codes. For other structures, dynamic analysis procedures are required. When using the ELF procedure, structures are designed to resist specified static lateral forces related to the properties of the structure and the seismicity of the region. Formulas are specified for the base shear and the distribution of lateral forces over the height of the building. This is done based on an estimate of the fundamental natural vibration period of the structure. The design forces, including shear and overturning moments for the various stories, are provided by static analysis of the building.

2.4.1 Elastic Seismic Coefficient, C_e

Related to the pseudo-acceleration spectrum for linearly elastic systems, the elastic seismic coefficient, C_e, is used in calculating the design base shear in the various building codes. The coefficient is larger than the pseudo-acceleration normalized with respect to gravitational acceleration, A/g. This is to account for the more complex dynamics of multistory buildings responding in several natural modes of vibration and to recognize uncertainties in a calculated value of the fundamental vibration period.

2.4.2 Design Force Reduction

The design base shear is specified to be smaller than the elastic base shear in most codes. The design forces that relate to the results obtained from dynamic response analysis are reduced by a certain reduction factor specified in the codes. In the 2006 IBC this factor is known as the response modification factor, R, and in Eurocode 8 it is known as the behavior factor, q. The seismic reduction factors vary with vibration period in a way that is consistent with structural dynamics theory.

2.4.3 Lateral Force Distribution

The expressions for base shear, V_{bn}, and equivalent static lateral force, f_{jn}, at floor level j for mode n of a multistory building is given by structural dynamics. The two expressions can be combined to produce the equation:

$$f_{jn} = V_{bn} \frac{w_j \varphi_{jn}}{\sum_{i=1}^{N} w_i \varphi_i}$$

The corresponding equation below can be found in the 2006 IBC[56], giving the distribution of lateral forces, F_j, based on the assumption that the natural mode of vibration, φ_{jn}, is proportional to the height of the building at the j-th floor, h_j This assumption states that the mode shape is linear, which is reasonable for the fundamental modes of many buildings.

$$F_j = V_b \frac{w_j h_j}{\sum_{i=1}^{N} w_i h_i}$$

where w_i is the weight at the i-th floor at height h_i above the base. Both of these equations also appear in the Eurocode[57], which implies that the distribution of lateral forces is based fully on the fundamental mode of vibration without considering the increasing higher-mode contributions to response.[58]

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[56] ICC, *2006 International Building Code*

[57] CEN, *Eurocode 8*

[58] Chopra, A.K., *Dynamics of Structures,* Section 21.5

2.4.4 Overturning Moment and Torsional Shear Stress

The overturning moment is the sum of all moments taken about the base due to the distributed lateral forces. If this moment is large enough, it can reverse the compression that normally exists in the outermost columns of a building. Because the structural members can be placed in tension, the overturning moment is a larger problem for concrete frame and shear wall construction than for steel frame construction. On the opposite side of the building, the overturning moment can cause an increase in the compression loads, which must be accounted for in the design of the columns.

Some of the building codes allow for a reduction of the overturning moments compared to the values found by the use of statics, because the response contributions of the higher modes are larger for story shears than for overturning moments. The 2006 IBC and Eurocode 8 do not permit any reduction in overturning moments, a specification that is not supported by the results of elastic dynamic analysis. Therefore, the overturning moments calculated from the code forces would exceed the values predicted by dynamic analysis.

A building's center of mass is a point through which the base shear can be assumed to act. The base shear is resisted by vertical members at the ground level. Each member may have a different rigidity and therefore provides a different lateral resisting force in the opposite direction of the base shear. The building's center of rigidity is a point through which the resultant of all the resisting forces acts. If the building's center of mass is different from its center of rigidity, it will be acted upon by a torsional moment. Even when the centers of mass and rigidity do coincide, an accidental eccentricity of 5% must be accounted for according to the codes. The torsional moment, $M_{torsional}$, is calculated as a product of the base shear, V, and the eccentricity, e. The eccentricity is the distance between the centers of mass and rigidity. Unlike the base shear, which is resisted only by walls parallel to the seismic force, the torsional shear is resisted by all walls and columns.[59]

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[59] Lindeburg, M.R., *Seismic Design of Building Structures*, Sections 76 and 78

3 Comparative Study of US and European Seismic Codes

3.1 Introduction

In this chapter, the structural provisions used in the US and Europe are reviewed and compared. In the United States, the International Building Code (IBC) is the authority to be followed. Due to the comprehensiveness of this code, most of the seismic provisions are given in a publication by the American Society of Civil Engineers (ASCE 7-05). The European code is reviewed with emphasis on the provisions for Norway given in the National Annex. The Norwegian Standard (NS 3491-12) will not be discussed here, because it is no longer the most current code used in design and it is also largely based on the Eurocode.

3.2 International Building Code (2006) and ASCE 7 (2005)

3.2.1 Scope

According to the 2006 IBC, all structures, including permanent nonstructural elements, must be designed and constructed to resist the effects of earthquake ground motions. This is to be done in accordance with ASCE 7-05[60], which specifies all minimum design loads. The seismic design criteria of a building are to be determined according to section 1613 of the 2006 IBC or section 11 of ASCE 7-05. The ASCE 7-05 is the primary reference document for the 2006 IBC, and any seismic design in the United States therefore mainly relies on this code to supply the appropriate provisions.[61]

3.2.2 Seismic Design Criteria

3.2.2.1 Seismic Ground Motion Values

3.2.2.1.1 Mapped Acceleration Parameters

S_s and S_1 are mapped parameters that indicate the 5% damped spectral acceleration of the Maximum Considered Earthquake (MCE) in short and long periods (0.2s and 1.0s), respectively. The parameters are determined from 0.2 and

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[60] American Society of Civil Engineers, *ASCE 7-05 Minimum Design Loads for Buildings and Structures*

[61] International Code Council, *2006 International Building Code*, Section 1613.1

1.0-second spectral response accelerations shown on maps provided from the US Geological Survey (see Fig 2.1), which are included in the code.[62]

3.2.2.1.2 Site Classification

Any site is to be classified on a scale from A to F in accordance with the site soil properties ranging from hard rock to soft soil. The properties for each category are defined in a table giving limits for the soil shear wave velocity, standard penetration resistance and soil undrained shear strength (see section 3.2.4).[63]

Table 3.1: *Site coefficient, F_a. (ASCE 7-05)*[xxiv]

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9

NOTE: Use straight-line interpolation for intermediate values of S_S.

Table 3.2: *Site coefficient, F_v. (ASCE 7-05)*[xxv]

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4

NOTE: Use straight-line interpolation for intermediate values of S_1.

[62] ICC, *2006 IBC*, Section 1613.5.1 / *ASCE 7-05*, Section 11.4.1

[63] ICC, *2006 IBC*, Section 1613.5.2 / *ASCE 7-05*, Section 11.4.2

3.2.2.1.3 Site Coefficients

The site coefficients, F_a and F_v, are based on short and long period ground motions and are defined in tables in the code (see Tables 3.1 and 3.2). They are obtained by interpolating values of the mapped spectral response acceleration and the previously determined site class.[64]

The maximum considered earthquake spectral response accelerations for short periods, S_{MS}, and at 1 second period, S_{M1}, adjusted for site class effect is determined from the following equations:

$$S_{MS} = F_a S_S \qquad\qquad S_{M1} = F_v S_1$$

3.2.2.1.4 Design Spectral Acceleration Parameters

The design spectral response acceleration is defined to be two-thirds of the MCE spectral response acceleration[65]. Therefore the design values for the previously determined parameters can be found using the following equations:

$$S_{DS} = 2/3 \cdot S_{MS} \qquad\qquad S_{D1} = 2/3 \cdot S_{M1}$$

3.2.2.2 Importance Factor and Occupancy Category

An importance factor, I, is to be assigned to each structure with values ranging from 1.0 to 1.5 (see Table 3.4) based on the occupancy category found in a table (see Table 3.3). This table describes the different types of buildings in each category and is sorted by the severity of consequences if a building is to collapse in a seismic event. Category I buildings include agricultural facilities and certain temporary or minor storage structures. Category III buildings include schools and assembly facilities that contain a large number of people, and other structures considered important, such as power plants and water treatment facilities. Category IV buildings are considered essential, and include hospitals and emergency facilities. Buildings that contain large amounts of hazardous or toxic materials are also included in this

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[64] ICC, *2006 IBC*, Section 1613.5.3 / *ASCE 7-05*, Section 11.4.3

[65] ICC, *2006 IBC*, Section 1613.5.4 / *ASCE 7-05*, Section 11.4.4

category. Category II includes all structures that are not included in any other category.

Table 3.3: *Occupancy category of buildings and other structures. (ASCE 7-05)*[xxvi]

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with daycare facilities with a capacity greater than 150 • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities • Jails and detention facilities Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to: • Power generating stations[a] • Water treatment facilities • Sewage treatment facilities • Telecommunication centers Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released. Buildings and other structures containing toxic or explosive substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the toxic or explosive substances does not pose a threat to the public.	III
Buildings and other structures designated as essential facilities, including, but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities • Fire, rescue, ambulance, and police stations and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power generating stations and other public utility facilities required in an emergency • Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency • Aviation control towers, air traffic control centers, and emergency aircraft hangars • Water storage facilities and pump structures required to maintain water pressure for fire suppression • Buildings and other structures having critical national defense functions Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. Buildings and other structures containing highly toxic substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the highly toxic substances does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	IV

[a] Cogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II.

Table 3.4: *Importance factors. (ASCE 7-05)*[xxvii]

Occupancy Category	*I*
I or II	1.0
III	1.25
IV	1.5

3.2.2.3 Seismic Design Category

The seismic design category (SDC) is a classification assigned to a structure based on its occupancy category and the severity of the design earthquake ground motion at a particular site. Structures in occupancy category I, II or III located where the S_1 parameter is greater than or equal to 0.75 is assigned to SDC E. Structures in occupancy category IV located where S_1 is greater or equal to 0.75 is assigned to SDC F. All other structures are assigned to a seismic category based on their occupancy category and the design spectral response acceleration coefficients, S_{DS} and S_{D1} (see Table 3.5 and 3.6) [66]

Table 3.5: *Seismic Design Category based on short period response acceleration parameter. (ASCE 7-05)* [xxviii]

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \le S_{DS} < 0.33$	B	B	C
$0.33 \le S_{DS} < 0.50$	C	C	D
$0.50 \le S_{DS}$	D	D	D

Table 3.6: *Seismic Design Category based on 1 s period response acceleration parameter. (ASCE 7-05)* [xxix]

Value of S_{D1}	OCCUPANCY CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067$	A	A	A
$0.067 \le S_{D1} < 0.133$	B	B	C
$0.133 \le S_{D1} < 0.20$	C	C	D
$0.20 \le S_{D1}$	D	D	D

3.2.3 Seismic Design Requirements for Buildings

3.2.3.1 Structural Design Basis

The basic requirement of the code states that a building must include complete lateral and vertical force resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions. The design seismic forces, and their distribution over the height of the building, are

[66] ICC, *2006 IBC*, Section 1613.5.6 / *ASCE 7-05*, Section 11.6

established in accordance with the relevant procedures. The corresponding internal forces and deformations in the members of the structure can then be determined.

The individual members, including those that are not part of the seismic force resisting system, must have adequate strength to resist the shear, axial forces, and moments determined in accordance with the standard. A continuous load path with adequate strength and stiffness is necessary to transfer all forces from the point of application to the final point of resistance. All parts of the structure between separation joints need to be interconnected to form a continuous path to the seismic force resisting system. The connections must also be capable of transmitting the seismic force induced by the parts that are being connected.[67]

3.2.3.2 Structural System Selection

The basic lateral and vertical seismic force resisting system is to conform to one of the types indicated in the code. These types include bearing wall, building frame, moment resisting frame systems and various dual systems, and are divided into subcategories that specifically describe the structural systems to be used. The appropriate response modification coefficient, R, system overstrength factor, Ω_0, and the deflection amplification factor, C_d, indicated for each system is to be used in determining the base shear, element design forces, and design story drift (see Table 3.7).[68]

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[67] ASCE 7-05, Section 12.1

[68] ASCE 7-05, Section 12.2.1

Table 3.7: *Design coefficients and factors for seismic force resisting systems. (Section of table from ASCE 7-05)*[xxx]

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D^d	E^d	F^e
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls	14.2 and 14.2.3.6	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls	14.2 and 14.2.3.4	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls	14.2 and 14.2.3.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls	14.2 and 14.2.3.1	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls	14.2 and 14.2.3.5	4	2½	4	NL	NL	40^k	40^k	40^k
6. Ordinary precast shear walls	14.2 and 14.2.3.3	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4 and 14.4.3	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4 and 14.4.3	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	6½	3	4	NL	NL	65	65	65
14. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2	2½	2	NL	NL	35	NP	NP
15. Light-framed wall systems using flat strap bracing	14.1, 14.1.4.2, and 14.5	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	14.1	8	2	4	NL	NL	160	160	100
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	14.1	7	2	4	NL	NL	160	160	100
3. Special steel concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
4. Ordinary steel concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35^j	35^j	NP^j

3.2.3.3 Seismic Load Effects and Combinations

All members of a structure, including those that are not part of the seismic force resisting system, are to be designed using the seismic load effects described in this section. Seismic load effects are the axial, shear, and flexural member forces resulting from applied horizontal and vertical seismic forces.

A redundancy factor, ρ, is assigned to the seismic force restraining system in each of two orthogonal directions for all structures, and is set to 1.3 for structures in Seismic Design Categories D, E or F. For all other structures it is set to 1.0.

The seismic load effect, E, is determined from the expression $E = E_h + E_v$, where E_h and E_v are the horizontal and vertical components of the seismic force. The horizontal load effect can be found from the equation $E_h = \rho \cdot Q_E$, where ρ is the redundancy factor defined above, and Q_E is the effects of horizontal seismic forces from the base shear, V, or the seismic design force, F_p. The vertical load effect is determined by the expression $E_v = 0.2 \cdot S_{DS} \cdot D$, where S_{DS} is the design spectral response acceleration parameter defined earlier, and D is the effect of the dead load.[69]

3.2.3.4 Modeling Criteria

The effective seismic weight, W, of a structure includes the total dead load, 25 percent of the floor live load for areas used for storage, and the total weight of permanent equipment and partitions.

For the purpose of determining forces and displacements resulting from applied loads and any imposed displacements or P-delta effects, it is necessary to construct a mathematical model of the structure. The model must include the stiffness and strength of the elements that are important to the distribution of forces and deformations in the structure. It must also represent the distribution of mass and stiffness throughout the structure. When determining seismic loads, it is permitted to consider the structure to be fixed at the base.[70]

3.2.3.5 Equivalent Lateral Force Procedure

3.2.3.5.1 Seismic Base Shear

The seismic base shear, V, in a given direction is determined using the expression:

$$V = C_S W$$

where C_S is the seismic response coefficient and W is the effective seismic weight.

[69] ASCE 7-05, Section 12.4

[70] ASCE 7-05, Section 12.7

3.2.3.5.2 Seismic Response Coefficient

The seismic response coefficient, C_S, is determined from the equation:

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

where S_{DS} is the design response acceleration parameter defined earlier, R is the response modification factor, and I is the occupancy importance factor.

The coefficient does not need to be greater than:

$$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$$

where T is the fundamental period of the structure.

The coefficient must not be less than:

$$C_S = 0.044 \cdot S_{DS} \cdot I$$

3.2.3.5.3 Period Determination

The fundamental period of the structure, T, in the direction under consideration is established using the structural properties and deformational characteristics of the resisting elements in a proper analysis. As an alternative to performing an analysis to determine the fundamental period, it is permitted to use the approximate building period, T_a, calculated by using the equation:

$$T_a = C_t h_n^x$$

where h_n is the height in feet above the base to the highest level of the structure . The coefficients C_t and x are determined from a table found in the code and depend on the structure type (see Table 3.8).

Table 3.8: *Values of approximate period parameters C_t and x. (ASCE 7-05)*[xxxi]

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 $(0.0724)^a$	0.8
Concrete moment-resisting frames	0.016 $(0.0466)^a$	0.9
Eccentrically braced steel frames	0.03 $(0.0731)^a$	0.75
All other structural systems	0.02 $(0.0488)^a$	0.75

^{a}Metric equivalents are shown in parentheses.

3.2.3.5.4 Vertical Distribution of Seismic Forces

The lateral seismic force induced at any level can be determined using the equation:

$$F_x = C_{vx}V$$

where C_{vx} is the vertical distribution factor, and V is the total design lateral force or shear at the base of the structure. The vertical distribution factor is given by the equation:

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{N} w_i h_i^k}$$

where w is the portion of the total effective seismic weight of the structure, W, located at the assigned level, and h is the height from the base to the assigned level. The subscript i indicates the total number of levels in the structure, and the subscript x indicates the level under consideration. The exponent k relates to the period of the structure and is assigned a value of 1 for T ≤ 0.5 s, a value of 2 for T ≥ 2.5 s. For periods between 0.5 s and 2.5 s, the value k is determined by linear interpolation between 1 and 2.

3.2.3.5.5 Horizontal Distribution

The seismic design story shear in any story, V_x, can be determined from the equation $V_x = \Sigma F_i$, where F_i is the portion of the seismic base shear, V, induced at level i. Based on the relative lateral stiffness of the vertical resisting elements and the diaphragm, the story shear is distributed to the various vertical elements of the seismic force-resisting system.

For diaphragms that are not flexible, the distribution of lateral forces at each level must consider the effect of the inherent torsional moment, M_t, resulting from eccentricity between the locations of the centers of mass and rigidity. For flexible diaphragms, the distribution of forces to the vertical elements must account for the position and distribution of the supported masses.

3.2.3.5.6 Story Drift Determination

The design story drift, Δ, is computed as the difference of the deflections at the top and bottom of the story under consideration. The deflections of level x at the center of the mass, δ_x, is determined from the equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I}$$

where C_d is the deflection amplification factor, δ_{xe} is the deflections determined by an elastic analysis, and I is the importance factor.

3.2.3.5.7 P-Delta Effects

P-delta effects on story shears and moments, and the story drifts induced by these effects, are not required to be considered where the stability coefficient, θ, as determined according to the following equation, is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d}$$

where P_x is the total vertical design load at and above level x, Δ is the design story drift, V_x is the seismic shear force acting between level x and x-1, h_{sx} is the story height below level x, and C_d is the deflection amplification factor.

Where the stability coefficient is greater than 0.10, the incremental factor related to P-delta effects on displacements and member forces must be determined by rational analysis.[71]

3.2.3.6 Modal Response Spectrum Analysis

An analysis must be conducted to determine the natural modes of vibration of a structure. The analysis must include a sufficient number of modes to obtain a combined modal mass contribution of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response used in the model.

The value for each force related design parameter of interest is computed using the properties of each mode and the response spectra divided by the quantity R/I. The value for displacement and drift quantities must be multiplied by the quantity C_d/I.[72]

3.2.4 Site Classification Procedure for Seismic Design

3.2.4.1 Definitions

The site soil must be classified in accordance with a table found in the code, which describes the site classes A through F, based on the soil properties of the upper 100 ft of the site (see Table 3.9). Where the soil properties are not known in sufficient detail to determine the site class, site class D is to be used unless the authority having jurisdiction or geotechnical data determines site class E or F soils are present at the site. Site classes A and B are not to be assigned to a site if there is more than 10 ft of soil between the rock surface and the bottom of the spread footing or mat foundation. The properties of the soil for each site class is defined by the average shear wave velocity, v_s, average field standard penetration resistance, N, and average undrained shear strength, s_u.

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[71] ASCE 7-05, Section 12.8

[72] ASCE 7-05, Section 12.9

Table 3.9: *Site classification. (ASCE 7-05)*[xxxii]

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: - Plasticity index PI > 20, - Moisture content $w \geq 40\%$, and - Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s 1 lb/ft^2 = 0.0479 kN/m^2

3.3 Eurocode 8 (2004)

3.3.1 Design Criteria and Requirements

3.3.1.1 Fundamental Requirements

There are two basic requirements that are to be met in the design and construction of structures in seismic regions. The no-collapse requirement states that the structure is to withstand the seismic design loads defined in the standard without local or global collapse. The seismic design loads are expressed in terms of the reference seismic action associated with a reference probability of exceedance, P_{NCR}, in 50 years, and the importance factor γ_I. In Norway the reference probability of exceedance, P_{NCR}, is set to the recommended value of 10%.

The other basic requirement is that of damage limitation. It states that a structure is to be designed and constructed to withstand a seismic load having larger probability of occurrence than the seismic design load, without causing damage of unreasonably high cost in comparison with the cost of the structure. The Norwegian National Annex (NA) states that this requirement is not applicable in Norway.[73]

3.3.1.2 Compliance Criteria

The compliance criteria present two limit states to be checked in order to satisfy the fundamental requirements above. The ultimate limit states are those associated with

[73] CEN, *Eurocode 8,* Section 2.1

collapse or other forms of structural failure that may endanger life safety. Damage limitation states are those associated with damage causing buildings to no longer meet the specified service requirements. Again, the latter limit state is not applicable in Norway.[74]

3.3.2 Ground Conditions and Seismic Loads

3.3.2.1 Ground Conditions

The ground conditions at a particular site must be identified by carrying out appropriate investigations of the soil. The construction site and the nature of the supporting ground should normally be free from risk of ground rupture, slope instability and permanent settlement caused by liquefaction in the event of an earthquake.

The influence of local ground conditions on the seismic loads is accounted for using ground types A through E described by soil properties ranging from rock to soft soil. The ground types are classified in a table, defining the appropriate soil properties in terms of parameters such as average shear wave velocity, standard penetration test blow count, and the undrained shear strength of the soil (see Table 3.10).[75]

[74] CEN, *Eurocode 8,* Section 2.2

[75] CEN, *Eurocode 8,* Section 3.1

Table 3.10: *Ground types. (Eurocode 8)*[xxxiii]

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	–	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

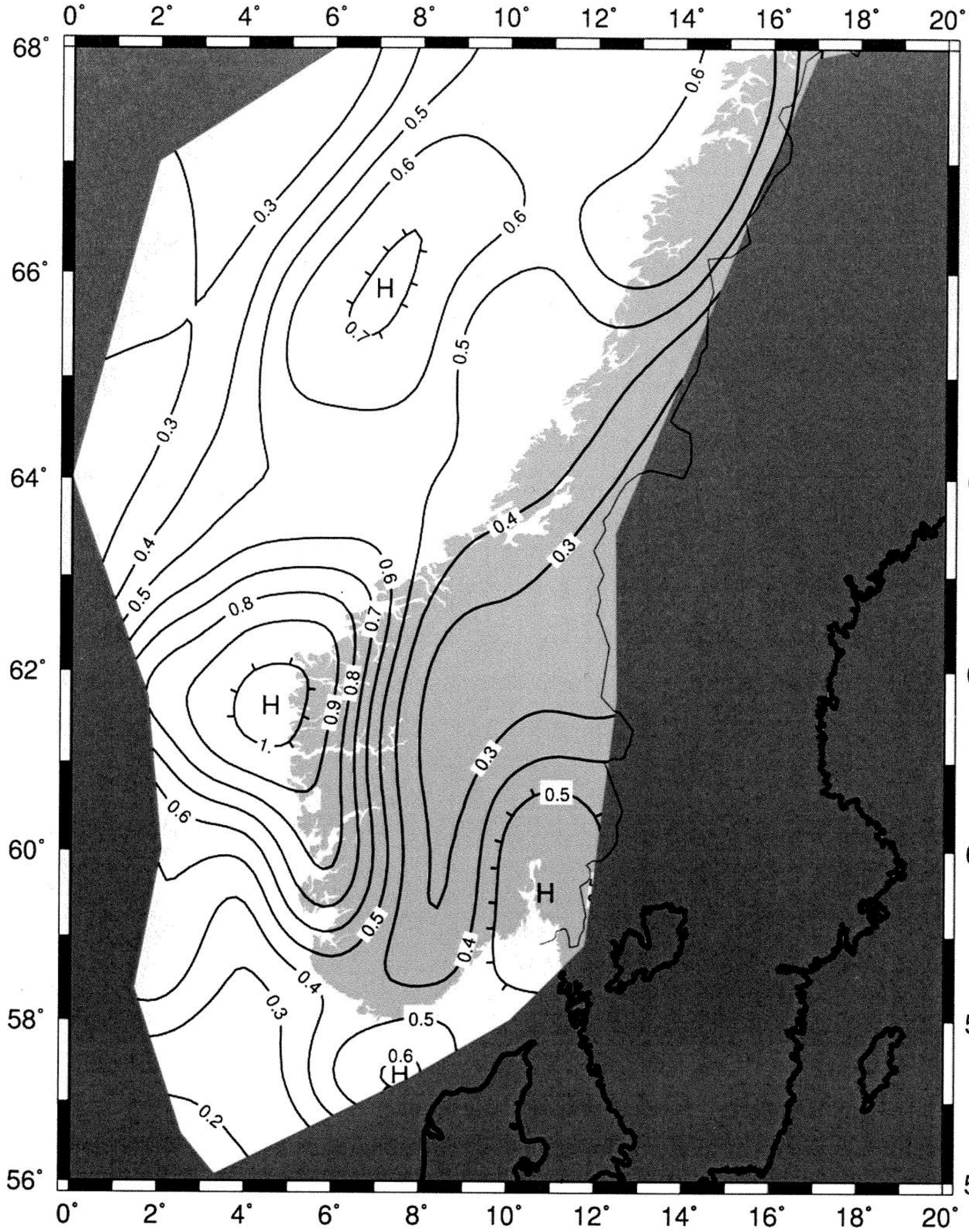

Fig 3.1: *Seismic zones in Southern Norway, a_{g40Hz} in m/s^2. (Eurocode 8)*[xxxiv]

3.3.2.2 Seismic Loads

3.3.2.2.1 Seismic Zones

National territories are subdivided by the authorities into seismic zones depending on the local hazard. By definition, the hazard within each zone is assumed to be constant. The hazard is for the most part described by a single parameter, the reference peak ground acceleration on ground type A, a_{gR}. The reference peak ground acceleration corresponds to the reference return period, T_{NCR}, of the seismic loads for the no-collapse requirement. The importance factor of 1.0 is assigned to this reference return period. The local peak ground acceleration normalized to 1.0g at the frequency of f = 40 Hz, a_{g40Hz}, is given on maps of southern and northern Norway in the National Annex (see Fig 3.1). The reference peak ground acceleration, a_{gR}, is set equal to $0.8 \cdot a_{g40Hz}$.

3.3.2.2.2 Basic Representation of the Seismic Loads

The earthquake motion at a given point on the surface is represented by an elastic response spectrum. The horizontal seismic action is described by two orthogonal components assumed to be independent.

For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by expressions for various intervals of vibration period, T. The parameters used in the expressions vary by ground type and are found in a specified table. Similarly, expressions are outlined in the code to give the elastic response spectrum, $S_{ve}(T)$, which represents the vertical component of the seismic load.

The design of structural systems for resistance to seismic forces generally permits the capacity in the non-linear range to be smaller than that corresponding to the linear elastic response. By performing an elastic analysis based on a design spectrum, the capacity of the structure to dissipate energy is taken into account. This is done to avoid explicit inelastic structural analysis in design, and is accomplished by introducing the behavior factor q (see Sections 3.3.4.2 and 3.3.5.1.2).[76]

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[76] CEN, *Eurocode 8,* Section 3.2

3.3.3 Design of Buildings

3.3.3.1 Characteristics of Earthquake Resistant Buildings

3.3.3.1.1 Structural simplicity

Modeling, analysis, design, detailing and construction of simple structures have a much lower level of uncertainty than complex projects. As a result, the prediction of the seismic behavior of a simple building is much more reliable. Structural simplicity, which is characterized by the existence of clear and direct load paths, is therefore an important goal in design of buildings.

3.3.3.1.2 Uniformity

A uniform plan is recognized from the even distribution of structural elements. This results in short and direct load paths to the elements resisting the inertial forces created by the induced motions. Uniformity can be achieved by subdividing a building into dynamically independent units by the use of seismic joints.

3.3.3.1.3 Torsional resistance

In addition to lateral resistance and stiffness, buildings should possess adequate torsional resistance and stiffness to limit the development of torsional motions. The torsional moment of inertia is the resisting force of any structural element, and is highly dependent on the distance between the resisting element and the center of mass. Since the center of mass usually is located near the center of a building, the main elements resisting should be located as near the building perimeter as possible for increased effect.

3.3.3.1.4 Criteria for structural regularity

Structures are generally categorized as being either regular or non-regular. This distinction has implications for the structural model, which can be either a simplified planar model or a spatial model. It also affects the method of analysis, which can be either a simplified response spectrum analysis or a modal one. The criteria for structural regularity apply in both plan and elevation, with specific requirements for each.

3.3.3.1.5 Importance classes and factors

Buildings are classified in 4 importance classes, depending on the consequences for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse. The definitions of the importance classes are given in a table in the code, and are based on the seriousness of the consequences of failure (see Table 3.11). The values for the importance factors are given for the various importance classes in the National Annex. In addition, the importance classes for various types of buildings are described in the same section.[77]

Table 3.11: *Importance classes for buildings. (Eurocode 8)*[xxxv]

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

3.3.3.2 Structural Analysis

3.3.3.2.1 Modeling

The primary objective of a building model is to represent the distribution of stiffness and mass sufficiently. It is important that all significant deformation shapes and inertia forces are properly accounted for under the considered seismic loading. The model must also include the distribution of strength in the case of non-linear analysis. The deformability of the building is also affected by the behavior of the joints, and this contribution should be accounted for in the model. Non-structural

[77] CEN, *Eurocode 8,* Section 4.2

elements can also influence the response of the primary seismic structure and should be accounted for as well.

Generally, a structure can be regarded as a series of vertical and lateral load resisting systems, which are connected by horizontal diaphragms. When these diaphragms are assumed to be rigid in their respective planes, the masses and moments of inertia can be lumped at the center of gravity.

3.3.3.2.2 Methods of analysis

Linear-elastic behavior of a structure is the basis of determination of seismic effects. The reference method for determining the seismic effects is the modal response spectrum analysis. This is done using a linear-elastic model of the structure and the design spectrum for the given situation. Depending on the structural characteristics of the building, either the lateral force method of analysis or the modal response spectrum analysis may be used. As an alternative to a linear method, a non-linear method such as a static pushover analysis or a dynamic time history analysis may also be used.

3.3.3.2.2.1 Lateral force method of analysis

The lateral force method can be applied in the analysis of buildings with response that is not significantly affected by contributions from modes of vibration that are higher than the fundamental mode. This requirement is deemed to be satisfied in buildings that have fundamental periods of vibration, T_1, in the two main directions smaller than $4 \cdot T_C$ and 2.0 s, and meet the criteria for regularity in elevation. T_C is the upper limit of the period of the constant spectral acceleration branch.

The seismic base shear force, F_b, is determined using the expression:

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

$S_d(T_1)$ is the value of the design spectrum at period T_1, m is the total mass of the building, and λ is the correction factor that accounts for the fact that in buildings with at least three stories and translational degrees of freedom in each horizontal direction, the effective modal mass of the fundamental mode is smaller than the total building mass. λ is set equal to 0.85 if $T_1 \leq 2 \cdot T_C$ and the building has more than two stories. This factor is otherwise set equal to 1.0.

The fundamental period, T_1, of a building can be determined by using expressions based on methods of structural dynamics. For buildings with heights of up to 40 m, the value of T_1 can be approximated by using the expression:

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

where C_t has different values based on the structural system and H is the height of the building. T_1 can also be approximated using the expression:

$$T_1 = 2 \cdot \sqrt{d}$$

where d is the lateral elastic displacement of the top of the building due to the gravity loads applied in the horizontal direction.

3.3.3.2.2.2 Modal response spectrum analysis

The modal response spectrum analysis is used for buildings that do not satisfy the conditions for applying the lateral force method of analysis. The response of all modes of vibration that give a considerable contribution to the global response must be evaluated. This requirement can be deemed to be satisfied if the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure or if all modes with effective modal masses greater than 5% of the total mass are taken into account. If this requirement cannot be satisfied, the minimum number of modes, k, to be taken into account in a spatial analysis should satisfy the conditions $k \geq 3 \cdot \sqrt{n}$ and $T_k \leq 0.20$ s. In these expressions n is the number of stories above the foundation or top of a rigid basement and T_k is the period of vibration of mode k.

When a spatial model is used for the analysis, the accidental torsional effects can be determined as the envelope of the effects resulting from static loading, consisting of sets of torsional moments about the vertical axis of each story. This can be expressed as:

$$M_{ai} = e_{ai} \cdot F_i$$

where M_{ai} is the torsional moment applied at story i about its vertical axis, e_{ai} is the accidental eccentricity of story mass i, and F_i is the horizontal force acting on story i.[78]

3.3.4 Specific Rules for Steel Buildings

3.3.4.1 Materials

In order for the dissipative zones in a structure to form where the design intended them, the distribution of material properties, such as yield strength and toughness, must be desirable. The actual maximum yield strength $f_{y,max}$ of the steel of dissipative zones must satisfy the expression $f_{y,max} \leq 1.1\ \gamma_{ov}\ f_y$, where γ_{ov} is the overstrength factor used in design and f_y is the nominal yield strength. In the National Annex, the overstrength factor is set to $\gamma_{ov} = 1.25$. [79]

3.3.4.2 Structural Types and Behavior Factors

Steel buildings are assigned to a structural type according to the behavior of their primary resisting structure under seismic events. These types include moment resisting frames, frames with concentric bracings, frames with eccentric bracings, inverted pendulum structures, structures with concrete cores or concrete walls, moment resisting frames combined with concentric bracings, and moment resisting frames combined with infills. In order for energy to be dissipated by means of cyclic bending, the dissipative zones should be located in plastic hinges in the beams or the beam column joints. In frames with concentric bracings, the dissipative zones should mainly be located in the tensile diagonals.

The behavior factor, q, accounts for the energy dissipation capacity of the structure. For regular structural systems, the behavior factor, q, should be taken to the reference values given in a table in the code. The values for q are assigned to the

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[78] CEN, *Eurocode 8,* Section 4.3

[79] CEN, *Eurocode 8,* Section 6.2

different structural types in ductility class medium (DCM), and are set to either 2 or 4 for steel structures.[80]

3.3.4.3 Design Criteria and Detailing Rules for Moment Frames

In the design of moment resisting frames, it is required that the plastic hinges form in the beams or in the connections of the beams to the columns, but not in the columns themselves. This requirement does not need to be accounted for at the base of the frame, at the top level of multi story buildings and for single story buildings.

Assuming the formation of a plastic hinge at one end of a beam, the beams should be verified as having sufficient resistance against lateral and lateral torsional buckling. The most stressed beam end in the seismic design situation is the end that should be considered. For plastic hinges occurring in beams, it should be verified that compression and shear forces do not decrease the full plastic moment of resistance and rotation capacity.

Considering the most unfavorable combination of the axial force and bending moments, columns are to be verified in compression. The connections of the beams to the columns should be designed for the required degree of overstrength, if the structure is designed to dissipate energy in the beams.[81]

3.3.5 Specific Rules for Reinforced Concrete Buildings

3.3.5.1 Design Concepts

3.3.5.1.1 Energy dissipation and ductility classes

Earthquake resistant design of concrete structures is used in order to provide the building with an adequate capacity to dissipate energy without substantial reduction of its overall resistance. Adequate resistance of all structural elements is to be provided in the prescribed seismic design situation. Non-linear deformation demands in critical regions should correspond with the overall ductility assumed in the calculations.

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[80] CEN, *Eurocode 8,* Section 6.3

[81] CEN, *Eurocode 8,* Section 6.6

The design of earthquake resistant buildings consists of providing energy dissipation capacity and an overall ductile behavior. Ductile behavior is ensured if the ductility demand involves distribution of the mass of the structure to different elements and locations of all its stories. This means that ductile modes of failure, such as flexure, should precede brittle failure modes, such as shear, with sufficient reliability.

Depending on their energy dissipation capacity, concrete buildings designed in accordance with the previous paragraph are classified in two ductility classes, DCM (medium ductility) and DCH (high ductility). Both classes correspond to buildings designed and detailed according to specific earthquake provisions. These provisions ensure that the structure develop the stable mechanisms associated with large dissipation of energy under repeated load reversal, without suffering brittle failures.

Specific provisions for all structural elements are to be satisfied to provide the appropriate amount of ductility in ductility classes M and H. With different available ductility in the two classes, different values of the behavior factor, q, are used. In Norway, the behavior factor is not to exceed that of ductility class M (DCM).[82]

3.3.5.1.2 Structural types and behavior factors

The code states that concrete structures are to be classified into certain structural types. These types include frame systems, dual systems, ductile wall systems, systems of large lightly reinforced walls, inverted pendulum systems, and torsionally flexible systems. With the exception of the torsionally flexible systems, a structure may be classified to one type of system in one horizontal direction and to another in the other horizontal direction. Certain requirements of the torsional rigidity of the system elements are given in the code.

The value of the behavior factor, q, introduced earlier to account for the energy dissipation capacity, is derived for each design direction according to the expression:

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[82] CEN, *Eurocode 8,* Section 5.2.1

$q = q_o k_w \geq 1.5$

where q_o is the basic value of the behavior factor, dependent on the type of the structural system and on its regularity in elevation, and k_w is the factor reflecting the prevailing failure mode in structural systems with walls. The basic value of the behavior factor, q_o, ranges from 1.5 to 3.0 for buildings that are regular in elevation. The factor k_w is to be taken as 1.0 for frame and frame equivalent dual systems, $(1 + \alpha_o)/3 \leq 1$, but not less than 0.5, for wall, wall equivalent, and torsionally flexible systems. α_o is the prevailing aspect ratio of the walls of the structural system, and may be determined from the expression $\alpha_o = \Sigma h_{wi} / \Sigma l_{wi}$, where h_{wi} is the height of wall i and l_{wi} is the length of the section of wall i.

For buildings that are regular in elevation, the basic values of the behavior factor for various structural types are given in a table. For buildings that are not regular in elevation, the basic values of the behavior factor should be reduced by 20%.[83]

3.3.5.1.3 Design criteria

Brittle failure or other undesirable failure mechanisms must be prevented. This is done by obtaining the design load effects of selected regions from equilibrium conditions, given that plastic hinges are formed in the nearby areas.

The potential regions for plastic hinge formation must have high plastic rotational capacities in order to achieve the required overall ductility of a structure. This is satisfied if sufficient curvature ductility is provided in all critical regions of primary seismic elements, and local buckling of compressed steel is prevented.

It is desired to provide a high degree of redundancy combined with a high capacity of redistributing the internal forces of a structure. This leads to a more widely spread energy dissipation and an increase in the total dissipated energy. As a result,

[83] CEN, *Eurocode 8,* Section 5.2.2

structural systems of lower static indeterminacy are assigned lower values of the behavior factor. The required capacity of redistribution is achieved through the use of local ductility rules.

Seismic loads are known to be highly uncertain due to the random nature of earthquakes. There is also a high level of uncertainty of post-elastic cyclic behavior of concrete structures. The overall uncertainty is therefore substantially higher for seismic loads than for non-seismic loads. As a consequence, measures must be taken to reduce the uncertainties related to the configuration, analysis, resistance and ductility of a structure. Geometric errors can in some cases produce important resistance uncertainties. To minimize this type of uncertainty, certain measures should be taken. These measures include respecting specific minimum dimensions of structural elements and limiting story drifts which in turn limits the P-Δ effects. The measures also involve continuing a substantial percentage of top reinforcement of beams along its entire length, and providing minimum reinforcement at the relevant side of beams to account for reversal of moments not predicted by analysis. In order to minimize ductility uncertainties, a minimum of local ductility must be provided regardless of the adopted ductility class. Also, a minimum amount of tension reinforcement must be provided to avoid brittle failure.[84]

3.3.5.1.4 Safety verification

The possible strength degradation of the materials due to cyclic deformations must be taken into account using the partial factors for material properties γ_c and γ_s. The values for the partial factors are given in the National Annex to be $\gamma_c = 1.5$ and $\gamma_s = 1.15$ for DCM.[85]

3.3.5.2 Design for Ductility Class Medium (DCM)

3.3.5.2.1 Geometrical constraints and materials

[84] CEN, *Eurocode 8,* Section 5.2.3

[85] CEN, *Eurocode 8,* Section 5.2.4

For concrete used in primary seismic elements, the concrete class must not be lower than C16/20. Only ribbed bars are to be used as reinforcing steel in critical regions of primary seismic elements. This requirement does not apply for stirrups and crossties.

In order to achieve efficient transfer of cyclic moments from a primary seismic beam to a column, the eccentricity of the beam axis relative to the axis of the column it frames into must be limited. To meet this requirement, the distance between the axes of the two members should be limited to less than $b_c/4$, where b_c is the largest cross sectional dimension of the column.[86]

3.3.5.2.2 Design load effects

Taking into account second order effects and the capacity design requirements, the design values of bending moments and axial forces are obtained from the analysis of the structure for the seismic design situation.

The design shear forces in primary seismic beams are determined in accordance with the capacity design rule. This is done on the basis of equilibrium of the beam under transverse load acting on it, and for end moments $M_{i,d}$, corresponding to plastic hinge formation. The plastic hinges are assumed to form at the ends of the beams or in the vertical elements connected to the joints where the beam ends are framed into. The end moments can be determined from the expression:

$$M_{i,d} = \gamma_{Rd} M_{Rb,i} \min(1, \Sigma M_{Rc}/ \Sigma M_{Rb})$$

where γ_{Rd} is the factor accounting for possible overstrength due to steel strain hardening, which in the case of DCM beams may be taken as 1.0. $M_{Rb,i}$ is the design value of the beam moment of resistance at end i. ΣM_{Rc} and ΣM_{Rb} are the sum of the design values of the moments of resistance of the columns and beams framing into the joint, respectively.

[86] CEN, *Eurocode 8,* Section 5.4.1

The design values of shear forces in primary columns are also determined in accordance with the capacity design rule. This is done on the basis of the equilibrium of the column under end moments, which correspond to plastic hinge formation for positive and negative direction of seismic loading. The plastic hinges are assumed to form at the ends of the beams connected to the joints where the column end are framed, or at the ends of the columns.[87]

3.3.5.2.3 ULS verification and detailing

The critical region of primary seismic beams is the distances from the end of a beam framing into a column to a critical length equal to the depth of the beam. In addition, any other cross-sections expected to yield in the seismic design situation are considered critical regions.

The local ductility requirement in the critical regions of primary beams is satisfied if two conditions are met. The first condition states that the reinforcement placed in the compression zone of a beam must be at least half of that placed in the tension zone. The second condition states that the reinforcement ratio of the tension zone ρ must be equal to or less than a value ρ_{max} given by the expression:

$$\rho_{max} = \rho' + \frac{0.0018}{\mu_{\varphi}\varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}}$$

where ρ' is the reinforcement ratio in the compression zone, μ_{φ} is the curvature ductility factor, $\varepsilon_{sy,d}$ is the design value of steel strain at yield, f_{cd} is the design value of concrete compressive strength, and f_{yd} is the design value of the yield strength of steel. Other detailing requirements, such as the minimum bar size for stirrups and their maximum spacing, are also given in the code.

Flexural and shear resistance of beams and columns must be computed in accordance with Eurocode 2: Design of concrete structures, using the values of the imposed forces from the analysis in the seismic design situation. For columns, the

[87] CEN, *Eurocode 8,* Section 5.4.2

reinforcement ratio must not be less than 0.01 and not more than 0.04. To ensure the integrity of the beam-column joints, at least one bar is to be provided between the corner bars along each column side. To ensure a minimum ductility and to prevent local buckling of the longitudinal bars, stirrups and cross-ties of at least 6 mm must be provided. The pattern of stirrups must be such that the triaxial stress conditions produced by them are beneficial for the column cross section.[88]

3.4 Evaluation of the Building Codes

3.4.1 Base Shear

In evaluating the code forces, the significance of the response contributions of the higher vibration modes is essential to the dynamic response of buildings. The combined responses of the second and higher modes mainly depend on two parameters. These parameters are the fundamental period, T_1, and the beam to column stiffness ratio, ρ_{bc}. The base shear of a building with T_1 within the acceleration sensitive region of the spectrum is mostly due to the first mode. However, the higher mode responses can be considerable for buildings with T_1 in the velocity and displacement sensitive regions of the spectrum.

When the total weight of the building, W, is used instead of the first mode effective weight, W_1*, as in the building codes, the base shear is overestimated. However, this overestimation in base shear may not be sufficient to compensate for the higher mode response of the building. Both the 2006 IBC and Eurocode 8 ignore the higher mode response and specify the seismic coefficient in terms of spectral acceleration. The 2006 IBC and Eurocode 8 deal with higher mode contribution to base shear in a different way. They do not permit the use of the equivalent lateral force procedure for buildings with T_1 exceeding 2.0 s and 3.5 T_c, respectively, where T_c is the period separating the acceleration and velocity sensitive regions of the spectrum. As a consequence, these buildings must be evaluated by the use of modal analysis.[89]

3.4.2 Story Shears and Equivalent Static Forces

[88] CEN, *Eurocode 8,* Section 5.4.3

[89] Chopra, A.K., *Dynamics of Structures,* Section 21.6

The lateral forces are specified in the codes in terms of the base shear. The story shears are then provided by using these forces in static analysis of the structure. When the fundamental period of a building is in the acceleration sensitive region of the spectrum, the distribution of the lateral forces and story shears specified by the codes are practically identical. When T_1 increases, the code distributions for lateral forces and story shears differ increasingly between the codes, and between the codes and the dynamic response. The code formulas clearly do not follow the results of dynamic response very closely or recognize that dynamic response is affected by the important building parameters.[90]

3.4.3 Overturning Moments

When the fundamental period of a building is in the acceleration sensitive region of the spectrum, and even extending into the velocity sensitive region, the distribution of overturning moments given in the codes are close to each other and to the theoretical dynamic response. As the fundamental period increases and the higher mode response becomes increasingly significant, there is an increased difference in code values relative to the dynamic response. Since the higher mode response contributions to the overturning moments are less significant than for story shears, the difference in overturning moments is much smaller.

Some codes introduce a reduction factor to account for the fact that higher vibration modes contribute more to shears than to overturning moments. The dynamically computed story shears are to be provided by the lateral forces specified in the building codes. The overturning moments will be overestimated if they are computed from these lateral forces. Although it is not supported by the results of elastic dynamic analysis, the 2006 IBC and Eurocode 8 have chosen not to permit any reduction in overturning moments.[91]

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[90] Chopra, A.K., *Dynamics of Structures,* Section 21.7

[91] Chopra, A.K., *Dynamics of Structures,* Section 21.8

4 Case Studies Using Both Codes

4.1 Seismic Analysis Software

4.1.1 Introduction

4.1.2 KPFF Consulting Engineers uses two major software applications to conduct complex analysis of buildings. The same two programs were used in the case studies of this thesis. A brief introduction to each of the two software applications is given in the sections below. This section concludes with a summary of important considerations used when modeling lateral systems.

4.1.3 RAM Structural System

The RAM Structural System is powerful and versatile special purpose software for the analysis and design of structures. The RAM Structural System automates the process of calculating tributary loads, live load reduction, gravity member selection, frame analysis, drift control, frame member and joint code checking, special seismic provisions member and joint checking. By automating these tedious and time consuming processes, an accurate design can be obtained quickly. Different framing configurations may be examined in a short period of time, resulting in a substantially more economical design. The interface with CAD software permits rapid generation of framing plans, saving significant drafting time and reducing the errors associated with manual information transfer.

The RAM Structural System is composed of a number of special purpose modules, which are launched from the RAM Manager. The RAM Modeler accommodates the creation of a model of the entire structure including beam, column, brace and wall geometry and locations. Slab properties, openings and edges are also assigned in this module. The result is a comprehensive database of building data, which can be accessed by the analysis and design modules, providing a completely integrated solution.

The RAM Steel Beam Design module provides a powerful capability for the gravity design of composite and noncomposite beams and girders. In addition to the automated optimization of beam sizes, existing conditions can be checked. Tributary loads from a user-defined surface, line and point load patterns, loads on girders due to beams which frame into them, live load reduction factors based on one of several available building codes, and effective flange width are all automatically calculated. Special design considerations, such as depth restrictions, can be specified. Designs can be performed using one of the included steel design codes.

The RAM Steel Column Design module provides a powerful capability for the design of gravity columns and their baseplates. Axial loads, unbalanced moments, live load reductions and bracing conditions are automatically calculated. Optimum sizes may be obtained or existing conditions analyzed.

RAM Frame provides the capability to perform a full three-dimensional static and dynamic frame analysis of the lateral system in the structure. Member locations and geometry, gravity loads with their corresponding live load reduction factors and story mass properties are obtained directly from the database. Lateral wind and seismic loads may be generated based on building code requirements or specified as user-defined story or nodal loads. In the analysis mode, frames of any material and type, including moment frames, braced frames and walls can be analyzed. In steel mode, a code check based on a selected steel design code can be performed for all lateral steel members and moment frame elements.[92]

4.1.4 ETABS

ETABS is a special purpose analysis and design program developed specifically for building systems. ETABS features a graphical interface coupled with modeling, analytical, and design procedures, all integrated using a common database. ETABS can handle the largest and most complex building models, including a wide range of nonlinear behaviors, making it an important tool for structural engineers in the building industry.

ETABS is a completely integrated system. Embedded beneath a simple, and fairly intuitive user interface are very powerful numerical methods, design procedures and international design codes, all working from a single comprehensive database. This

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[92] Bentley Systems Inc., *RAM Structural System Reference Manual*

integration means that only one model of the floor systems and the vertical and lateral framing systems needs to be created to analyze and design the entire building. No external modules are required, because everything is integrated into one versatile analysis and design package with one Windows-based graphical user interface. The effects on one part of the structure from changes in another part are instantaneous and automatic. The integrated components include a modeling module, a seismic and wind load generation module, a gravity load distribution module, a finite element based linear static and dynamic analysis module, and various design modules.

The ETABS building is idealized as a group of area, line and point objects. Those objects are used to represent wall, floor, column, beam, brace and link/spring physical members. The basic frame geometry is defined with reference to a simple three-dimensional grid system. With relatively simple modeling techniques, very complex framing situations may be considered.

The buildings may be unsymmetrical and non-rectangular in plan. Torsional behavior of the floors and interstory compatibility of the floors are accurately reflected in the results. Semi-rigid floor diaphragms may be modeled to capture the effects of in-plane floor deformations.

The effects of the finite dimensions of the beams and columns on the stiffness of a frame system are included using end offsets that can be automatically calculated. The floors and walls can be modeled as membrane elements with in-plane stiffness only, plate bending elements with out-of-plane stiffness only or full shell-type elements, which combine both in-plane and out-of-plane stiffness. Floor and wall objects may have uniform load patterns in-lane or out-of-plane, and they may have temperature loads. The column, beam, brace, floor and wall objects are all compatible with one another.

Static analyses for user specified vertical and lateral floor or story loads are possible. If floor elements with plate bending capability are modeled, vertical uniform loads on the floor are transferred to the beams and columns through bending of the floor elements. Otherwise, vertical uniform loads on the floor are automatically converted to span loads on adjoining beams, or point loads on adjacent columns, thereby automating the tedious task of transferring floor tributary loads to the floor beams without explicit modeling of the secondary framing.

The program can automatically generate lateral wind and seismic load patterns to meet the requirements of various building codes. Three-dimensional mode shapes and frequencies, modal participation factors, direction factors and participating mass percentages are evaluated using eigenvector or Ritz-vector analysis. P-Delta effects can be included with static or dynamic analysis. Response spectrum analysis, linear time history analysis, nonlinear time history analysis, and static nonlinear (pushover) analysis are all possible.

Results from the various static load cases can be combined with each other or with the results from the dynamic response spectrum or time history analyses. Output can be viewed graphically, displayed in tabular output, or sent to a printer. Types of output include reactions and member forces, mode shapes and participation factors, static and dynamic story displacements and story shears, interstory drifts and joint displacements, and more. Import and export of data may occur between third-party applications such as Revit or AutoCAD from Autodesk.[93]

4.1.5 Lateral Modeling Verifications

When the lateral system has been modeled using computer software like those described above, errors can easily occur in the process. For this reason, there are certain verifications that must be made to ensure that the model is accurate and that the analysis will generate correct values for the imposed forces.

First of all the basic verifications must be performed to verify that the correct material properties, units, dimensions, boundary conditions and object assignments have been selected.

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[93] Computers & Structures Inc., *ETABS Reference Manual*

In lateral analysis, the mass of the structure is one of the most significant contributors to the design forces. The mass of the modeled structure should always be check by a hand calculation. Each diaphragm should be checked to make sure that the mass is accounted for at each level. The assignment of the mass should be coordinated in the settings. When using superimposed loads instead of having the program calculated the self-weight, the settings must be adjusted to make sure that mass is not been accounted for twice.

Analysis of the model must always be run with static lateral loads in each direction. It is much simpler to verify the behavior of the model under static loads. It is important to verify that the applied lateral forces are accounted for and the load path makes sense. For example, if one wall seems to carry the entire load and the other wall carries none, then there must be something wrong.

Make sure that the calculated building period makes sense. The period should be within a reasonable range, and depending on the height of the building this could be between 2 and 10 seconds. If a tall building has a very short period, and a short building has a very long period, it is likely to be some errors in the modeling of the structure.

It is important to make sure that the mass participation factors are reasonable. If any modes have very little participation in any direction, there is probably a problem with the model. Boundary conditions, unconnected elements, unsupported elements, and null assignments must be checked. It is also important to make sure that enough modes are included in the analysis to achieve the code requirement of 90% mass participation.

Investigating the mode shapes can reveal serious problems with the model. It is important to make sure that the modes follow the general pattern for 1st order, 2nd order, etc. shapes. If a very symmetrical building has torsional modes as one of the primary modes, there is also reason for concern in regards to modeling errors.

Making sure that the base shear for dynamic analysis is scaled properly is also vital. According to ASCE 7-05, the dynamic base shear must be scaled up to 85% of static base shear, but it must not be scaled down if it exceeds this value[94].

It must be verified that the center of mass and center of rigidity appear to be in the correct locations.

The use of cracked section properties must be verified if required. This can easily be accomplished by modifying the value for the Modulus of Elasticity, E, or by applying stiffness modification factors to the frame and shell elements.

Verification of proper assignment of diaphragms must be conducted. Any components that are not physically connected to the diaphragm should not be assigned to the diaphragm. It is also important to assign the proper rigidity to the diaphragm, considering aspect ratios, openings etc. A diaphragm is set to act either as rigid, or semi-rigid.

When using concrete coupling beams, it is important to properly account for stiffness degradation, i.e. cracking. It is typically more degraded than the assumed 50% of gross properties.

4.2 Practical Applications of the Codes

4.2.1 Introduction

Two buildings with different seismic resisting systems have been selected for this section. The first building is called 1st and Main, and is steel office building located close to the Willamette river waterfront in downtown Portland. The second is known as the Ardea, and is a concrete residential tower on the South Waterfront also located in Portland. These projects have been chosen for several reasons. The buildings are both high-rise buildings and represent two of the most common systems for seismic resistance. Both structures are designed by KPFF Consulting Engineers, which is the company where I have been working on this thesis. With

[94] ASCE 7-05, Section 12.9.4

both buildings situated in Portland, the seismic criteria are the same for both design situations. The Ardea was recently completed and 1st and Main is currently in the construction phase, which means that they have both been designed using the most recent building codes.

4.2.2 1st and Main Building[95]

4.2.2.1 Project description

Fig. 4.1: *Rendering of the completed 1st and Main building.[xxxvi]*

Project location:

- Portland, OR

Building description:

- 16 stories plus 3 levels of parking
- 346,500 square feet of office space

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[95] http://www.firstandmainportland.com/index.php

- 20,000 square feet of ground-floor retail space
- Total footprint (ground floor): approx. 200 ft x 200 ft

Seismic force resisting system:

- Eccentrically braced steel frames (EBF)

Seismic Design Criteria per 2006 IBC / ASCE 7-05:

- Importance Factor, I_E = 1.0 {Section 3.2.2.2}
- Site Class: B {Section 3.2.4}
- Seismic Design Category: D {Section 3.2.2.3}
- Design Parameters: {Section 3.2.2.1}
  - S_S = 1.048 g
  - S_1 = 0.344 g
  - F_a = 1.00
  - F_v = 1.00
  - S_{MS} = 1.048 g
  - S_{M1} = 0.344 g
  - S_{DS} = 0.699 g
  - S_{D1} = 0.229 g
- Response Modification Factor {Section 3.2.3.2}
  - R_x = 7
  - R_y = 7
- Calculated period
  - T_x = 2.94 s
  - T_y = 2.52 s
- Seismic Response Coefficient {Section 3.2.3.5.2}
  - C_{sx} = 0.031g
  - C_{sy} = 0.031g

Seismic Design Criteria per Eurocode 8:

- Importance Factor, γ_I = 1.0 {Section 3.3.3.1.5}
- Ground Type: A {Section 3.3.2.1}
- Design Ground Acceleration, $a_g = 0.8 \cdot 0.20$ g = 0.16 g = 5.15 ft/s^2

(see Fig.4.2)

- Behavior factor {Section 3.3.4.2}
  - $q_x = 4$
  - $q_y = 4$
- Calculated period
  - $T_x = 2.93$ s
  - $T_y = 2.52$ s
- Design spectrum (see Fig. 4.3) {Section 3.3.2.2.2}
  - $S_{dx}(T_1) = 0.032$ g
  - $S_{dy}(T_1) = 0.032$ g

Analysis procedure:

- Modal response spectrum analysis per ASCE 7-05
- Modal response spectrum analysis per Eurocode 8

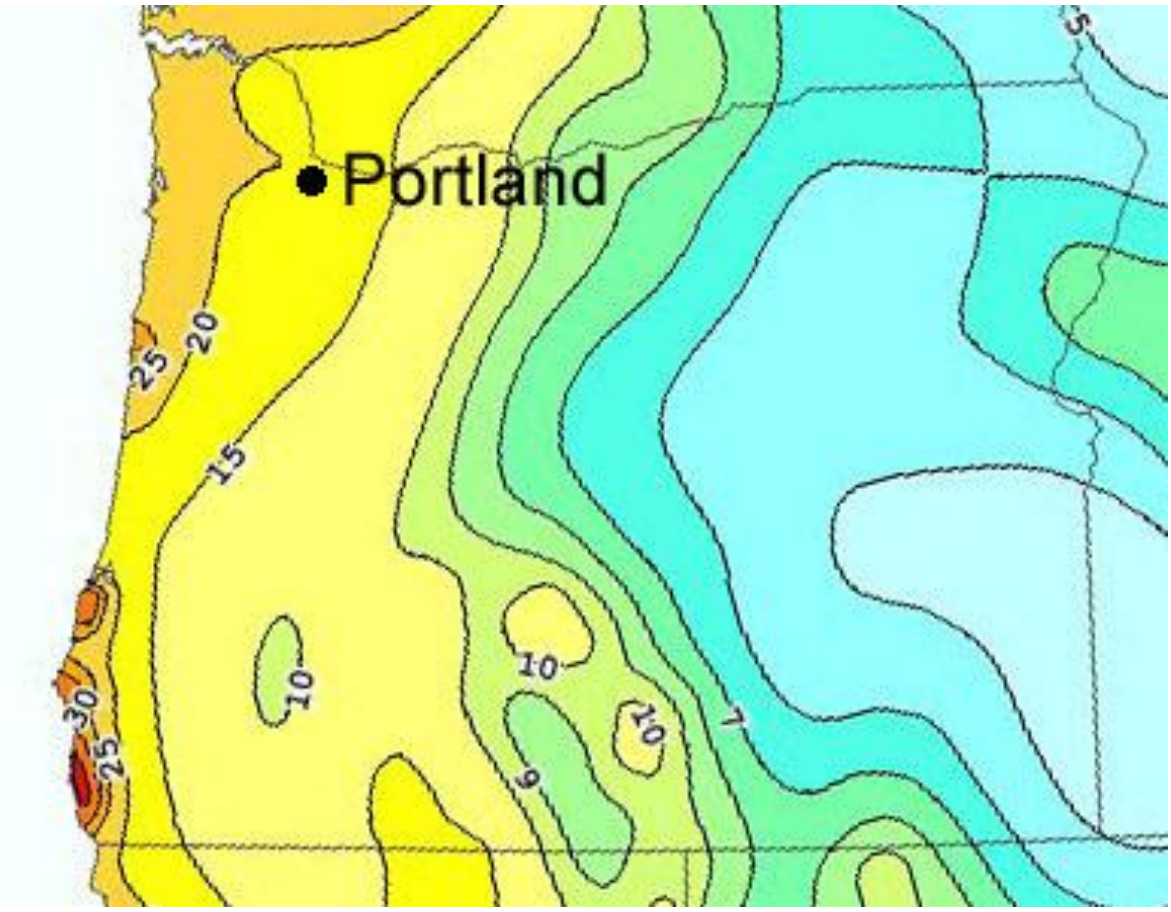


Fig. 4.2: *PGA values in % of gravity for Oregon. (USGS)*[xxxvii]

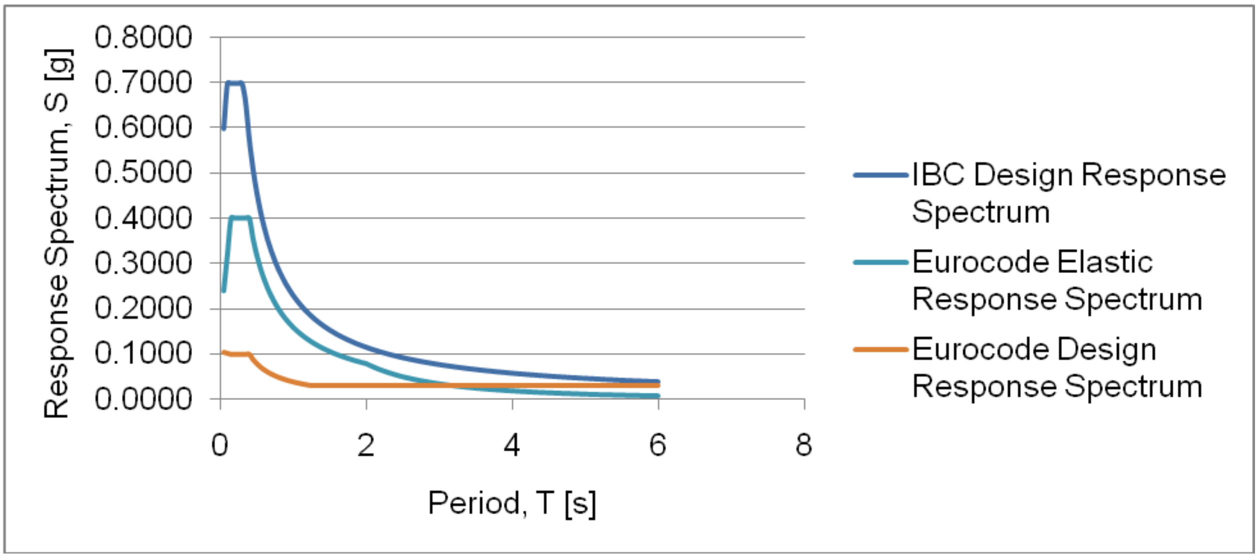


Fig. 4.3: *Response Spectra for the 2006 IBC and Eurocode 8 for 1st and Main.*

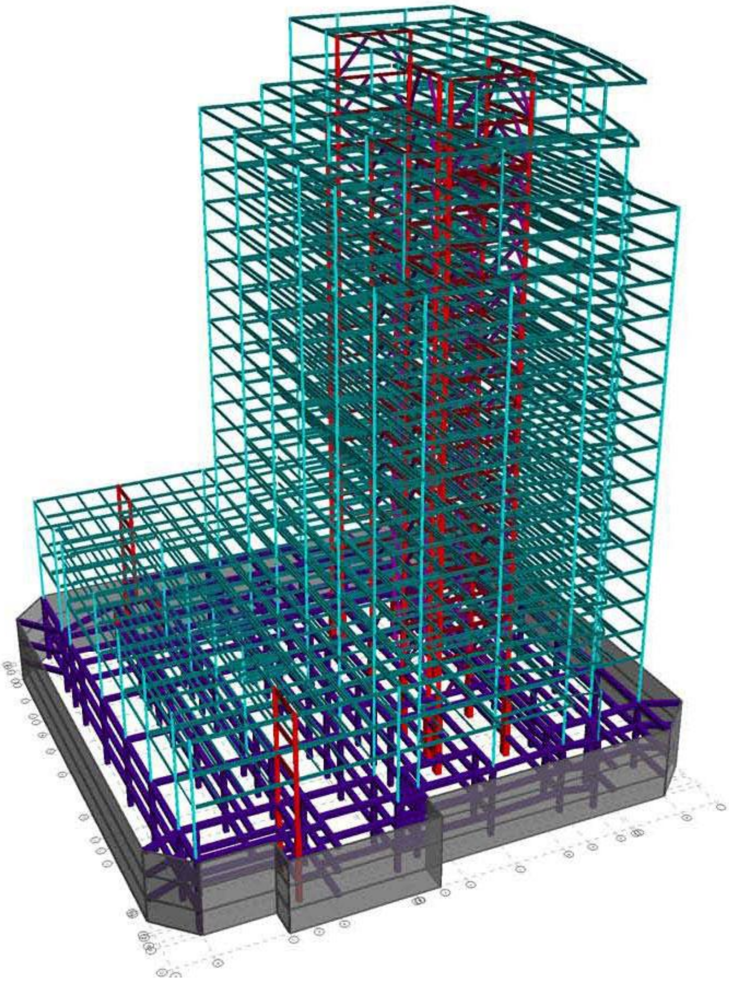


Fig. 4.4: *Structure modeled in RAM Structural System.*

4.2.2.2 Modeling

RAM Modeler was used when modeling this building, and accounted for the entire gravity and seismic system. The modeling was based on structural drawings provided by the architect. The main seismic restraint system consisted of eccentrically braced frames. The reason for this selection was based on architectural and mechanical needs, namely allowing adequate space in the frames for doorways and mechanical ducts.

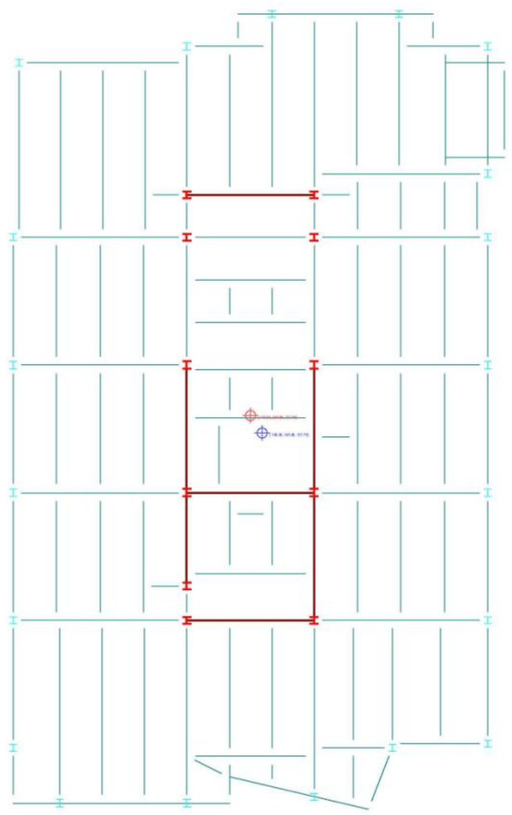


Fig. 4.5: *Plan of typical floor showing centers of mass (red) and rigidity (blue).*

The seismic restraint system was established in the center of the tower to reduce torsional irregularities. This means that the centers of mass and rigidity must be within close proximity (see Fig. 4.5). For the podium levels near the base of the structure, the bulk of the seismic system is offset quite a lot from the center of mass. Additional measures to ensure the stiffness of those levels were taken by adding moment resisting frames near the corners opposite the main seismic resisting system (see Fig 4.4). This corrects the center of rigidity to fall close to the center of mass. As a result of these modifications, the structure was not categorized as a torsionally irregular building. If it had been deemed torsionally irregular, additional provisions would have to be met. The seismic base was set at ground level, and the below grade levels where simplified since they would not be considered in the seismic analysis.

When all structural members had been modeled, including columns, beams, braces, decks and slabs, the program ran a check to make sure that the model was functioning properly without any discrepancies. The first time this check was run, it displayed some components that were malfunctioning as a result of modeling errors. These problems were repaired so that the model would perform as desired.

Part of the modeling process is also applying loads. The program calculates the self-weight of the modeled components, but other dead loads can also occur, such as the weight of partition walls, flooring, mechanical components and cladding. The cladding load was applied along the perimeter of the building. Live loads were also added to each floor depending on the type of occupancy. This building primarily consists of office space, and a live load of 80 psf was applied to all levels. Snow loads are also applied in the modeling module. Seismic and wind loads are lateral loads and are therefore applied in the Frame analysis module.

4.2.2.3 Analysis

Using the frame analysis module of the RAM Structural System, a complete evaluation of the steel frame system was conducted. The first thing to do in this module was to define the load cases. This was done by selecting the type of load, and what code to use in the calculation of the load on the structure. For example, when choosing the seismic load case, the option of several different codes were given. For this case study, both the 2006 IBC and the Eurocode were used. The load cases focused on in these design examples where the dead and live load cases, in addition to seismic and dynamic load cases.

The calculated base shear from the dynamic analysis must be scaled to comply with the ASCE 7-05 requirement that the base shear must be at least 85% of the static base shear calculated in the seismic load case. The static base shear can also be calculated by hand using Section 12.8 of the ASCE 7-05 or Section 4.3.3.2 of Eurocode 8 to verify the output from the model. The scaling of dynamic base shear is done by running an analysis with the unscaled dynamic load case, and using the results to compute scaling factors in accordance with the code (see Table 4.1). When the correct scaling factors have been applied to the dynamic load case, the analysis is run again to attain the correct dynamic base shear and distributed story forces. When using Eurocode 8, however, the results from response spectrum analysis can be used directly. [96]

Table 4.1: *Dynamic scaling in accordance with ASCE 7-05 Section 12.9.4*

R =		7.00	
Scale factor =	X:	0.1429	
	Y:	0.1429	
V_{static}	X:	1004.00	k
	Y:	1025.00	k
$V_{dynamic}$	X:	374.38	k
	Y:	369.12	k
85% V_{static}	X:	853.40	k
	Y:	871.25	k
New scale factor	X:	0.3256	
	Y:	0.3372	

Furthermore, load combinations are generated based on selected load cases. When the analysis is run, the demands of each member in the structure are checked against the standard and seismic steel provisions to determine the capacity. When the analysis is completed, the demand-capacity ratio for each member is displayed in a color-coded 3D model. Those components with inadequate capacity (DCR of 1.0 or higher) are displayed in a red color. By clicking on a red member, the option is given to update the member size to provide acceptable capacity.

When every member of the structure is given sufficient capacity, the analysis process must be repeated to account for the redistribution of the forces in the global structure. When the entire system is deemed structurally acceptable, all components

[96] E. Booth, D. Key, *Earthquake design practice for buildings*, Section 6.4.2

are displayed as green (see Fig. 4.6). In this design example, no standard steel section could give adequate capacity for the four corner columns at the base. These columns must be specifically designed as a custom section using a wide flange section with plates running along the length of the column on both sides.

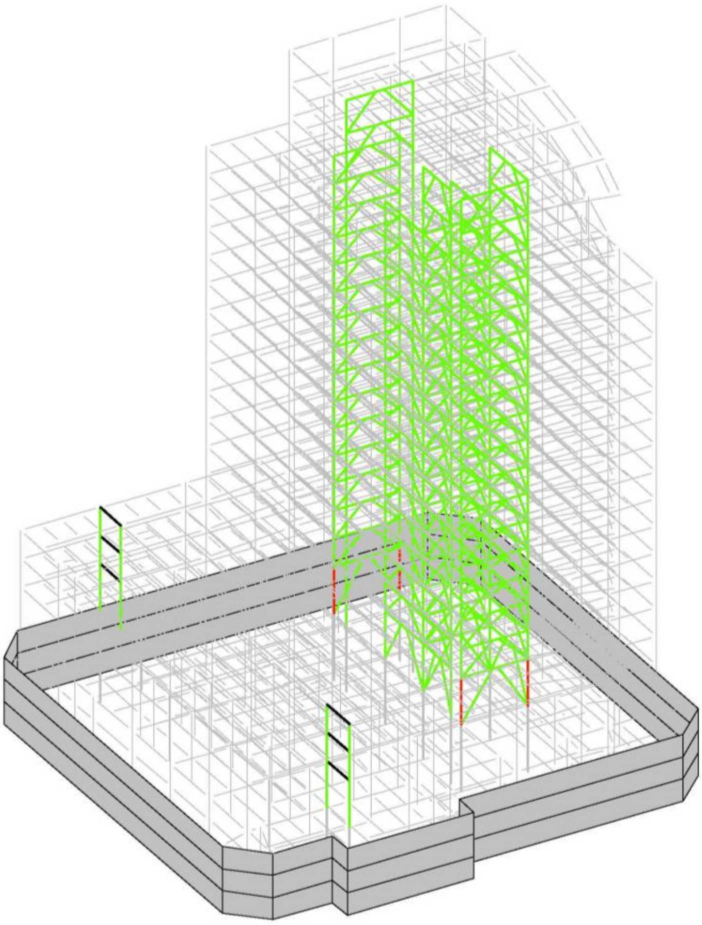


Fig. 4.6: *Completed analysis of structure in RAM Frame.*

4.2.2.4 Results

4.2.2.4.1 Static Analysis

Equivalent Lateral Force Procedure per ASCE 7-05 Section 12.8

Building Period:	$T =$	2.9	s	{ASCE 7-05 Section 12.8.2}
Seismic Parameters:	$S_{DS} =$	0.699	g	{ASCE 7-05 Section 11.4}
	$S_{D1} =$	0.229	g	
Response modification factor:	$R =$	7		{ASCE 7-05 Table 12.2-1}
Importance factor:	$I_E =$	1		
Long-period transition period:	$T_L =$	16	s	{ASCE 7-05 Section 11.4.5}
Base Shear:	$V = C_S W$			

	C_S =	0.0999	
or	C_S =	0.0113	{Maximum}
or	C_S =	0.0308	{Minimum}
USE	**C_S =**	**0.0308**	

V = 0.0308 W

Vertical distribution of forces per ASCE 7-05 Section 12.8.3

Building period =	2.5	s	k =	1.800	
Building weight =	33531	k	Base Shear, V =	0.0308	W
Base Shear, V =	1031.29	k			

Level	Floor height [ft]	Elevation h_x [ft]	w_x [k]	$w_x h_x^k$	C_{vx}	F_x [k]	V_x [k]
19	12.25	228.00	321.2	5637133	0.032	32.57	
18	12.25	215.75	610.3	9697389	0.054	56.03	32.57
17	12.25	203.50	1605	22955723	0.129	132.63	88.60
16	12.25	191.25	1702.2	21771742	0.122	125.79	221.22
15	12.25	179.00	1785.1	20267398	0.114	117.10	347.01
14	12.25	166.75	1788.6	17874411	0.100	103.27	464.11
13	12.25	154.50	1792	15610229	0.087	90.19	567.38
12	12.25	142.25	1795.2	13477425	0.076	77.87	657.57
11	12.25	130.00	1798.3	11480483	0.064	66.33	735.43
10	12.25	117.75	1801.9	9626321	0.054	55.62	801.76
9	12.25	105.50	1805.7	7915895	0.044	45.73	857.38
8	12.25	93.25	1810.8	6356798	0.036	36.73	903.11
7	12.25	81.00	1816.1	4947807	0.028	28.59	939.84
6	12.25	68.75	1821.4	3693994	0.021	21.34	968.43
5	12.25	56.50	1827.9	2603992	0.015	15.04	989.77
4	12.75	43.75	3151.8	2833473	0.016	16.37	1004.81
3	12.75	31.00	3212.8	1553584	0.009	8.98	1021.19
2	21.00	10.00	3084.9	194644	0.001	1.12	1030.16
Ground	10.00	0.00					1031.29
Σ			33531	178498443.6		1031.29	

Lateral force method per Eurocode 8 Section 4.3.3.2

Fundamental Period:	T_1 =	2.9	s
Peak ground acceleration:	a_{g40hz} =	0.2	g
Reference pga on type A ground:	a_{gR} =	0.16	g
Importance factor:	γ_I =	1	
Design pga on type A ground:	a_g =	0.16	g
Lower limit period of constant a:	T_B =	0.05	s
Upper limit period of constant a:	T_C =	0.25	s
Period defining constant disp.:	T_D =	1.2	s
Soil factor:	S =	1	
Behavior factor:	q =	6	
Design Response Spectrum:	S_d =	0.0320	g
Correction factor;	λ =	1.00	
Base shear :	F_b =	$S_d(T_1)\ m\ \lambda$	

$$\boxed{\mathbf{F_b = 0.0320 \cdot m\ \lambda}}$$

Vertical distribution of forces per Eurocode 8 Section 4.3.3.2.3

Building period =	2.1	s			
Building weight =	33531	k	Base Shear, F_b =	0.0320	m
Base Shear, F_b =	1073.00	k			

Level	Floor height [ft]	Elevation z_i [ft]	m_i [k]	$z_i m_i$	C	F_i [k]	V_i [k]
19	12.25	228.00	321.2	73234	0.021	22.73	
18	12.25	215.75	610.3	131672	0.038	40.88	22.73
17	12.25	203.50	1605	326618	0.094	101.40	63.61
16	12.25	191.25	1702.2	325546	0.094	101.06	165.01
15	12.25	179.00	1785.1	319533	0.092	99.20	266.07
14	12.25	166.75	1788.6	298249	0.086	92.59	365.27
13	12.25	154.50	1792	276864	0.080	85.95	457.86
12	12.25	142.25	1795.2	255367	0.074	79.28	543.81
11	12.25	130.00	1798.3	233779	0.068	72.58	623.09
10	12.25	117.75	1801.9	212174	0.061	65.87	695.66
9	12.25	105.50	1805.7	190501	0.055	59.14	761.53
8	12.25	93.25	1810.8	168857	0.049	52.42	820.67

7	12.25	81.00	1816.1	147104	0.043	45.67	873.09
6	12.25	68.75	1821.4	125221	0.036	38.87	918.76
5	12.25	56.50	1827.9	103276	0.030	32.06	957.63
4	12.75	43.75	3151.8	137891	0.040	42.81	989.69
3	12.75	31.00	3212.8	99597	0.029	30.92	1032.50
2	21.00	10.00	3084.9	30849	0.009	9.58	1063.42
Ground	10.00	0.00					1073.00
Σ			33531	3456332.15		1073.00	

4.2.2.4.2 Dynamic Analysis

Dynamic base shear and story forces using ASCE 7.05 and RAM Structural system

Level	Height [ft]	V_X [k]	V_Y [k]	F_X [k]	F_Y [k]
Roof	236.0	39.48	25.85	0.00	0.00
18th	220.9	39.48	25.85	131.71	94.08
17th	205.8	171.19	119.93	108.14	91.02
16th	193.5	279.33	210.95	67.19	81.96
15th	181.3	346.52	292.91	26.75	69.36
14th	169.0	373.27	362.27	5.77	56.09
13th	156.8	379.04	418.36	8.07	45.05
12th	144.5	387.11	463.41	26.56	36.55
11th	132.3	413.67	499.96	37.00	31.73
10th	120.0	450.67	531.69	33.86	29.17
9th	107.8	484.53	560.86	24.82	29.30
8th	95.5	509.35	590.16	21.56	30.21
7th	83.3	530.91	620.37	25.51	32.85
6th	71.0	556.42	653.22	36.83	35.08
5th	58.8	593.25	688.30	92.18	60.48
4th	46.5	685.43	748.78	103.44	68.80
3rd	33.8	788.87	817.58	64.43	53.68
2nd	21.0	853.30	871.26	-956.49	-1417.02
Ground		-103.19	-545.76		

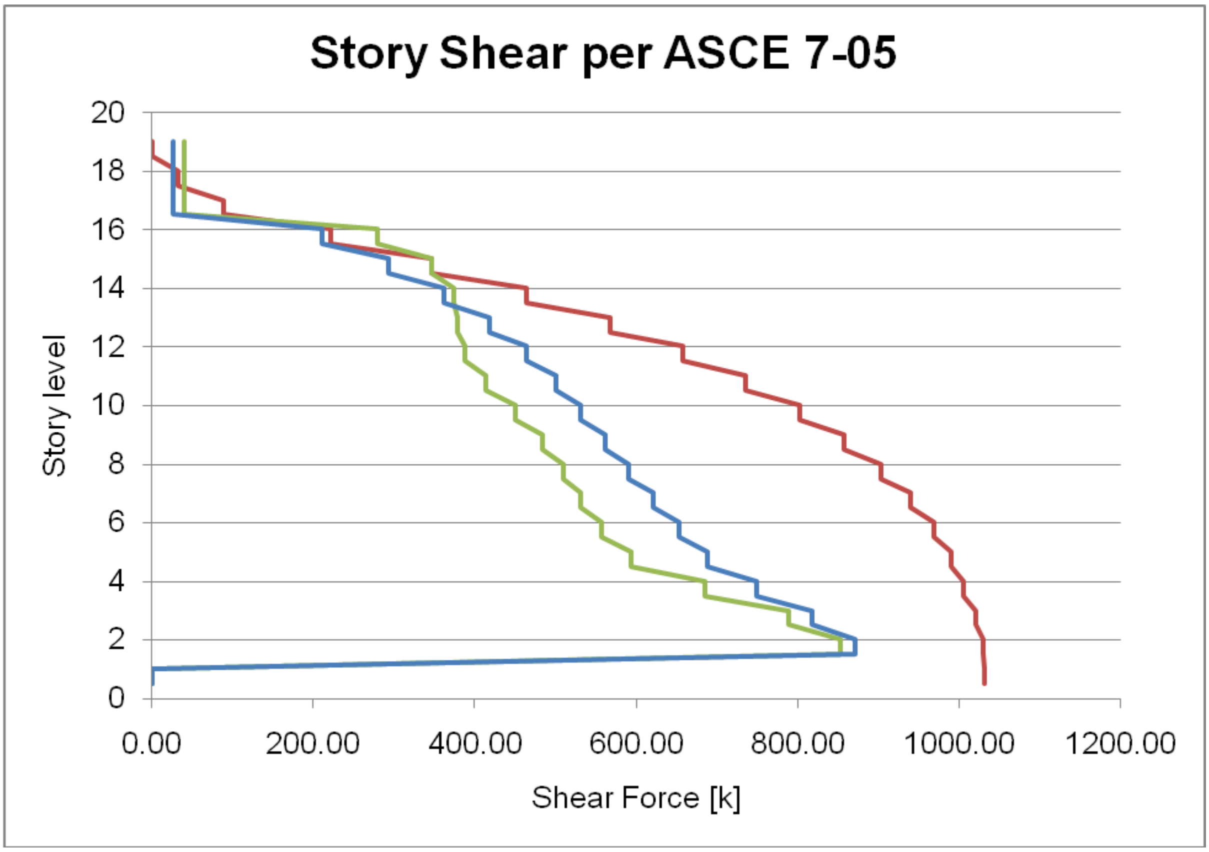


Fig. 4.7: *The red graph is the static story shear and the green and blue graphs are the dynamic story shear in the x and y direction, respectively.*

Dynamic base shear and story forces using Eurocode 8 and RAM Structural System

Level	Height [ft]	V_X [k]	V_Y [k]	F_X [k]	F_Y [k]
Roof	236.0	21.85	13.73	0.00	0.00
18th	220.9	21.85	13.73	72.89	49.65
17th	205.8	94.74	63.38	59.66	48.01
16th	193.5	154.40	111.39	36.62	43.15
15th	181.3	191.02	154.54	13.94	36.38
14th	169.0	204.96	190.92	2.36	29.34
13th	156.8	207.32	220.26	4.15	23.35
12th	144.5	211.47	243.61	14.98	18.67
11th	132.3	226.45	262.28	20.89	16.24
10th	120.0	247.34	278.52	18.83	15.07
9th	107.8	266.17	293.59	13.30	15.28
8th	95.5	279.47	308.87	11.17	15.83
7th	83.3	290.64	324.7	13.32	17.37
6th	71.0	303.96	342.07	19.89	18.31
5th	58.8	323.85	360.38	51.11	32.01
4th	46.5	374.96	392.39	58.01	36.56

3rd	33.8	432.97	428.95	36.29	28.25
2nd	21.0	469.26	457.2	-528.09	-754.51
Ground		-58.83	-297.31		

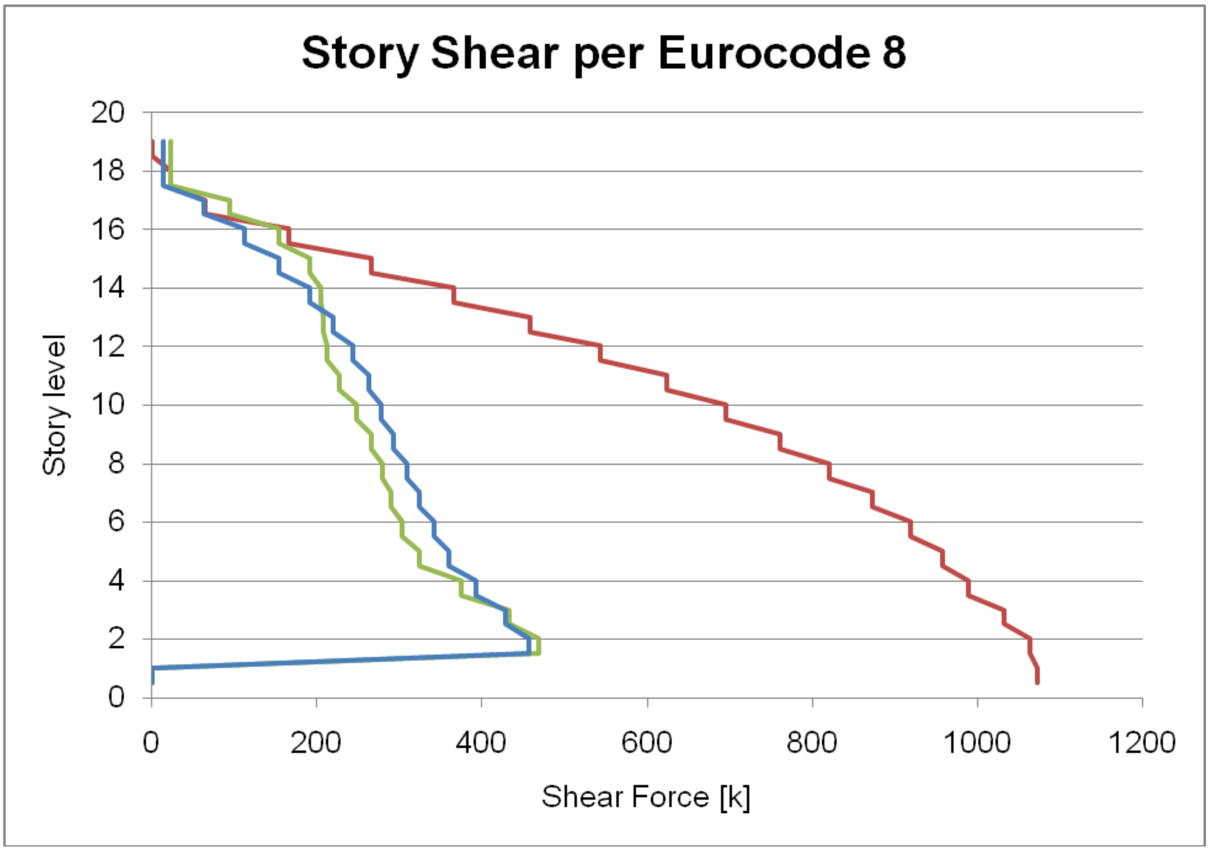


Fig. 4.8: *The red graph is the static story shear and the green and blue graphs are the dynamic story shear in the x and y direction, respectively.*

4.2.3 The Ardea (Block 38)[97]

4.2.3.1 Project description

Fig. 4.9: *Rendering of the completed Ardea tower.*[xxxviii]

Project location:

- Portland, OR

Building description:

- 30 story residential building
- 323 apartment homes and 33 townhomes
- Total footprint (ground floor): approx. 220 ft x 200 ft

[97] http://www.theardea.com/

Seismic force resisting system:

- Dual system: Concrete shear walls and special moment resisting frames (SMRF)

Seismic Design Criteria per 2006 IBC / ASCE 7-05:

- Importance Factor, $I_E = 1.0$ {Section 3.2.2.2}
- Site Class: C {Section 3.2.4}
- Seismic Design Category: D {Section 3.2.2.3}
- Design Parameters: {Section 3.2.2.1}
    - $S_S = 1.048$ g
    - $S_1 = 0.344$ g
    - $F_a = 1.00$
    - $F_v = 1.46$
    - $S_{MS} = 1.048$ g
    - $S_{M1} = 0.502$ g
    - $S_{DS} = 0.699$ g
    - $S_{D1} = 0.335$ g
- Response Modification Factor {Section 3.2.3.2}
    - $R_x = 7$
    - $R_y = 7$
- Calculated period
    - $T_x = 2.10$ s
    - $T_y = 2.10$ s
- Seismic Response Coefficient {Section 3.2.3.5.2}
    - $C_{sx} = 0.031$
    - $C_{sy} = 0.031$

Seismic Design Criteria per Eurocode 8:

- Importance Factor, $\gamma_I = 1.0$ {Section 3.3.3.1.5}
- Ground Type: B {Section 3.3.2.1}
- Design Ground Acceleration, $a_g = 0.8 \cdot 0.20\ g = 0.16\ g = 5.15\ ft/s^2$
- Behavior factor {Section 3.3.5.1.2}
    - $q_x = 3.6$

- $q_y = 3.6$

- Calculated period
  - $T_x = 2.1$ s
  - $T_y = 2.1$ s
- Design spectrum (see Fig. 4.11) {Section 3.3.2.2.2}
  - $S_{dx}(T_1) = 0.032$ g
  - $S_{dy}(T_1) = 0.032$ g

Analysis procedure:

- Modal response spectrum analysis per ASCE 7-05
- Modal response spectrum analysis per Eurocode 8

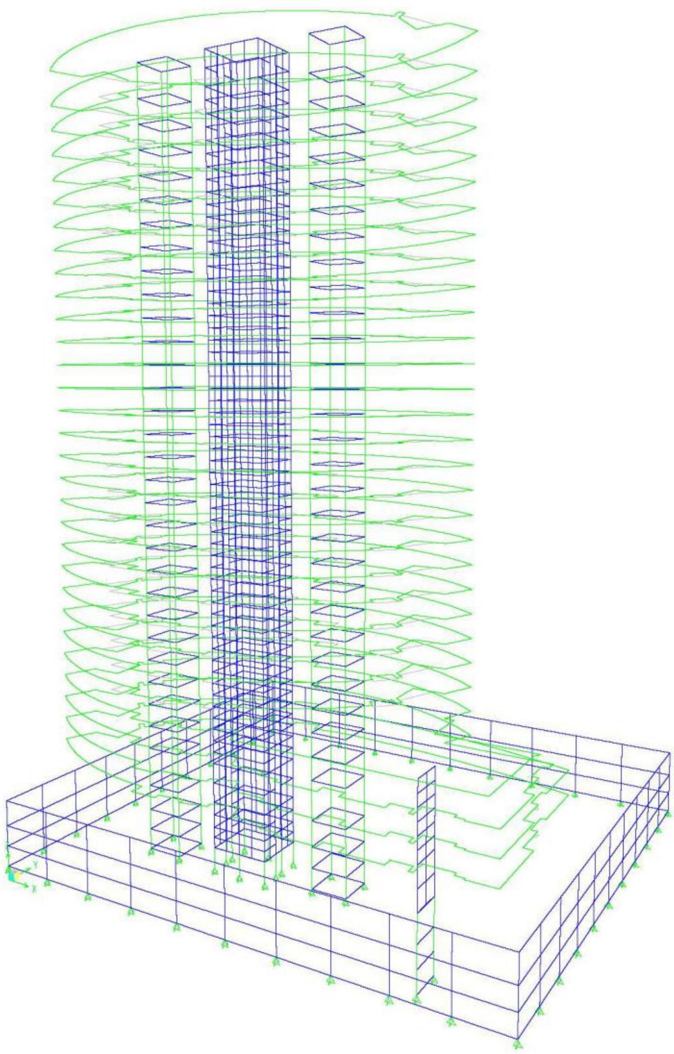


Fig. 4.10: *Structure modeled in ETABS.*

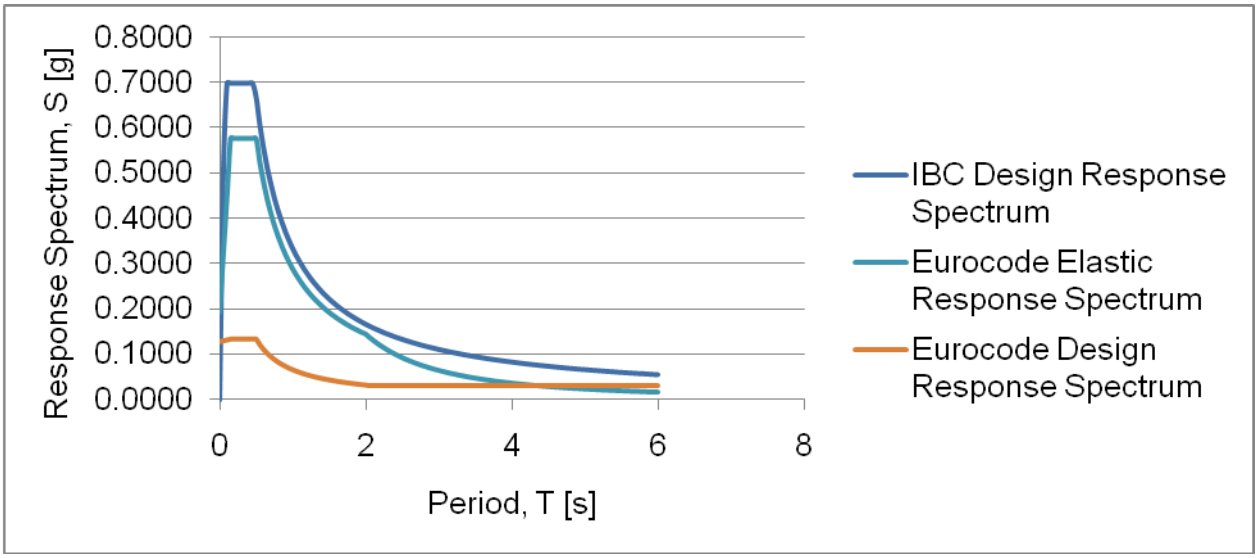
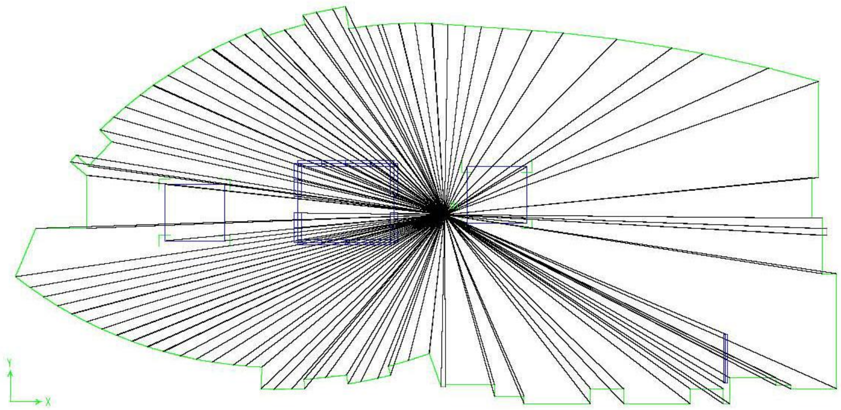


Fig. 4.11: *Response Spectra for the 2006 IBC and Eurocode 8 for the Ardea.*

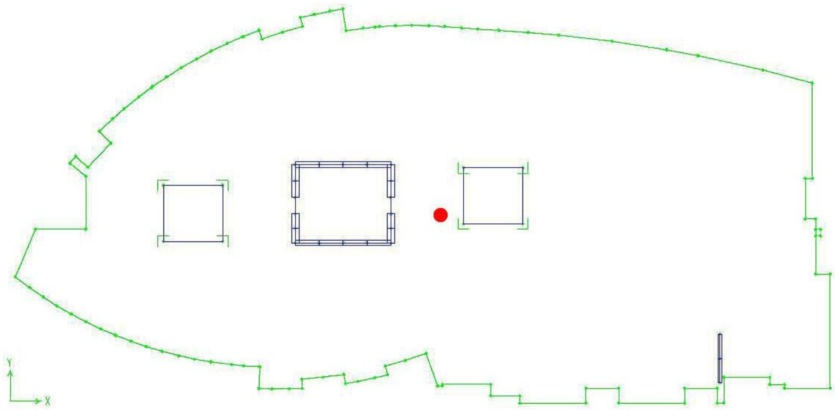
4.2.3.2 Modeling

When modeling this building the analysis software ETABS was used. Since this analysis was restricted to seismic behavior of the building, only the contributing components were modeled. This is a building with a dual system, i.e. a seismic restraint system consisting of both concrete shear walls and moment resisting frames. When using such a system, the 2006 IBC requires that the frames alone must be able to withstand 25% of the seismic load. This building was therefore first modeled with moment frames only, for use in the frames analysis. Later the model was modified to also include the shear walls, which is the main resisting system.

Because of the irregular shape of the diaphragms of the tower, the architectural plans were imported to the model. This gave an accurate floor plan to the various levels without the excessive work required to duplicate the outline of the floor slabs. Even with the irregular shape of the typical floor, the centers of mass and rigidity corresponded well. For the levels near the base, the extent of the podium level diaphragms lead to an additional shear wall being required throughout the podium levels (see Fig. 4.12). The seismic base was set at ground level for this building as well.



(a)



(b)

Fig. 4.12: *Podium level floor plan (a) Center of mass (b) Center of rigidity.*

Since this was a simplified model, all mass that was unaccounted for in the model had to be added as a distributed load. This included mechanical equipment, flooring, cladding, gravity columns and walls. For the first model with only moment frames, the mass of the missing shear walls was also added onto the total mass. In addition, live load was added throughout the building. Since this is a residential tower, a 40 psf surface load was applied to all floors. The cladding load is applied along the perimeter of the building to give an accurate account of the effects of the cladding weight.

4.2.3.3 Analysis

Load cases for the 2006 IBC and Eurocode 8 were defined in two separate models. In ETABS the required Response Spectrum Functions were chosen depending on the applied code. Response spectrum cases were then added for each orthogonal direction, and for the combined motion. The desired response spectrum function was selected for each, and a preliminary scale factor was applied based on the unit of gravity and the response modification factor, R. Static load cases were then added, which included the seismic load in two orthogonal directions. The parameters determined in the codes were applied to the load case data sheet. The static load case can also be calculated by hand using the codes, and is a good way to verify that the building is modeled correctly.

The model with moment frames only was analyzed using a load combination with full dead load and 25% of the static earthquake load. This was the controlling analysis for the moment frames. This means that the complete design of the moment frames is based on this analysis. The full model of the building must be updated to make sure that the right properties of the moment frame columns are accounted for.

When the dynamic load case was run with the preliminary scaling factors, these factors had to be updated so that the dynamic base shear accounted for at least 85% of the static base shear (see Table 4.2). When the new factors had been determined, they replaced the preliminary ones in the response spectrum cases. This is the seismic load that is used in designing the building. This dynamic load was included in the load combinations along with the dead and live load.

When the analysis of the building is completed, the structural members can be designed as well. Generally this program is only used for analysis, and the moments and shear forces of each structural member is exported. The exported data is then enveloped using a spreadsheet, and the capacity needed for each member is determined based on the flexural and shear demands of each component.

Table 4.2: *Dynamic scaling in accordance with ASCE 7-05 Section 12.9.4*

R =		7.00
Scale factor =	*X:*	*55.14*
	Y:	55.14
V_{static}	X:	2090.25
	Y:	2090.25
$V_{dynamic}$	X:	617.08
	Y:	504.44
85% V_{static}	X:	1776.71
	Y:	1776.71
New Scale factor	X:	158.77
	Y:	194.22

4.2.3.4 Results

4.2.3.4.1 Static Analysis

Equivalent Lateral Force Procedure per ASCE 7-05 Section 12.8:

Building Period:	$T =$	2.1	s	{ASCE 7-05 Section 12.8.2}
Seismic Parameters:	$S_{DS} =$	0.699	g	{ASCE 7-05 Section 11.4}
	$S_{D1} =$	0.334	g	
Response modification factor:	$R =$	7		{ASCE 7-05 Table 12.2-1}
Importance factor:	$I_E =$	1		
Long-period transition period:	$T_L =$	16	s	{ASCE 7-05 Section 11.4.5}
Base Shear:	$V = C_S W$			
	$C_S =$	0.09986		
or	$C_S =$	0.02272		{Maximum}
or	$C_S =$	0.03076		{Minimum}
USE	$\mathbf{C_S =}$	**0.03076**		

$$\boxed{V = 0.03076 \text{ W}}$$

Vertical distribution of forces per ASCE 7-05 Section 12.8.3:

Building period =	2.1	s	k =	1.800	
Building weight =	67997	k	Base Shear, V =	0.0308	W
Base Shear, V =	2091.3085	k			

Level	Floor height [ft]	Elevation h_x [ft]	Floor wt. Mass	w_x [k]	$w_x h_x^k$	C_{vx}	F_x [k]	V_x [k]
31	---	311.25	5.5197	2131	65495600	0.090	187.92	
30	14.67	296.58	5.1818	2001	56368412	0.077	161.73	187.92
29	10.50	286.08	5.4781	2115	55847952	0.077	160.24	349.64
28	10.50	275.58	5.4781	2115	52212635	0.072	149.81	509.88
27	10.50	265.08	5.4639	2110	48560268	0.067	139.33	659.69
26	9.92	255.16	5.4643	2110	45341617	0.062	130.09	799.01
25	9.92	245.24	5.4999	2124	42493152	0.058	121.92	929.10
24	9.92	235.32	5.5167	2130	39569912	0.054	113.53	1051.02
23	9.92	225.40	5.5167	2130	36618134	0.050	105.06	1164.55
22	9.92	215.48	5.5167	2130	33768495	0.046	96.89	1269.62
21	9.92	205.56	5.5499	2143	31208612	0.043	89.54	1366.50
20	9.92	195.64	5.5981	2161	28798126	0.040	82.63	1456.05
19	9.92	185.72	5.6507	2182	26469618	0.036	75.95	1538.67
18	9.92	175.80	5.6507	2182	23979273	0.033	68.80	1614.62
17	9.92	165.88	5.6507	2182	21598883	0.030	61.97	1683.42
16	9.92	155.96	5.6507	2182	19329733	0.027	55.46	1745.39
15	9.92	146.04	5.6507	2182	17173204	0.024	49.27	1800.85
14	9.92	136.12	5.666	2188	15171757	0.021	43.53	1850.12
13	9.92	126.20	5.6866	2196	13287991	0.018	38.13	1893.65
12	9.92	116.28	5.6866	2196	11467309	0.016	32.90	1931.77
11	9.92	106.36	5.6866	2196	9766827	0.013	28.02	1964.67
10	9.92	96.44	5.7252	2210	8244293	0.011	23.65	1992.70
9	9.92	86.52	5.7637	2225	6826701	0.009	19.59	2016.35
8	9.92	76.60	5.7637	2225	5482933	0.008	15.73	2035.94
7	9.92	66.68	5.7791	2231	4283039	0.006	12.29	2051.67
6	9.92	56.76	5.7996	2239	3216430	0.004	9.23	2063.96
5	9.92	46.84	7.0204	2711	2755320	0.004	7.91	2073.19
4	9.92	36.92	7.399	2857	1892099	0.003	5.43	2081.09
3	9.92	27.00	7.8951	3048	1149509	0.002	3.30	2086.52

2	10.00	17.00	8.2028	3167	519362	0.001	1.49	2089.82
Ground	17.00	0.00						2091.31
Σ				67997	728897196.9		2091.31	

Lateral force method per Eurocode 8 Section 4.3.3.2:

Fundamental Period:	T_1 =	2.1	s
Peak ground acceleration:	a_{g40hz} =	0.2	g
Reference pga on type A ground:	a_{gR} =	0.16	g
Importance factor:	γ_I =	1	
Design pga on type A ground:	a_g =	0.16	g
Lower limit period of constant a:	T_B =	0.05	s
Upper limit period of constant a:	T_C =	0.25	s
Period defining constant disp.:	T_D =	1.2	s
Soil factor:	S =	1.35	
Behavior factor:	q =	5.4	
Design Response Spectrum:	S_d =	0.0320	g
Correction factor;	λ =	1.00	
Base shear :	F_b =	$S_d(T_1)$ m λ	

$$\boxed{\mathbf{F_b = 0.0320 \cdot m\,\lambda}}$$

Vertical distribution of forces per Eurocode 8 Section 4.3.3.2.3:

Building period = 2.1 s

Building weight = 67997 k   Base Shear, F_b = 0.032 m

Base Shear, F_b = 2175.897 k

Level	Floor height [ft]	Elevation z_i [ft]	Floor wt. Mass	m_i [k]	$z_i m_i$	C	F_i [k]	V_i [k]
31	---	311.25	5.5197	2131	663322	0.063	137.98	
30	14.67	296.58	5.1818	2001	593366	0.057	123.43	137.98
29	10.50	286.08	5.4781	2115	605086	0.058	125.86	261.41
28	10.50	275.58	5.4781	2115	582878	0.056	121.25	387.27
27	10.50	265.08	5.4639	2110	559216	0.053	116.32	508.51
26	9.92	255.16	5.4643	2110	538328	0.051	111.98	624.84
25	9.92	245.24	5.4999	2124	520770	0.050	108.33	736.82
24	9.92	235.32	5.5167	2130	501231	0.048	104.26	845.14

23	9.92	225.40	5.5167	2130	480102	0.046	99.87	949.40
22	9.92	215.48	5.5167	2130	458972	0.044	95.47	1049.27
21	9.92	205.56	5.5499	2143	440477	0.042	91.62	1144.74
20	9.92	195.64	5.5981	2161	422861	0.040	87.96	1236.37
19	9.92	185.72	5.6507	2182	405192	0.039	84.28	1324.33
18	9.92	175.80	5.6507	2182	383549	0.037	79.78	1408.61
17	9.92	165.88	5.6507	2182	361906	0.035	75.28	1488.39
16	9.92	155.96	5.6507	2182	340263	0.033	70.78	1563.67
15	9.92	146.04	5.6507	2182	318621	0.030	66.28	1634.45
14	9.92	136.12	5.666	2188	297782	0.028	61.94	1700.73
13	9.92	126.20	5.6866	2196	277084	0.026	57.64	1762.67
12	9.92	116.28	5.6866	2196	255304	0.024	53.11	1820.31
11	9.92	106.36	5.6866	2196	233524	0.022	48.58	1873.41
10	9.92	96.44	5.7252	2210	213181	0.020	44.34	1921.99
9	9.92	86.52	5.7637	2225	192539	0.018	40.05	1966.33
8	9.92	76.60	5.7637	2225	170463	0.016	35.46	2006.38
7	9.92	66.68	5.7791	2231	148784	0.014	30.95	2041.84
6	9.92	56.76	5.7996	2239	127098	0.012	26.44	2072.79
5	9.92	46.84	7.0204	2711	126963	0.012	26.41	2099.23
4	9.92	36.92	7.399	2857	105471	0.010	21.94	2125.64
3	9.92	27.00	7.8951	3048	82304	0.008	17.12	2147.58
2	10.00	17.00	8.2028	3167	53841	0.005	11.20	2164.70
Ground	17.00	0.00						2175.90
Σ				67997	10460477.7		2175.90	

4.2.3.4.2 Dynamic Analysis

Dynamic base shear and story forces using ASCE 7.05 and ETABS

Story	Height [ft]	V_X [k]	V_Y [k]	F_X [k]	F_Y [k]
31st	311.25	234.45	283.96	167.01	187.02
30th	296.58	401.46	470.98	137.35	138.96
29th	286.08	538.81	609.94	100.18	85.75
28th	275.58	638.99	695.69	66.38	43.46
27th	265.08	705.37	739.15	39.46	18.71
26th	255.16	744.83	757.86	18.22	8.63
25th	245.24	763.05	766.49	2.97	9.24

24th	235.32	766.02	775.73	-6.26	14.62
23rd	225.40	759.76	790.35	-9.76	19.49
22nd	215.48	750.00	809.84	-8.32	21.49
21st	205.56	741.68	831.33	-2.78	21.53
20th	195.64	738.90	852.86	5.29	21.03
19th	185.72	744.19	873.89	14.31	20.48
18th	175.80	758.50	894.37	22.93	20.28
17th	165.88	781.43	914.65	30.40	20.84
16th	155.96	811.83	935.49	36.34	22.82
15th	146.04	848.17	958.31	40.83	26.85
14th	136.12	889.00	985.16	44.01	32.49
13th	126.20	933.01	1017.65	45.85	37.36
12th	116.28	978.86	1055.01	46.88	39.40
11th	106.36	1025.74	1094.41	47.98	39.10
10th	96.44	1073.72	1133.51	49.81	39.62
9th	86.52	1123.53	1173.13	52.40	43.08
8th	76.60	1175.93	1216.21	56.56	49.60
7th	66.68	1232.49	1265.81	62.33	56.10
6th	56.76	1294.82	1321.91	85.97	75.72
5th	46.84	1380.79	1397.63	104.64	97.66
4th	36.92	1485.43	1495.29	127.46	123.90
3rd	27.00	1612.89	1619.19	146.54	125.68
2nd	17.00	1759.43	1744.87	19.44	32.14
Ground	0.00	1778.87	1777.01		

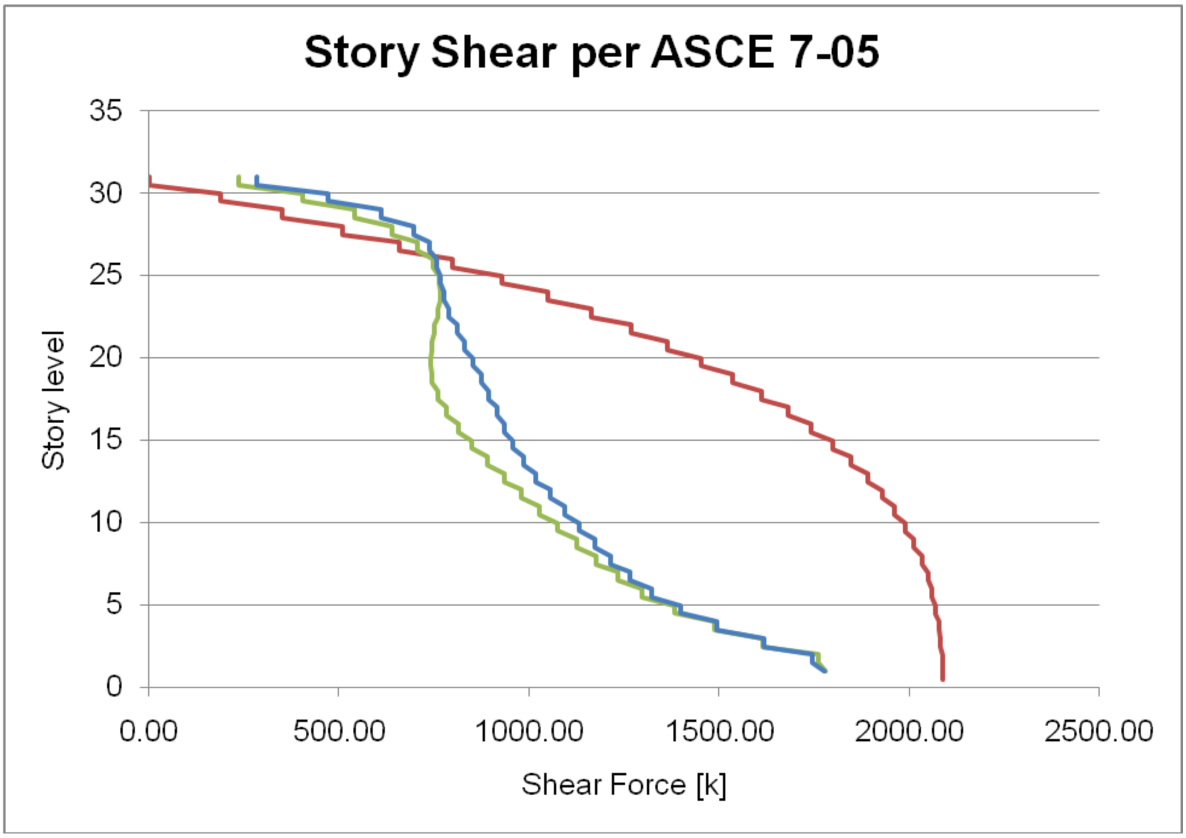


Fig. 4.13: *The red graph is the static story shear and the green and blue graphs are the dynamic story shear in the x and y direction, respectively.*

Dynamic base shear and story forces using Eurocode 8 and ETABS

Story	Height [ft]	V_X [k]	V_Y [k]	F_X [k]	F_Y [k]
31st	311.25	82.7	79.3	58.60	52.84
30th	296.58	141.3	132.14	47.70	39.63
29th	286.08	189	171.77	33.96	24.47
28th	275.58	222.96	196.24	21.09	11.59
27th	265.08	244.05	207.83	10.39	2.76
26th	255.16	254.44	210.59	1.38	-2.61
25th	245.24	255.82	207.98	-5.68	-4.95
24th	235.32	250.14	203.03	-10.63	-5.16
23rd	225.40	239.51	197.87	-13.28	-4.20
22nd	215.48	226.23	193.67	-13.66	-2.92
21st	205.56	212.57	190.75	-11.66	-1.45
20th	195.64	200.91	189.3	-7.60	0.17
19th	185.72	193.31	189.47	-2.07	1.71
18th	175.80	191.24	191.18	3.75	2.94

17th	165.88	194.99	194.12	8.90	3.89
16th	155.96	203.89	198.01	12.77	4.77
15th	146.04	216.66	202.78	15.34	5.93
14th	136.12	232	208.71	16.86	7.48
13th	126.20	248.86	216.19	17.60	9.04
12th	116.28	266.46	225.23	18.01	10.18
11th	106.36	284.47	235.41	18.62	11.21
10th	96.44	303.09	246.62	19.76	12.96
9th	86.52	322.85	259.58	21.38	15.77
8th	76.60	344.23	275.35	23.68	19.30
7th	66.68	367.91	294.65	26.54	22.44
6th	56.76	394.45	317.09	36.80	29.82
5th	46.84	431.25	346.91	44.36	35.81
4th	36.92	475.61	382.72	53.00	41.30
3rd	27.00	528.61	424.02	59.41	39.79
2nd	17.00	588.02	463.81	6.99	8.19
Ground	0.00	595.01	472		

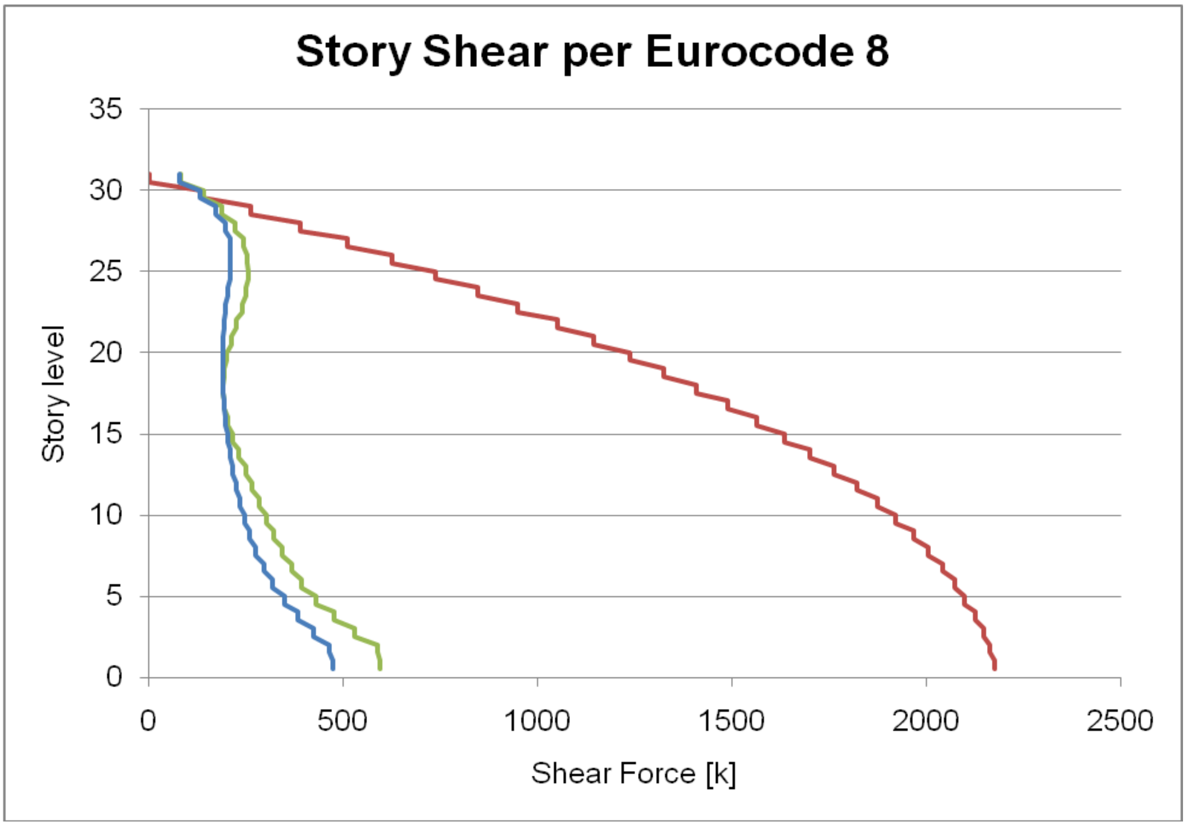


Fig. 4.14: *The red graph is the static story shear and the green and blue graphs are the dynamic story shear in the x and y direction, respectively.*

4.3 Evaluation of the Results

As the previous sections show, the linear results from the two codes in each case give very similar results. For the calculation of the static base shear, both codes revert back to the minimum value of seismic response coefficient / design spectrum value. This is therefore not a realistic value, but rather a minimum value set by the authorities. The design coefficients used in the ASCE 7-05 approach is limited, and the calculated base shear is mainly dependent on the mass of the building. This is also the case for the values in Eurocode 8. The design spectra for both buildings have a minimum value of 0.032 for building periods of 1.25 s and higher and 2 s and higher for the soil conditions of the 1st and Main building and the Ardea, respectively. Both these buildings have a higher period than the limiting values, and the static base shear is therefore 3.2% of the building mass for both structures. Because seismic forces are so uncertain, it is difficult to say whether or not this is overly conservative.

For the dynamic analysis, however, the IBC give very different results from those obtained using the Eurocode (see Figures 4.7, 4.8, 4.13, and 4.14). This is mainly because of the IBC requirement to use 85% of the static base shear as a minimum. As mentioned earlier, the Eurocode has no such requirement. Since the dynamic base shear for both these buildings was much less than the static, there is a large discrepancy between the design shear forces of the IBC and Eurocode. The dynamic analysis of the buildings performed using computer software is obviously much more accurate than the static analysis conducted using a simplified procedure. One can therefore argue that scaling the up the dynamic output according to ASCE 7-05 is unnecessary.

The difference in return period between the IBC and the Eurocode gives reason to expect different results. However, as discussed in Section 2.1.1.1, the difference in design values for Portland, OR for the different return periods is not significant. The results using both the IBC and Eurocode 8 therefore closely correspond.

4.4 Conclusion

Although the two codes have certain differences, it is clear that they are both based on a common understanding of earthquake behavior. The science behind the provisions are founded on common scientific ground, and even though the analysis approach differ in context, the results achieved closely correlate.

Figures 4.7, 4.8, 4.13, and 4.14 all show that the Eurocode static story shears for both buildings are slightly higher than that of the IBC. This means that if the buildings were to be designed based on the static story shears, the Eurocode would have provided a more seismically resistant structure. The same figures show that the dynamic story shears are much higher for the IBC analysis approach. Since buildings are designed for the dynamic story shears, the IBC would therefore provide a higher level of safety than the Eurocode. In other words, if the two buildings considered in these case studies would have been constructed based on Eurocode design, they would not have nearly the same lateral strength as with the IBC.

Considering that the United States is an authority on seismic design, where earthquake hazard has been part of the structural design criteria for a long time, it is obvious that most other countries look to the US provisions for examples of earthquake resisting design. This seems to be the case with the Eurocode as well. The US standards have changed significantly over the last few years. Therefore, it looks as though there may be a closer link between the previous US code, the 1997 Uniform Building Code, and Eurocode 8. The most significant change is the design earthquake changing from one with a 475 year return period to one with a 2500 year return period. As mentioned in Section 2.1.1.1, the 475 year return period is still used for Eurocode 8.

Seismic design of buildings is a very complex endeavor, and for experienced engineers the complete seismic design process for a high-rise building takes several months. The scope of these case studies has been limited accordingly. When comparing these two codes, the most interesting aspects are those concerning the seismic analysis.

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