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ETE Road Map

According to Chapter IV and V of the "Conclusions of the Melk Process and Follow-Up"

Seismic Design

Preliminary Monitoring Report

Report to the Federal Ministry of Agriculture, Forestry, Environment and Water Management of Austria

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EXECUTIVE SUMMARY

The Republic of Austriyhe Czech Republic have, using the good offices of Commissioner Verheugen, reached an accord on the “Conclusions of the Melk Process and Follow-up” on 29 November 2001. In order to enable an effective use of the “Melk Process” achievements in the area of nuclear safety, the ANNEX I of this “Brussels Agreement” contains details on specific actions to be taken as a follow-up to the “trialogue“ of the “Melk Process” in the framework of the pertinent Czech-Austrian Bilateral Agreement.

Furthermore, the Commission on the Assessment of Environmental Impact of the Temelín NPP – set up based on a resolution of the Government of the Czech Republic – presented a report and recommended in its Position the implementation of twenty-one concrete measures (ANNEX II of the “Brussels Agreement).

The signatories agreed that the implementation of these measures would also be regularly monitored jointly by Czech and Austrian experts within the Czech-Austrian Bilateral Agreement.

A “Roadmap” regarding the monitoring on the technical level in the framework of the pertinent Czech-Austrian Bilateral Agreement as foreseen in the “Brussels Agreement” has been elaborated and agreed by the Deputy Prime Minister and Minister of Foreign Affairs of the Czech Republic and the Minister of Agriculture and Forestry, Environment and Water Management of the Republic of Austria on 10 December 2001.

The Federal Ministry of Agriculture, Forestry, Environment and Water Management entrusted the Umweltbundesamt (Federal Environment Agency) with the general management of the implementation of the “Roadmap”. Each entry to the “Roadmap” corresponds to a specific technical project.

During the discussion within the Roadmap item 6 site seismicity the Czech side offered to report about the issue re-evaluation / seismic design during the bilateral meeting 2003. For the preparation of the Austrian delegation on this subject the project PN8 has been created. The subject is related to issue 7 (“Seismic Design and Seismic Hazard Assessment”) of the Melk process, whereas here the relation is limited to seismic design. Seismic hazard assessment is subject to project PN6.

VCE Holding GmbH of Vienna was committed by the Umweltbundesamt (Federal Environment Agency) on behalf of the Austrian Government to give technical support for the monitoring on the technical level of the implementation of the conclusions regarding the issue seismic design. This technical support will have to focus on the evaluation of the extent of conformity of the seismic design for NPP Temelin with state of the art practice in European Union member states and IAEA guidelines.

This specific technical project is referred to as project PN 8 comprising all together 7 predefined “project milestones” (PM).

To focus the preparatory work of the Austrian expert team and to guide the Austrian delegation through the specialist presentation in the 1st step (Project Milestone 1) the safety objective regarding seismic design was broken down to Verifiable Line Items (VLIs). After the presentation of the Czech side during the bilateral meeting in December 2003 in Vienna, the Austrian Expert team prepared a list of information, the Specific Information Request (SIR), considered to contain the necessary background required to provide for profound answers in the VLIs. The VLIs treat the subjects of legal framework, seismic design input, re-evaluation methodology, identification of critical structures, interfaces and components, and implementation of seismic upgrade measures.

Based on the recognition that the pertinent Czech-Austrian-Bilateral agreement is the appropriate framework giving the opportunity for further discussion and sharing additional information on these issues, it would be appreciated if the major findings could be resolved in a further monitoring process.

The Information provided by the Czech Side

The key information has been provided by the Czech side in the presentation named:

Temelin NPP: Seismic qualification of civil engineering structures, prepared by Mr. Maly of the nuclear research institute, REZ, division Energoproject Praha, which he presented during the bilateral meeting in Vienna on December 18th 2003.

The analysis of the information made available through the power point presentation: Temelin NPP, seismic qualification of civil engineering structures and the inquiries and discussions held with Stevenson and Associates and the Technical University of Praha are the basis for the present preliminary monitoring report of the Austrian expert team.

The Approach by the Austrian Expert Team

Based on the preliminary monitoring results of project PN6 (site seismicity) the Austrian expert team broke down the safety objective regarding seismic design into Verifiable Line Items (VLIs) in a 1st step. This defined the items of interest and the questions to be answered.

In a 2nd step, after the presentation of the Czech side, a list of information, the Specific Information Request (SIR), has been compiled considered to contain the necessary background required to provide for profound answers in the VLI's.

The 3rd step is the compilation of background information and materials necessary to assess the gathered information. It is represented in the preliminary monitoring report (PMR) as separate chapter and comprises the current practice in seismic design.

In meetings with the involved subcontractor of CEZ ETE, Stevenson and Associates, and the Technical University of Praha, Prof. Bidnar, additional information has been collected.

The current PMR is based on the process outlined above.

Preliminary Result of the Monitoring

The monitoring process so far clarified the VLIs. Based on the information available the expert team formulates its view on the status of the seismic design of the Temelin MPP as follows:

The seismic design practice has made considerable progress in the last years based on the experience made and the measurements taken from past events such as Northridge (1994, U.S.A.), Kobe (1995, Japan), Kozaeli (1999. Turkey), and ChiChi (1999, Taiwan). This resulted in considerable changes in the approach as well as related codes and standards. Probabilistic approach and performance based design philosophies are prevailing, analysing the global behaviour of the technical complex systems.

Traditional approaches as represented by national codes and also some of the valid IAEA guidelines do not satisfy the requirements of a realistic assessment of the seismic capacity of structures. New guidelines reflecting the current practice are in the drafting process and are expected to come into force soon.

In Temelin seismic design has been limited to structures, neglecting such important seismic effects as:

- Interaction between adjacent structures
- Differential local movements at vital interfaces
- The performance and eventual collapse of non structural components

There is a clear consensus among the Austria expert team that the information and material provided by the Czech side during the bilateral meeting on December 18th 2003 was very informative and conclusive. The Czech experts demonstrated that they made efforts to fulfil the requirements specified in the IAEA guidelines concerning seismic design.

From the Austrian point of view the seismic design re-evaluation conforms to the existing standards and recommendation which on the other hand do not consider the current best engineering practice. The question of interfaces and non structural components has not been addressed in the re-evaluation process based on the lack of formal requirements.

Recommendations

In order to improve the knowledge on the seismic performance and the eventual identification of necessary retrofit measures the Austrian expert team recommends to the Austrian government to propose to the Czech side the following:

- To perform a probabilistic safety analysis (PSA) on the level of the recommendation of IAEA and the current practice in Western Europe.
- To open the chapter of seismic qualification of civil structures, interfaces and components again to be incorporated into the 10 year periodic safety review.
- To actively improve the monitoring system and enhance the use of actual data in the evaluation process including an improvement of the existing database

ZUSAMMENFASSUNG

Die Republik Österreich und die Tschechische Republik haben mit der Unterstützung des Mitgliedes der Kommission Verheugen am 29. November 2001 eine Übereinstimmung über die „Schlussfolgerungen des Melker Prozesses und seine Fortsetzung erzielt. Um eine wirksame Umsetzung der Ergebnisse des Melker Prozesses im Bereich der nuklearen Sicherheit zu ermöglichen, enthält der Anhang I dieses „Brüsseler Abkommens“ Details zu spezifischen Maßnahmen die als Weiterführung des „Trialogs“ des Melker Prozesses im Rahmen des betreffenden bilateralen tschechisch-österreichischen Abkommens durchzuführen sind.

Weiters legte die Kommission zur Prüfung der Umweltverträglichkeit des KKWs Temelin, die auf Grund einer Resolution der Regierung der Tschechischen Republik eingesetzt wurde, einen Bericht vor und schlug in ihrer Stellungnahme die Umsetzung einundzwanzig konkreter Maßnahmen vor (Anhang II des „Brüsseler Abkommens“).

Die Unterzeichner kommen überein, dass die Umsetzung der genannten Maßnahmen von tschechischen und österreichischen Experten regelmäßig und gemeinsam im Rahmen des bilateralen Abkommens über den Austausch von Informationen überwacht wird.

Zur Überwachung auf technischer Ebene im Rahmen des diesbezüglichen tschechisch-österreichischen bilateralen Abkommens wurde, wie im „Brüsseler Abkommen“ vorgesehen, ein „Fahrplan“ („Roadmap“) ausgearbeitet und am 10. Dezember 2001 vom stellvertretenden Premierminister und Außenminister der Tschechischen Republik sowie vom Bundesminister für Land- und Forstwirtschaft, Umwelt und Wasserwirtschaft der Republik Österreich vereinbart.

Das österreichische Bundesministerium für Land- und Forstwirtschaft, Umwelt und Wasserwirtschaft beauftragte das Umweltbundesamt (Federal Environment Agency) mit der Gesamtkoordination der Umsetzung dieses „Fahrplans“. Jeder Eintrag im „Fahrplan“ entspricht einem spezifischen technischen Projekt.

Im Zuge der Diskussion innerhalb des Roadmap Item 6 Site Seismicity wurde von tschechischer Seite angeboten, gesondert über das Thema Re-Evaluierung / Seismic Design im Rahmen des bilateralen Treffens 2003 zu berichten. Für die Vorbereitung der österreichischen Delegation auf dieses Thema entstand das Projekt PN8. Inhaltlich knüpft das Thema an den Issue 7 („Seismic Design and Seismic Hazard Assessment“) des Melker Prozesses an, wobei hier der ausschließliche Bezug auf Seismic Design besteht. Seismic Hazard Assessment (Seismische Gefährdungseinschätzung) wird im Rahmen des Projektes PN6 behandelt.

VCE Holding GmbH wurde im Namen der Österreichischen Bundesregierung vom Umweltbundesamt (Federal Environment Agency) beauftragt, technische Unterstützung für den Monitoringprozess auf technischer Ebene zur Umsetzung der Schlussfolgerungen für das Thema seismische Auslegung zu geben. Diese technische Unterstützung wird sich auf die Bewertung des Ausmaßes der Übereinstimmung der seismischen Auslegung des Kernkraftwerks Temelin mit der derzeit in den Mitgliedstaaten der Europäischen Union geübten Praxis und den Richtlinien der IAEO konzentrieren müssen.

Dieses spezielle technische Vorhaben wird als Projekt PN8 bezeichnet und enthält insgesamt sieben vorgegebene „Projektmeilensteine“ (PM).

Um die Vorbereitungsarbeit der österreichischen Expertengruppe zu bündeln und die österreichische Delegation durch das Arbeitstreffen der Experten zu leiten, wurde das Sicherheitsziel betreffend seismische Auslegung, in einem ersten Arbeitsschritt (Projekt Milestone 1) zu nachprüfbaren Programmpunkten (VLI) aufgespaltet. In einem zweiten Arbeitsschritt bereitete die Expertengruppe eine Liste von Dokumenten vor, den sogenannten „Specific Information Request – SIR“, die – nach Meinung des Expertenteams – die notwendige Information enthält, um eine fundierte Beantwortung der VLIs zu ermöglichen. Zusammenfassend lässt sich sagen, dass die VLIs die Verfahren der Abschätzung der seismischen Kapazität, rechtliche Aspekte, Eingangswerte der Anregung, Re-Evaluierungsverfahren, die Identifizierung von

kritischen Bauwerken, Übergängen und Komponenten und die Implementierung von seismischen Aufrüstungsmassnahmen behandeln.

Im Bewusstsein, dass das einschlägige Tschechisch-Österreichische Bilaterale Nuklearinformationsabkommen einen geeigneten Rahmen für weitere Diskussion und einen zusätzlichen Informationsaustausch darstellt, wäre es wünschenswert, wenn die wesentlichen Ergebnisse dieses Berichtes im Verlauf eines weiteren Monitoringprozesses behandelt werden könnten.

Informationen der Tschechischen Seite

Den Schwerpunkt im Verlaufe des Monitoringprozesses stellte folgende Präsentation dar:

Temelin NPP: „Seismische Qualifizierung von Ingenieurbauwerken“, bearbeitet von Herrn Maly des Nuclear Research Institute, REZ, Abteilung Energoproject Prag, welche er im Zuge des bilateralen Treffens in Wien am 18. Dezember 2003 vorstellte.

Die Analyse der Information aus der Power Point Präsentation: „Temelin NPP, die seismische Qualifizierung von Ingenieurbauwerken (in Englisch)“, sowie die Prüfung und die Diskussion, geführt mit Stevenson & Associates und der Technischen Universität von Prag, dienen als Grundlage für den vorliegenden vorläufigen Überprüfungsbericht (Preliminary Monitoring Report, PMR) des österreichischen Expertenteams.

Der Ansatz des Österreichischen Expertenteams

Basierend auf den vorläufigen Ergebnissen aus Projekt PN6 (Standort Seismizität) gliederte das österreichische Expertenteam die Sicherheitsüberlegungen bzgl. seismischer Auslegung in nachprüfbare Programmpunkte (VLI). Dadurch wurden jene Bereiche definiert, zu welchen Antworten erwartet werden.

In einem zweiten Arbeitsschritt – nach der Präsentation der tschechischen Seite – bereitete die Expertengruppe eine Liste von Dokumenten vor, den sogenannten „Specific Information Request – SIR“, die – nach Meinung des Expertenteams – die notwendige Information enthält, um eine fundierte Beantwortung der VLIs zu ermöglichen.

Der dritte Schritt stellt eine Sammlung der Hintergrundinformationen und des zur Bewertung der erhaltenen Information notwendigen Materials dar. Diese ist in dem vorliegenden Bericht (Preliminary Monitoring Report) als separates Kapitel dargestellt und beinhaltet die derzeitige Praxis der seismischen Auslegung.

Zusätzliche Informationen wurden in Treffen mit den beteiligten Auftragnehmern, Stevenson & Associates, sowie der Technische Universität von Prag, Prof. Bidnar, gesammelt.

Der vorliegende Bericht (PMR) baut auf diesem Ansatz auf.

Bisheriges Ergebnis des Monitoringprozesses

Bisher half der Monitoringprozess bei der Abklärung einer Reihe von nachprüfbaren Programmpunkten (VLIs). Basierend auf der verfügbaren Information formuliert die Expertengruppe ihre Sichtweise über den Stand der seismischen Kapazität des KKW Temelin wie folgt:

Die Praxis der seismischen Auslegung hat im Laufe der vergangenen Jahre beachtliche Fortschritte auf Grund der gemachten Erfahrungen und der Messungen, welche in vergangenen Ereignissen gemacht wurden (z.B.: Northridge (1994, U.S.A.), Kobe (1995, Japan), Kozaeli (1999, Türkei) und ChiChi (1999, Taiwan), erlebt. Dies führte zu erheblichen Änderungen in der Betrachtungsweise sowie bei den entsprechenden Richtlinien und Standards. Ein probabilistischer Ansatz und die Philosophien des verhaltensbasierten Entwerfens, um das globale Verhalten von technisch komplexen Systemen zu analysieren, sind Stand der Technik.

Traditionelle Ansätze, wie sie in den nationalen Normen und auch einigen derzeit gültigen IAEA Richtlinien bestehen, erfüllen nicht die Anforderungen einer sachlichen Abschätzung der seismischen Aufnahmefähigkeit von Bauwerken. Neue Richtlinien sind in Kürze zu erwarten.

Die seismische Bemessung ist beschränkt auf Baukörper. Folgende wichtige seismische Einwirkungen wurden vernachlässigt:

- Beeinflussung zwischen benachbarten Bauwerken
- Unterschiedliche lokale Bewegungen an wesentlichen Schnittstellen
- Das Verhalten und letztlich das Versagen von nicht tragenden Komponenten

Es besteht eine Übereinstimmung innerhalb des österreichischen Expertenteams, dass die Informationen und das von der tschechischen Seite während des bilateralen Treffens am 18. Dezember 2003 zur Verfügung gestellte Material sehr informativ und schlüssig war. Die tschechischen Experten zeigten sich bemüht die Anforderungen der IAEA betreffend der seismischen Auslegung zu erfüllen.

Vom österreichischen Standpunkt aus entspricht die Neubewertung der seismischen Auslegung den derzeit gültigen Standards und Empfehlungen. Diese entsprechen, auf der anderen Seite, nicht dem Stand der Technik und der in Westeuropa geübten Praxis. Die Fragestellung des Verhaltens und dem letztendlichen Versagen von nicht tragenden Komponenten wurde im Verlauf der Neubewertung, aufgrund von Mangel an Formvorschriften, nicht behandelt.

Empfehlungen

Um das Wissen über das Verhalten der Anlage während eines Erdbebens zu verbessern und um eventuell notwendige Aufrüstungsmaßnahmen zu identifizieren, empfiehlt das österreichische Expertenteam der österreichischen Regierung folgende Empfehlungen an die tschechische Seite zu richten:

- Durchführung einer probabilistischen Sicherheitsanalyse (PSA) im Umfang der Empfehlungen der IAEO und der gängigen Praxis in Westeuropa.
- Die Neubehandlung des Kapitels Seismische Qualifizierung der Bauwerke, Übergänge und Komponenten im Zuge des fälligen periodischen Sicherheitsberichtes.
- Die aktive Verbesserung des Überwachungssystems und die Einbeziehung aktueller Messwerte in den Evaluierungsprozess inkl. der Verbesserung der existierenden Datenbank.

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1 INTRODUCTION

The Republic of Austria and the Czech Republic have, using the good offices of Commissioner Verheugen, reached an accord on the “Conclusions of the Melk Process and Follow-up” on 29 November 2001. In order to enable an effective use of the “Melk Process” achievements in the area of nuclear safety, the ANNEX I of this “Brussels Agreement” contains details on specific actions to be taken as a follow-up to the “trialogue“ of the “Melk Process” in the framework of the pertinent Czech-Austrian Bilateral Agreement.

To enable an effective ”trialogue” follow-up in the framework of the pertinent Czech-Austrian Bilateral Agreement, a seven-item structure given in ANNEX I of the “Brussels Agreement” has been adopted. Individual items are linked to:

- Specific objectives set in licensing case for NPP Temelin units;
- Description of present status and future actions foreseen by the licensee and SÚJB respectively.

Each item under discussion will be pursued according to the work plan agreed at the Annual Meeting organised under the pertinent Czech-Austrian Bilateral Agreement.

Furthermore, the Commission on the Assessment of Environmental Impact of the Temelín NPP – set up based on a resolution of the Government of the Czech Republic – presented a report and recommended in its Position the implementation of twenty-one concrete measures (Annex II of the “Brussels Agreement).

The signatories agreed that the implementation of the said measures would also be regularly monitored jointly by Czech and Austrian experts within the Czech-Austrian Bilateral Agreement.

A “Roadmap” regarding the monitoring on the technical level in the framework of the pertinent Czech-Austrian Bilateral Agreement as foreseen in the “Brussels Agreement” has been elaborated and agreed by the Deputy Prime Minister and Minister of Foreign Affairs of the Czech Republic and the Minister of Agriculture and Forestry, Environment and Water Management of the Republic of Austria on 10 December 2001.

This „Roadmap“ is based on the following principles:

- The implementation of activities enumerated in ANNEX I and II of the “Brussels Agreement” will be continued to ensure that comprehensive material is available for the monitoring activities set out below.
- Having in mind the peer review procedure foreseen by the EU to monitor the implementation of the recommendations of the AQG/WPNS Report on Nuclear Safety in the Context of Enlargement, the Czech and Austrian sides agree that this peer review should serve as another important tool to handle remaining nuclear safety issues.
- As a general rule the regular annual meetings according to Art. 7(1) of the bilateral Agreement between the Government of Austria and the Government of the Czech Republic on Issues of Common Interest in the Field of Nuclear Safety and Radiation Protection will serve to monitor the implementation of those measures referred to in Chapter V of the Conclusions and to address questions regarding nuclear safety in general, in particular those issues which – according to Chapter IV of the Conclusions – have been found, due to the nature of the respective topics, suitable to be followed-up in the framework of this Bilateral Agreement.
- In addition, specialists’ workshops and topical meetings will take place, organised as additional meetings according to Art. 7(4) of the bilateral Agreement between the Government of Austria and the Government of the Czech Republic on Issues of Common Interest in the Field of Nuclear Safety and Radiation Protection, as set out in the “Roadmap”.

The Federal Ministry of Agriculture, Forestry, Environment and Water Management entrusted the Umweltbundesamt (Federal Environment Agency) with the general management of the implementation of the "Roadmap". Each entry to the "Roadmap" corresponds to a specific technical project.

During the discussion within the Roadmap item 6 site seismicity the Czech side offered to report about the issue re-evaluation / seismic design during the bilateral meeting 2003. For the preparation of the Austrian delegation on this subject the project PN8 has been created. The subject is related to issue 7 ("Seismic Design and Seismic Hazard Assessment") of the Melk process, whereas here the relation is limited to seismic design. Seismic hazard assessment is subject to project PN6.

VCE Holding GmbH of Vienna was committed by the Umweltbundesamt (Federal Environment Agency) on behalf of the Austrian Government to give technical support for the monitoring on the technical level of the implementation of the conclusions regarding the issue seismic design. This technical support will have to focus on the evaluation of the extent of conformity of the seismic design for NPP Temelin with state of the art practice in European Union member states and IAEA guidelines.

This specific technical project is referred to as project PN 8 (refer to ANNEX I) comprising all together 7 predefined "project milestones" (PM).

To focus the preparatory work of the Austrian expert team and to guide the Austrian delegation through the specialist presentation in the 1st step (Project Milestone 1) the safety objective regarding seismic design was broken down to Verifiable Line Items (VLIs). After the presentation of the Czech side during the bilateral meeting in December 2003 in Vienna, the Austrian Expert team prepared a list of information, the Specific Information Request (SIR), considered to contain the necessary background required to provide for profound answers to the VLIs. The VLIs treat the subjects of legal framework, seismic design input, re-evaluation methodology, identification of critical structures interfaces and components, and implementation of seismic upgrade measures.

Based on the recognition that the pertinent Czech-Austrian-Bilateral agreement is the appropriate framework giving the opportunity for further discussion and sharing additional information on these issues, it would be appreciated if the major findings could be resolved in the further monitoring process.

1.1 Objective of this Report

The objective of this report is to present the evaluation of the seismic design for the Temelin NPP based on the information available to the Austrian side. In particular, the aim of the report is to clarify and establish the issues which were resolved and to point out issues which are still pending. This will create the basis for further monitoring within bilateral Czech-Austrian activities.

In particular this report shall provide a technical background for the assessment of the seismic design of the NPP Temelin. Four major tasks are identified to comply with these objectives:

- Identification of major unresolved safety issues and of areas of future improvements/-clarification
- Overview of adopted approaches in comparable member states for seismic design
- Preparation of a synthesis on the most updated engineering practice for a common understanding for major safety issues connected with seismic design.

1.2 Report Structure

This report provides under chapter 2 a simplified guideline through the requirements for earthquake resistant constructions. This should give a kind of preview how seismic analyses of constructions are undertaken following the current practice in Western Europe and the United States. The essential elements of the Eurocode 8, comprising the valid standard for seismic design in Europe, are shown as well as the identification of the principal current areas of concern to construction professionals, owners and regulators.

Under chapter 3 the information received during the presentations of the bilateral meeting in December 2003 is provided representing the approach of the Czech side including the assessment of the Austrian Expert Team.

Under chapter 4 the seismic design is evaluated along the Verifiable Line Items defined for this process.

Recommendations to the Austrian government to propose to the Czech side in order to improve the knowledge on the seismic performance and the eventual identification of necessary retrofit measures are provided under chapter 5.

Supplementary information is provided in the annexes.

2 SEISMIC DESIGN – A SIMPLIFIED GUIDELINE THROUGH THE REQUIREMENTS FOR EARTHQUAKE RESISTANT CONSTRUCTIONS

2.1 Prelude

This chapter provides a simplified guideline through the requirements for earthquake resistant constructions. This should give a kind of preview how seismic analyses of constructions are undertaken following the current practice in Western Europe and the United States. The essential elements of the Eurocode 8, comprising the valid standard for seismic design in Europe, are shown as well as the identification of the principal current areas of concern to construction professionals, owners and regulators.

2.1.1 Motivation

The threat of earthquakes (vibrations of the earth's crust) on structures is a consequence of subterranean ground faults, which can lead to an occurrence all over the world, even if major earthquakes occur most frequently in particular areas that are called zones of high probability. Site seismicity has been discussed in detail in PN6.

Normally most of our buildings are routinely designed for vertical gravity loads. The movements of the ground surface in all directions with their most damaging effects on structures in the direction parallel to the ground surface (horizontally) demand a completely different concept considering horizontal force-resistance (Figure 2.1). This effect might be compared to a similar one induced by the wind.

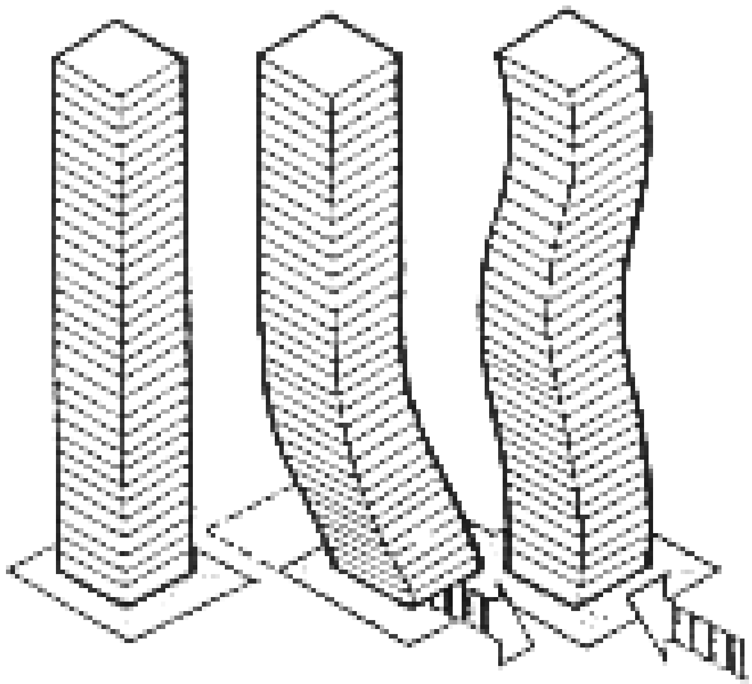


Figure 2.1: Typical seismic excitation to a building structure [4]

Usually major earthquakes are rather short in duration, they often last only a few seconds and seldom more than a minute. The maximum intensity of the earthquake – represented by one or more major peaks of magnitude of motion – is measured as the energy at the location of the ground fault.

Ground movements caused by earthquakes have several types of damaging effects. Defining support motions as a consequence of earthquake excitations of structures is the most difficult and uncertain phase of predicting structural responses to earthquakes. This chapter deals with the *Direct Movement of Structures* caused by their attachment to the ground. This effect forces us to guarantee a certain level of dynamic stability (general resistance to shaking) and some quantified resistance to energy loading for the affected structures.

*Figure 2.2: Damages due to the **Chi-Chi earthquake**, Taiwan, September 20, 1999, Magnitude 7.6 [5]*

The pictures above bring to mind the need and the responsibility of structural engineers for the design of new structures and the redesign and upgrade of existing structures (which have been built on the base of older standards) to minimize the damage caused by earthquakes to save lives. Even if it is desirable – in certain cases – to carry out a stochastic seismic analysis by describing structural response in probabilistic terms, the following rules refer to deterministic methods, which provide valuable insights into the seismic behavior. They represent the current practice in Seismic Design.

2.1.2 Approach

This section should give an example, how seismic analyses of constructions are undertaken. The essential elements of the Eurocode 8 (which is in a state of continuous evolution in research and changes in construction practice) are to be shown as well as the identification of the principal current areas of concern to construction professionals, owners and regulators. As this work is a guideline, the author desists from discussing the EC 8 with all its exceptions and details. This chapter should be a useful help to get exposed to earthquake-engineering and a motivation to busy oneself with that topic. This essay will focus on the main features affecting buildings, even if the Eurocode 8 standardizes provisions for almost every common type of structure (Part 1-General rules & buildings; Part 2-bridges; Part 3-seismic strengthening & repair of existing buildings; Part 4-tanks, silos and pipelines; Part 5-foundations & geotechnical aspects and Part 6- special provisions relevant to towers, masts and chimneys).

Typical seismic design codes are based on the following multi-level philosophy of the structure's resistance visualized in Table 2.1:

Earthquake	Basic Requirement for Design	Compliance Criterion for Design	Method of analysis
Minor	to prevent damage		
Moderate	No damage to structural elements	Analysis of “Damage limitation states”	dealing with linear-elastic behavior
Major	Prevention of collaps – retaining the structure's integrity	Analysis of “Ultimate limit states”	dealing with nonlinear-elastic or idealized elastic behavior

Table 2.1: The multi-level philosophy of seismic design codes

During an analysis of the consequences due to moderate earthquakes a reliable calculation will be made under the premise of linear-elastic behavior of the structure. Even if major earthquake-analyses would require nonlinear behavior, which is much closer to reality (thinking of post-cracking or post-yielding properties of certain materials), these physical laws are replaced by intended linear-elastic laws, which have to be modified. Nevertheless the engineers have to ensure with the help of certain design features, that their constructions are able to tolerate these nonlinear, plastic deformations.

2.2 Main features for design of constructions

2.2.1 General

The EN 1998 (Eurocode 8) must be observed for the design of structures in seismic regions (buildings, bridges, etc.). It contains additional provisions to all other relevant Eurocodes. Its purpose is to ensure, that in the event of earthquakes

- human lives are protected,
- damage is limited,
- structures important for civil protection remain operational.

Special structures with increased risks for the population, such as nuclear power plants and large dams, are beyond the scope of EN 1998 [1]. The demands of these facilities have to be analyzed and solved under specialized, explicitly developed provisions on top and above the valid standards for structures.

2.2.2 Fundamental requirements

2.2.2.1 Introduction

In seismic regions structures shall be designed and constructed in a way, that the following requirements are met, each with an adequate degree of reliability:

- **No-collapse requirement (Ultimate limit states)**

Those states are associated with collapse or with other forms of structural failure which may endanger the safety of people [1]. The structure has to resist the design seismic action without global (the whole structure) or local (certain parts of the structure) collapse, which may endanger the safety of people, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. The design seismic action has a recurrence interval of 475 years (reference return period $\mathbf{T_{NCR}}$) corresponding to a 10% probability of exceedance in 50 years, which is commonly accepted to be the expected life of a building.

The IAEA safety guide 50-SG-S1 of 1992 [22] provides a return period of 10.000 years for the save shut down earthquake (SSE). Current practice is to consider return periods of up to 100.000 years [48].

- **Damage limitation requirement (Serviceability limit state)**

Those states are associated with damage occurrence, corresponding to states beyond which specified service requirements are no longer met [1]. The structure has to resist a seismic action having a larger probability of occurrence than the design seismic action without damage and the associated limitations of use. The design seismic action has a recurrence interval of 95 years (reference return period $\mathbf{T_{DLR}}$) corresponding to a 10% probability of exceedance in 10 years. By this way states with no longer met specified service requirements are avoided.

For the service ability case SL1 (OBE) a return period of 100 years has been selected.

In order to classify structures into different importance classes, individual importance factors γ_I have been determined. Each of these factors should be derived to correspond to a higher or lower value of the period of the seismic event (with regard to the reference return period). By this way certain levels of reliability are obtained by multiplying the reference seismic action- or the corresponding action effects- with this importance factor.

There are **certain basic parameters** to be concerned for seismic design procedures leading to a modification of the origin, gravity-dominated structural concept:

- A numeric value representing the seismic zone
- Code specified ground motion estimates and their consequence to the structure's response
- The mass of the structure including an assessment of live load
- Structural Concept & Irregularity
- A factor dependent on the building type and its ductility level
- A factor representing the importance of the structure
- Soil-structure interaction (number of independently moving parts)
- Critical regions and details demanding a great deal of attention

Even if the application of these parameters will be described during the following sections, their main features and principles should be elucidated.

2.2.2.2 Seismic zones

The key-note of the EN 1998 is the subdivision of national territories by National Authorities into seismic zones. The groundwork for these zones is permanently updated with data of new earthquakes and more precise data of old ones. This can lead to modified standards or to substitutions of their concepts under certain circumstances. These zones are assumed to be threatened by the same, individual hazard. This hazard is described with a single parameter **k· a_{gR}.** In respect of the five return period **T_{NCR}** defined in 2.2.2.1. **k** is a modification factor to account for special regional situations and may be found in the individual National Annex of the EC 8. For return periods unequal to the reference one the equation (2.1) shows, how the design ground acceleration on type A ground **a_g** is being derived.

$$a_g = \gamma_I \cdot k \cdot a_{gR} \tag{2.1}$$

γ_I.................. represents the importance factor and will be precised in Section 2.3.1.9.

2.2.2.3 The structure's ductility

A ductile structure has the ability to tolerate nonlinear, plastic deformations. It can be determined by values representing the ratio between a displacement at the state of failure and the state when plasticizing starts. A remarkable ductility leads to the advantage of carrying bigger loads than the one at the state of plastification for a short time. Another fundamental advantage is the energetic aspect in the case of cyclic loading, as it happens during earthquakes. Due to ground shaking energy is being induced to the structure the whole time, which is conserved as vibrational energy inside the structure and dissipated by damping and other processes. Figure 2.3 shows the difference between a structure, whose ductility results in an almost constant load absorption under cyclic excitation with still rising displacements and another one under destruction. We have to think about the fact, that major earthquakes induce much more energy than could be damped, which results in local collapses. The engineer's principle task during the analysis and design of facilities is to regulate, spread and convert the mechanical energy in order to prevent damage. In such cases the dissipation of mechanical energy by cyclic plastic deformation is of particular importance. Places, where such dissipations take place are those, where the most adverse combination of action effects occurs (**critical regions** or **dissipative zones**). That is the reason, why so-called **plastic hinges** may form. The attitude of plastic hinges can be swayed to prevent unfavorable collapse mechanisms. Figure 2.10 (left) shows a typical unpleasant collapse mechanism, the so-called **Soft Story**.

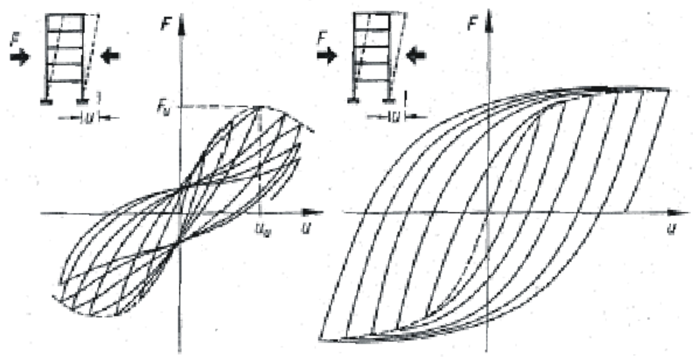


Figure 2.3: Unfavorable and favorable nonlinear behavior of a structure affected by cyclic excitation [15]

2.2.2.4 The "elastic response spectrum" as the most common representative of the seismic action

One of the most important relationships is the one which occurs between the period **T** of the structure (which is defined as the duration of one cycle of motion) and its response to an earthquake, which is typically defined as the maximum absolute value **S(T)** of displacement or acceleration over the entire earthquake history of the earthquake response integral [13]. These so-called response spectra curves are derived from a large number of earthquake "playbacks" on structures with different periods and are available for different seismic zones. Such an elastic ground acceleration response spectrum represents the **horizontal earthquake motion** at a given point of the surface for the two orthogonal directions. The same shape of the elastic response spectrum is taken for both levels of seismic action – the no-collapse requirement and the damage limitation requirement.

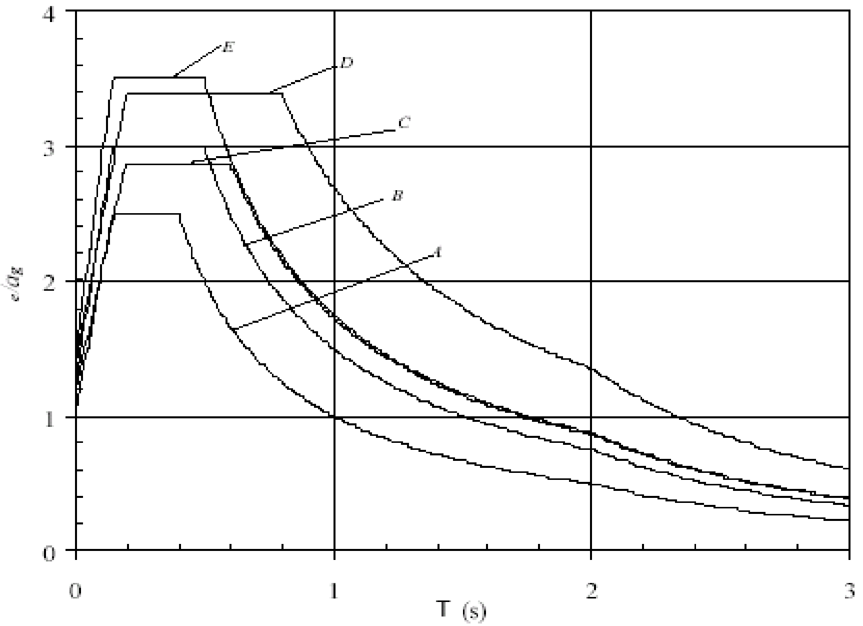


Figure 2.4: Recommended Type 1 elastic response spectrum for ground type A to E (5 % damping) [1]

Figure 2.4 shows, how ground type-dependent elastic response spectra **S_e(T)**, defined individually in every National Annex, could look like. The period of vibration **T** is defined as the time required for a vibrating structure to complete one cycle of motion. The detailed equations belonging to that picture can be taken from EN 1998-1 and are listed in section 2.6.4.1. They include a Soil factor **S**, the design ground acceleration on type A ground **a_g** correction factor η (for damping values unequal to 5%) as well and ground-type dependent special parameters **T_B**, **T_C** and **T_D** to give the curve its characteristically run. The distinction between Type 1 and Type 2 spectra is related with the value of the surface wave magnitude **M** (The size of the earthquake, a measure of how large the shake was and which is measured on a scale known as the Richter Scale).

As the response of a certain structure depends on its damping ability, this lowering of the magnitude effect has to be considered.

Design spectrum for elastic analysis:

According to Table 2.1 the provisions of the EC 8 avoid explicit inelastic structural analysis in design. The capability of the structure to dissipate energy, through mainly ductile behavior of its elements, is taken into account by performing an elastic analysis based on a so-called "design spectrum" which is obtained by reduction of the elastic one with the behavior factor **q**. The behavior factor (**q** = 1 is equal to linear-elastic behavior) is determined in tables for various materials and structural systems according to the relevant ductility classes in EN 1998. The value of the behavior factor may vary in different horizontal directions of the structure, although the ductility classification must be the same in all directions.

The **vertical earthquake motion** shall be also represented by an elastic response spectrum $\mathbf{S_{Ve}(T)}$. The derived equations can be taken again from EN 1998-1. They are quite similar to those for horizontal earthquake motion, depending on the same parameters as before (with the soil factor **S** taken equal to 1,0) but replacing the design ground acceleration $\mathbf{a_g}$ with the vertical design ground acceleration $\mathbf{a_{vg}}$.

2.2.3 Specific measures

2.2.3.1 Design

- Appropriate concepts demand the emphasis of regularity in plan and in the vertical distribution of mass and stiffness. If necessary this may be realized by subdivision of the structure by joints into dynamically independent units. It must be pointed out, that the force effect caused by motion is generally directly proportional to the gravity loads borne by the structure. The **inertial effects of the design seismic action** have to be derived with the following combination of actions:

$$\sum G_{k,j} + \sum \Psi_{E,i} \cdot Q_{k,i} \tag{2.2}$$

$\mathbf{G_{k,j}}$ represents the deadweight of the structure, $\mathbf{Q_{k,i}}$ represents the structure's live load. The combination coefficients $\psi_{E,i}$ ($=\varphi \cdot \psi_{2,i}$) for variable action **i** take into account the fact, that the loads $\psi_{2,i} \cdot \mathbf{Q_{k,i}}$ are not present over the entire structure during the occurrence of the earthquake. Furthermore the reduced participation of masses in the motion of the structure due to non-rigid connections between them can be taken into account. Values of □2,i are given in EN 1990 and values of φ for calculating $\psi_{E,i}$ may be found in every National Annex.

The complete **design seismic action**, represented with the value $\mathbf{E_d}$, shall be derived from the following combination of actions:

$$\boldsymbol{E}_d = \boldsymbol{G}_k \text{"+"} \boldsymbol{P}_k \text{"+"} \boldsymbol{A}_{ed} \text{"+"} \psi_{21} \cdot \boldsymbol{Q}_{1k} \text{"+"} Q_2 \tag{2.3}$$

"+" .... implies "to be combined with".

G_k .... represents the permanent loads with their characteristic values.

P_k .... represents the characteristic value of prestressing after all losses.

A_{ed}................ is the most unfavourable combination of the components of the earthquake action due to the elastic response spectrum. The equivalent equations to determine that representative are eq. 2-13; 2-14; 2-16 and 2-17 replacing E_{ed} with A_{ed}

Q_{1k} .... represents the characteristic value of the traffic load.

ψ_{21} .... *is the combination factor.*

Q_2................. *is the quasi permanent value of actions of long duration (earth pressure, etc.).*

- We should envision that an earthquake shakes the whole building leading to a so-called **global response** (Figure 2.1). As the building might remain completely intact, the potential movement of all its parts has to be ensured. The failure of any part of the system or of connections between the parts (**local response**) can result in a major damage of the building, including the possibility of total collapse. To ensure an overall dissipative and ductile behavior, brittle failure or the premature formation of unstable mechanisms must be avoided. That is why the detailing of connections between structural elements and of regions where nonlinear behavior is foreseeable should receive special care in design. It may be necessary to resort to the so-called *capacity design procedure*, which will be discussed later.
- It is to ensure, that an adequate structural model for the analysis is used, which shall take into account the influence of soil deformability, of non-structural elements, adjacent structures, etc.
- Structures with special characteristics – with irregularities in plan and/or in elevation – or the problem of unequal excitation at all support points shall be analyzed with appropriate spatial models.

2.2.3.2 Foundations

- The foundation's stiffness shall be adequate for transmitting the actions received from the superstructure to the ground as uniformly as possible.
- Unless the object consists of dynamically independent units, only one foundation type should in general be used for the same structure (except in bridges).
- The construction site and the nature of the supporting ground should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification in the event of an earthquake [1].

Quality system plan

Structures of special importance in regions of high seismicity shall possess formal system plans, covering design, construction and use – additional to the control procedures prescribed in other relevant Eurocodes.

2.3 Application to Buildings

2.3.1 Basic principles of conceptual design

Engineers designing a building being capable of withstanding an earthquake have the possibility to choose various structural components and then combining them to a lateral load resisting system. Such systems, which will have to balance the demands of earthquake resistance, building costs, building use and architectural design, normally include:

- Diaphragms
- Shear walls
- Braced frames
- Moment resisting frames
- Horizontal trusses

Diaphragms are horizontal resistant elements, generally floors and roofs, that collect and transmit the lateral forces to the vertical resistance elements (shear walls or frames) and ensure that those systems act together in resisting the horizontal seismic action.

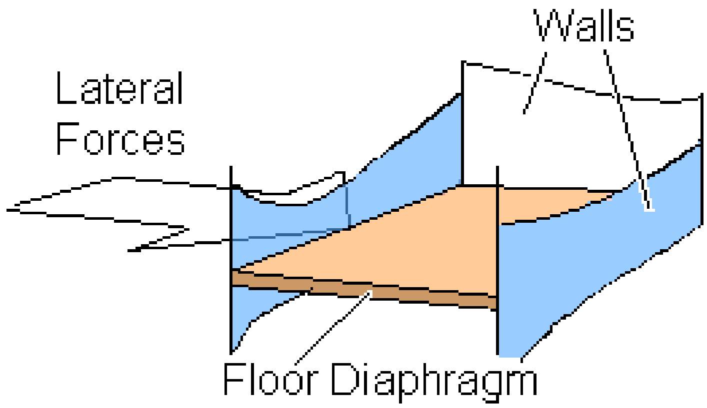


Figure 2.5: Horizontal Diaphragm action [6]

Shear Walls are vertical elements that are designed to receive lateral forces from diaphragms and transmit them to the ground. The forces in these walls are predominantly shear forces.

Braced Frames act in the same manner as shear walls but are offering lower resistance depending on the details of their design and construction (normally they are made of steel). Such details depend on the material's **ductility** in situations, when the bracings are elonged or compressed as a consequence of vibration.

Moment Resistant Frames are typically created by the joints between columns (vertical) and beams (horizontal). As these details are becoming highly stressed they are very important. For such frames the principle of energy absorbtion obtained by permanent deformation of the structure prior to ultimate failure is utilized. Steel structures with bolts or welded joints, as well as properly reinforced concrete frames provide resistance capacity by distortion prior to failure and do not fail in a brittle manner.

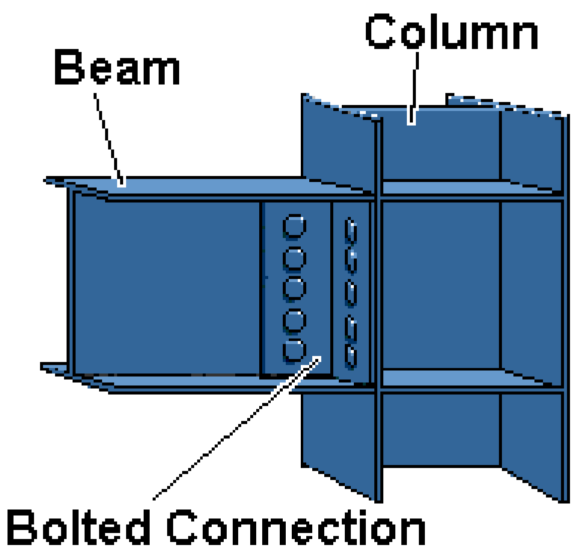


Figure 2.6: Beam-Column Joint [6]

The conceptual design against seismic hazards including the structural elements, that have just been discussed, is governed by the following guiding demands. They are going to be discussed and specified below:

2.3.1.1 Structural simplicity

The existence of clear and direct paths for the transmission of seismic forces to make the prediction of the seismic behavior much more reliable (Figure 2.7).

2.3.1.2 Uniformity, symmetry and redundancy

Uniformity in-plan (Figure 2.7), an even distribution of the structural elements (implemented by a symmetrical structural layout), allows short and direct transmission of the inertia forces created by the distributed masses of the building. Possibly uniformity may be realised by subdividing the entire building by seismic joints into dynamically independent units.

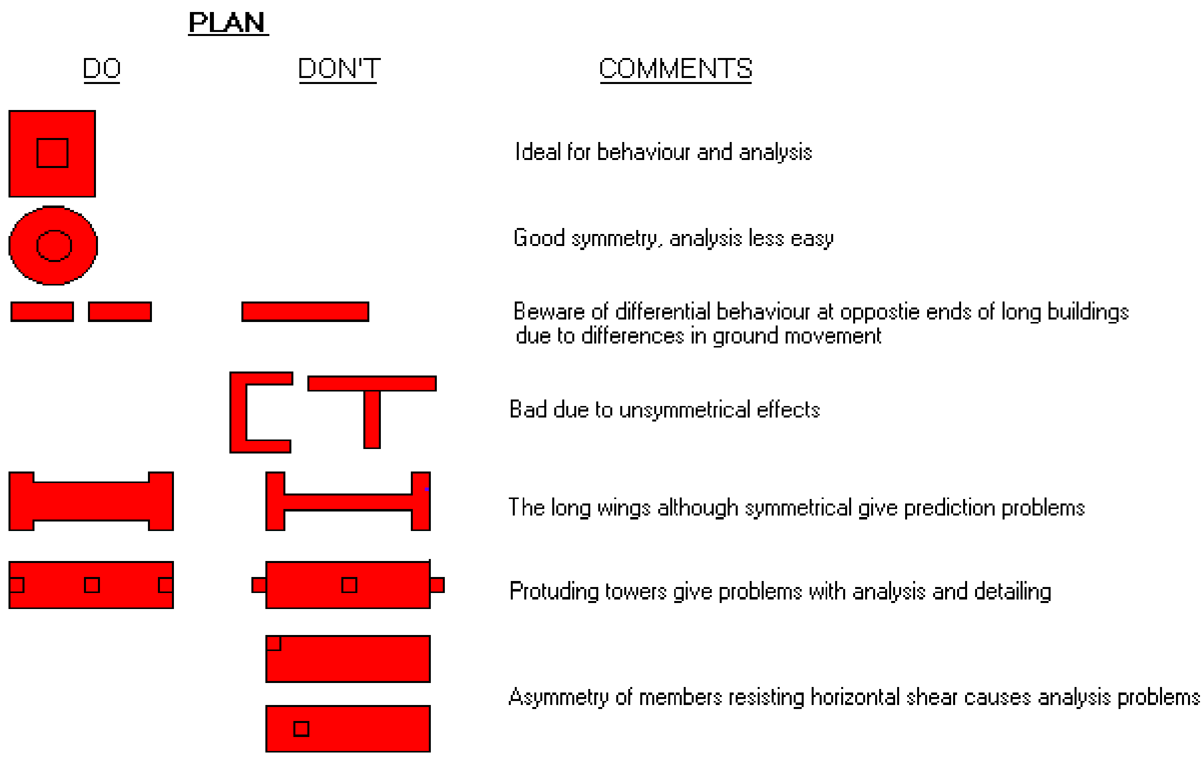


Figure 2.7: Typical visuals to make clear advantageous and disadvantageous in-plan concepts [7]

A close relationship between the distribution of masses *(center of this distribution **M**)* **and the distribution of resistance and stiffness** *(center of this stiffness **S**)* eliminates large eccentricities between mass and stiffness – leading to unpleasant torsional effects initiated by the residual seismic force **E** (Figure 2.8).

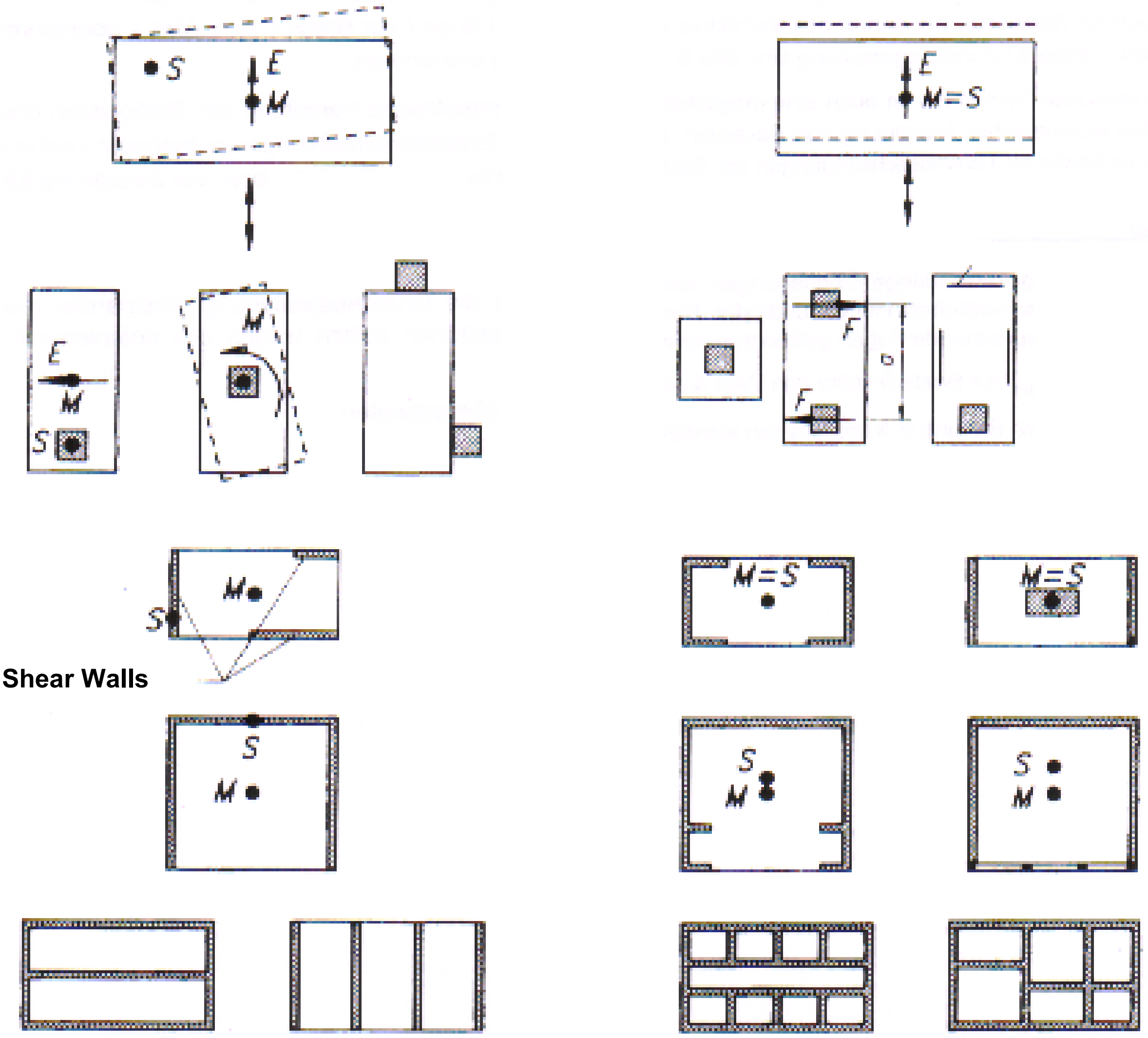


Figure 2.8: Adverse (left) and advisable (right) relationship between the centers of mass and stiffness [8]

Uniformity along the height of the building tends to eliminate the occurrence of sensitive zones, where concentrations of stress and large ductility demands might prematurely cause collapse (Figure 2.10 – left shows the unpleasant collapse mechanism of the so-called Soft-Story).

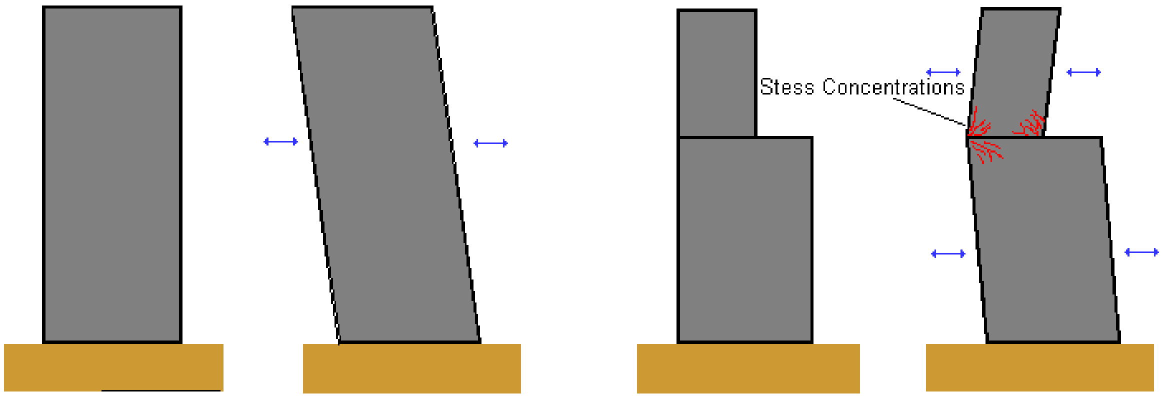


Figure 2.9: Regular and irregular shape along the height of earthquake excited structure [7]

This also requires that the centers of lateral stiffness **S** and mass **M** are approximately on a vertical line and close to each other.

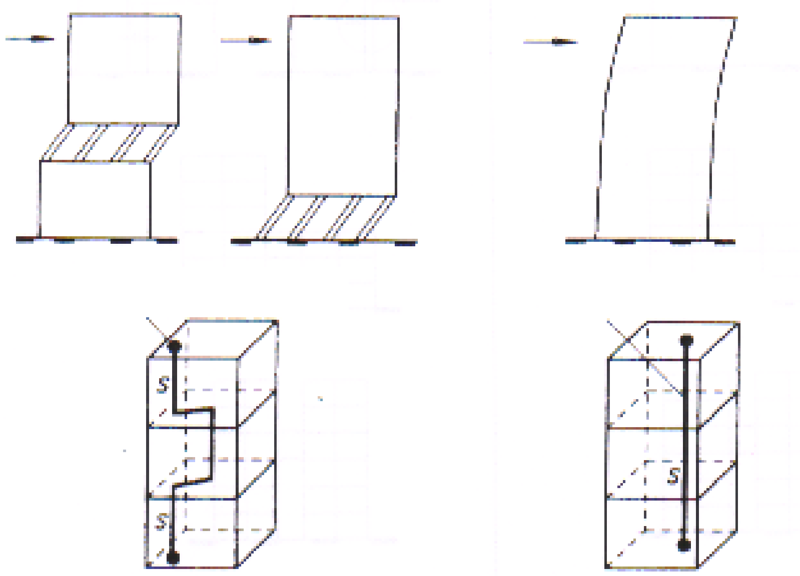


Figure 2.10: Adverse (left) and advisable (right) run of the vertical axis of stiffness [8]

Redundancy, implemented with the use of evenly distributed structural elements, allows a more favourable redistribution of action effects and widespread energy dissipation across the entire structure, which leads to an increased total dissipated energy.

2.3.1.3 Bi-directional resistance and stiffness

As horizontal seismic motion is a bi-directional phenomenon, the building structure shall be able to resist horizontal actions in any direction. This resistance realized by appropriate stiffness characteristics of the structure should minimize the effects of seismic action and should limit the development of excessive displacements leading to instabilities or large damages (Figure 2.8).

2.3.1.4 Torsional resistance and stiffness

Building structures should possess adequate torsional resistance and stiffness in addition to their lateral resistance and stiffness in order to limit the development of torsional motions which tend to stress the different structural elements in a non-uniform way. To avoid these problems, the main elements resisting the seismic action should be distributed close to the periphery of the building (Figure 2.7, Figure 2.8).

2.3.1.5 Diaphragmatic behavior at storey level

Floor systems and the roof should have sufficient in-plane stiffness to ensure the distribution of horizontal and inertia forces to the vertical structural systems with the help of effective connections, particularly when there are significant changes in stiffness or offsets of vertical elements above and beneath the diaphragm.

The function of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together. Particularly care should also be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements [1].

2.3.1.6 Adequate foundation with regard to seismic action

Foundations and their connection to the superstructure shall be designed and constructed in a way to ensure the building's subjection to a uniform seismic excitation.

Structures composed of a discrete number of structural walls, likely to differ in width and stiffness, should be based on a rigid, box-type or cellular foundation, containing a foundation slab and a cover slab (Figure 2.11).

Buildings with individual foundation elements (footings or piles) should get retrofitted with a foundation slab or tie-beams between these elements in both main directions.

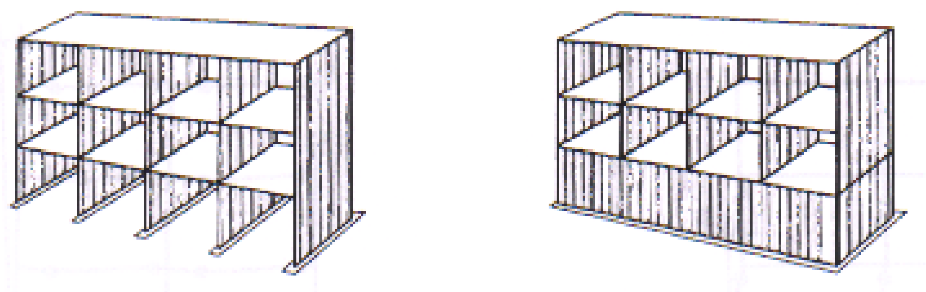


Figure 2.11: The modification of a building's foundation using a box-type solution [8]

2.3.1.7 Primary and secondary seismic elements

Certain structural members like beams and/or columns may be designated as so-called secondary seismic members. They are not part of the seismic action resisting system of the building, therefore the strength and stiffness of these elements against seismic action shall be neglected. Nonetheless these members and their connections shall be designed and detailed to maintain support of gravity loading when subjected to the displacements caused by the most unfavorable seismic design condition *[1*]. All members not designed as secondary seismic are treated as primary seismic members.

2.3.1.8 Criteria for structural regularity

For seismic design, building structures have to be distinguished as regular and non-regular (that term was determined for the first time in section 2.2.3.1 and has already been used in the subchapters in Section 2.3). As a consequence of certain regularity characteristics, Table 2.2 shows the implementation into the process of analysis.

Regularity		**Allowed Simplification**		**Behaviour factor**
Plan	**Elevation**	**Model**	**Linear-Elastic Analysis**	**(for linear analysis)**
Yes	Yes	Planar	Lateral force	Reference value
Yes	No	Planar	Modal	Decreased value
No	Yes	Spatial	Lateral force	Reference value
No	No	Spatial	Modal	Decreased value

Table 2.2: Consequences of structural regularity on seismic analysis and design [1]

For reasons of expediency this paper has a limited level of detail. The certain rules for regularity with all their exceptions are shown in EC 8-1, Chapter 3.4.2.3. The table below determines the need of decreasing the behavior factor **q** (the decreased values of the behavior factor are given by the reference values multiplied by 0,8). The values for **q** depend on the type of construction and the used material. Section 2.4 will demonstrate the main differences, which have to be followed.

2.3.1.9 Importance classes

Table 2.3 shows that buildings are classified in 4 importance classes, depending on the consequences of collapse 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.

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

Table 2.3: Importance classes for buildings [1]

The value of γ_I for importance class III is by definition equal to 1.0 – what is associated with a seismic event having the reference period $\mathbf{T_{DLR}}$ as indicated in Section 2.2.2.1. The values to be ascribed for γ_I are included in every National Annex, due to the various seismic zones in every country. Equation (*2.4*) shows the implementation of that parameter into the process of analysis.

$$A_{Ed} = \gamma_I \cdot A_{Ek} \tag{2.4}$$

In the absence of reliable statistical evaluation of seismological data, different targets of reliability may be implemented by defining the design seismic action $\mathbf{A_{Ed}}$ as the product of the characteristical seismic action coming from the response spectrum and the importance factor γ_I.

2.3.2 Structural analysis in dynamics

2.3.2.1 Modeling

- The analyzed building shall be represented by a model having appropriate distributions of stiffness and mass so that all significant deformation shapes and inertia forces are properly considered under seismic action.
- The model should also account for the contribution of significant joint regions to the deformability of the building.
- In general the structure may be considered to consist of a number of vertical and lateral load resisting systems, connected by horizontal diaphragms. Usually, when the latter may be considered as rigid in their plane, the masses and the moments of inertia of each floor may be lumped at the centre of gravity.
- Buildings, that fulfil the criteria of regularity in plan and along the height may be analyzed using two planar models, one for each main direction.
- The consequence of cracking for the elastic flexural and shear stiffness has to be taken into account for concrete buildings, steel-concrete composite buildings or in masonry buildings.
- Deformability of foundations, having an adverse overall influence on the structural response, shall be taken into account.
- Corresponding to Figure 2.8, the calculated centre of mass **M** at each floor **i** shall be displaced from its nominal location in each direction by an accidental eccentricity defined in equation (2.5):

$$e_{1i} = \pm 0{,}05 \cdot L_i \tag{2.5}$$

$\mathbf{e_{Ii}}$........................ *is the accidental eccentricity of storey mass **i** from its nominal location, applied in the same direction at all floors. With this measurement uncertainties in the location of mass and in the spatial variation of the seismic action are covered. $\mathbf{L_i}$ represents the floor-dimension perpendicular (at right angles of 90°) to the direction of the seismic action.*

2.3.2.2 The Classical Methods of Calculation of Forces and Displacements due to earthquake-motion

	Lateral force method	Modal response spectrum analysis	Time history analysis
Calculation type	static, linear	dynamic, linear	dynamic, non linear
Application	Design	Structural analysis, Design	Structural analysis
Complexity	relatively low	appropriate	high
range of application	regular, nornal structures	irregular, and/or more important structures	irregular, and/or more important structures
Parameters for structural analysis & design	bearing capacity, displacements	bearing capacity, displacements	local ductility requirements, displacements
representative for earthquake	response spectrum	response spectrum	accelerogramms

Table 2.4: Overview of practically applied methods of earthquake analysis

Table 2.4 shows two types of linear-elastic analyses. The **"lateral force method"** is applicable only to buildings meeting certain conditions defined in Section 2.3.2.3. The **"modal response spectrum analysis"** is applicable to all types of buildings. Even if the degree of difficulty is too high to be described with a few words, the so-called **"Non-linear time-history analysis"** is being discussed in the closing ANNEX, as it is the only method taking into account the time dependency of the response of the structure to an earthquake. For further details the EC 8 and complementary literature may be used.

Normally linear-elastic analysis may be performed using two planar models, one for each main horizontal direction, if the criteria for regularity in plan are satisfied (positioned in EC 8-1). Referring to Table 2.2 buildings, which do not achieve the criteria of regularity, have to be analyzed using a spatial model. In such cases the seismic action shall be applied along the relevant horizontal directions with regard to the structural layout of the building.

2.3.2.3 Lateral force method

In the course of using the **lateral force method** a quasi-static calculation with linear-elastic material behavior is made. The earthquake action is substituted by horizontal static forces. This type of calculation is applied to buildings whose response is not significantly affected by contributions from higher modes of vibration than the first one. A so-called **mode** is the **characteristical shape** of a vibrating structure (excited with a certain frequency) displaced from its neutral position. The time, which is spent for the full cycle of vibration has already been defined as the period of vibration **T** in Section 2.2.2.4.

The following condition for the fundamental period of vibration T_1 (which is the one with the lowest corresponding vibration frequency) has to be met in the two main directions.

$$T_1 \leq \begin{cases} 4 \cdot T_C \\ 2.0s \end{cases} \tag{2.6}$$

The ground-type dependent parameters T_C (also defined in 2.2.2.4) is given in the National Annex.

Base shear force

The seismic base shear force F_b, which has to be used to analyse each remarkable horizontal direction is determined in equation (2.7):

$$\boldsymbol{F}_b = \boldsymbol{S}_d(\boldsymbol{T}_1) \cdot \boldsymbol{m} \cdot \boldsymbol{\lambda} \tag{2.7}$$

S_d (T_1) .... is the ordinate of the design spectrum (see 2.2.2.4) at period T_1.

T_1 .... is the fundamental period of vibration of the building for lateral motion in the direction considered.

M.................. is the total mass of the building computed in accordance with 2.2.3.1.

λ................... is a correction factor, the value is equal to:

λ = 0,85 if $T_1 < 2\ T_C$ and the building has more than two storys, otherwise λ = 1,0.

The fundamental vibration period T_1 of the building can be determined with different expressions based on methods of structural dynamics in literature (compared to the value determined with a structural dynamics-software).

Alternatively there are different reliable approximations for the fundamental vibration period T_1 depending on the specific situation and the building's layout.

One of these estimations is formulated in equation (2.8):

$$T_1 = 2 \cdot \sqrt{d} \tag{2.8}$$

In this expression **d** represents the lateral elastic displacement of the top of the building (in m) due to the gravity loads applied in the horizontal direction.

Distribution of the horizontal seismic forces

Assuming that the analyzed building has rigid floors, the horizontal forces $\mathbf{F_i}$ shall be distributed to the lateral load-resisting system as follows:

The fundamental mode shape in the horizontal direction of a building may be calculated with the help of methods of structural dynamics or may be approximated assuming the horizontal displacements increasing linearly along the height of the building.

- When applying the first possibility, the horizontal forces $\mathbf{F_i}$ for the analyzed planar model in every storey mass $\mathbf{m_i}$ have to be calculated with the following equation:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_j \cdot m_j} \tag{2.9}$$

F_i .... is the horizontal force acting on storey i.

F_b .... is the seismic base shear force according to expression (2.7)

S_i ,s_j .... displacements of masses m_i , m_j in the fundamental mode shape, when i is the index of the actual calculated storey force and j is an index over all existing storeys (Figure 2.24; Figure 2.12).

M_i ,m_j .... masses computed in accordance with 2.2.3.1.

- When the fundamental mode shape is approximated by horizontal displacements- increasing linearly along the height- the horizontal forces $\mathbf{F_i}$ are given by equation (2.10):

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j} \tag{2.10}$$

z_i , z_j .... heights of the masses m_i , m_j above the level of application of the seismic action (foundation).

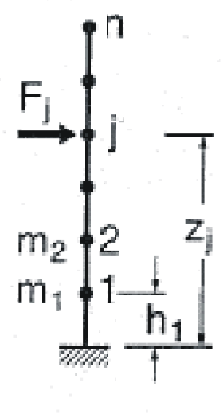


Figure 2.12: Modeling of a building and clarification of the equation (2.10).

2.3.3 Modal response spectrum analysis

When using the **modal response spectrum method** a dynamic calculation with linear-elastic material behavior and viscous damping is made. Again the earthquake action is substituted by horizontal static forces. This type of calculation is applied to buildings, which meet the conditions Table 2.2 of and Table 2.4. Even if some of the following remarks and details are not clear at this stage (they can not all be explained right here), they will become so after having studied the practical demonstrations in Section 2.6.

The response of all relevant modes of vibration – contributing significantly to the global response – shall be taken into account. This can be satisfied by either of the following:

- The sum of the effective modal masses for the modes taken into account amounts to at least 90 % of the total structure's mass.
- All modes with effective modal masses greater than 5 % of the total mass are to be considered.

Whether a spatial model is used, the above conditions have to be verified for each relevant direction. In the case of non-compliance of the two conditions above, the minimum number k of modes to be taken into account in a spatial analysis should satisfy the following conditions:

$$T_k \leq 0{,}20s \quad \text{and} \quad k \geq 3 \cdot \sqrt{n} \tag{2.11}$$

k .... is the number of modes taken into account.
N .... is the number of storys above ground.
T_k .... represents the period of vibration of mode k.

Combination of modal responses

The response in two vibration modes **I** and **j** (including both translational and torsional modes) may be considered as independent of each other, if their periods $\mathbf{T}_i$ and $\mathbf{T}_j$ (with $\mathbf{T}_j \leq \mathbf{T}_i$) satisfy the following condition [1].

$$T_j \leq 0{,}9 \cdot T_i \tag{2.12}$$

Provided that all relevant modal responses may be regarded as independent of each other, **the probable maximum value $\mathbf{E}_E$ of a seismic action effect** (force, displacement etc.) shall be taken in general equal to the square root of the sum of squares of the modal responses $\mathbf{E}_{Ei}$ due to the vibration mode **i**. The fundamental idea of the SRSS-rule comes from statistics.

$$E_E = \sqrt{\sum E_{Ei}{}^2} \tag{2.13}$$

In cases, when equation (2.12) is not satisfied, the SRSS rule becomes unconservative and more accurate procedures (such as the CQC – the "Complete Quadratic Combination") shall be applied.

2.3.3.1 Combination of effects of the components representing seismic action

Horizontal seismic components

In general the horizontal components of the seismic action shall be considered to act simultaneously. There are certain different ways how to combine the horizontal components of the seismic action:

- The structure's response to each component shall be evaluated separately, using the combination rule in equation (2.13):
- The probable maximum action effect E on the structure due to simultaneous occurrence of seismic actions along horizontal axes X and Y may be estimated from the maximum action effects E_x, E_y – eq. (2.13) due to independent seismic action along each axis as follows:

$$E = \sqrt{E_x^2 + E_y^2} \tag{2.14}$$

  This procedure generally gives a safe side estimate of the probable values of other action effects acting simultaneous with the maximum value.

- Therefore the following alternative is possible: The action effects due to the combination of the horizontal components of the seismic action may be computed using both of the combinations in equation (2.15). The sign of each component in the following combinations shall be taken as the most unfavourable for the action effect under consideration.

$$E_{Edx} \text{"+"} 0{,}30 \cdot E_{Edy}$$

$$0{,}30 \cdot E_{Edx} \text{"+"} E_{Edy} \tag{2.15}$$

E_{Edx} .... *are the action effects due to the application of the design seismic action along the chosen horizontal axis x of the structure*

E_{Edy} .... *are the action effects due to the application of the same design seismic action along the orthogonal horizontal axis* y *of the structure*

It is to bear in mind that the analysis of different horizontal directions can also lead to different structural systems or regularity classifications in elevation and to a varying behavior factor **q**.

Vertical seismic components

The vertical component of the seismic action was defined in section 2.2.2.4. It is to take into account in the cases below, provided $\mathbf{a_{vg}}$ is greater than 0,25g:

- Horizontal or nearly horizontal structural members spanning 20 m or more
- Horizontal or nearly horizontal cantilever components
  (= the end of a local structure without supports) longer than 5 m or more
- Horizontal or nearly horizontal prestressed components
- Beams supporting columns
- Base-isolated structures

To determine the effect of the vertical component of seismic action the analysis may be based on a partial model of the structure. This includes the elements on which the vertical component is considered to act (e.g. those listed in the previous paragraph) and takes into account the stiffness of the adjacent elements.

In cases, when the **horizontal components of the seismic action are also relevant** for these elements, the rules for combining the effects of horizontal seismic components may be applied – extended to three components of the seismic action. As an alternative, all three of the following combinations (2.16) may be used for the computation of the action effects:

$$0{,}30 \cdot E_{Edx} "+" 0{,}30 \cdot E_{Edy} "+" E_{Edz}$$

$$E_{Edx} "+" 0{,}30 \cdot E_{Edy} "+" 0{,}30 \cdot E_{Edz}$$

$$0{,}30 \cdot E_{Edx} "+" E_{Edy} "+" 0{,}30 \cdot E_{Edz} \tag{2.16}$$

E_{Edz} .... are the action effects due to the application of the vertical component of design seismic action.

2.3.4 Safety verifications

This section is just a short preview to the explicit requirements formulated in EC 8. They consist of the specific measures formulated in Section 2.2.3 and of the relevant limit states with certain individualities in dependence of the used material.

Ultimate limit state – No collapse requirement

The no collapse requirement under the seismic design situation is considered to be ensured if the following conditions are met:

- **Resistance condition (most important of all):**

For all structural elements, connections and non-structural elements the design value of the action effect $\mathbf{E_d}$ due to the seismic design situation must fall below or equalize the corresponding design resistance $\mathbf{R_d}$ of the element, calculated according to the rules specific to the pertinent material.

- **Second-order effects (P-Δ effects):**

It needs to be verified, if – due to the displacements coming from seismic action – the demand of an equilibrium-calculation is to refer to the neutral position or to the position in the state of maximum displacement.

- **Global and local ductility:**

Structural elements and the structure as a whole shall have a verified adequate ductility, taking into account the expected exploitation of ductility, which depends on the selected system and the behavior factor.

Specific material-related requirements as defined in Section 2.4 in general, and much more detailed in EC 8-1, shall be satisfied

- **Equilibrium condition:**

Stability is demanded under the set of actions of the seismic design situation, including such effects as overturning and sliding.

- **Resistance of horizontal diaphragms:**
- **Resistance of foundations:**
- **Seismic joint condition:**

Buildings shall be protected from earthquake-induced pounding with adjacent structures or between structurally independent units of the same building [1].

Serviceability limit state – Damage limitation requirement

Serviceability –under seismic action – is considered to be fulfilled, when the interstorey drifts are limited according to EC 8. Interstorey drifts are evaluated as the difference of the average lateral displacements at the top and bottom of the storey under consideration and calculated with equation (2.17):

$$d_s = q_d \cdot d_e \tag{2.17}$$

This expression assumes that the displacements induced by the design seismic action shall be calculated on the basis of the elastic deformations of the structural system.

D_s................. is the displacement of a point of the structural system induced by the design seismic action.

Q_d.................... represents the displacement behavior factor, assumed equal to q unless otherwise specified.

D_e................. displacement of the same point of the structural system, determined by a linear analysis based on the design response spectrum according to 2.2.2.4 (torsional effects shall be taken into account).

Additional verifications for the serviceability limit state may be required in the case of buildings important for civil protection or containing sensitive equipment.

2.4 Specific rules for the different types of applied materials

In the following there is a short excerpt of the individual properties and differences between designing buildings made of concrete, steel, steel-concrete composite, timber and masonry. As this section should show just the basic differences in classification and treatment due to the different materials, continuative information is to extract from the EC 8.

2.4.1 Concrete buildings

This section applies to the design of reinforced concrete buildings in seismic regions. The following rules are additional to those given in EN 1992-1:200X.

Earthquake resistant concrete buildings should possess an adequate energy dissipation capacity and an overall ductile behavior without substantial reduction the structure's overall resistance against horizontal and vertical loading. That means that a large volume of the structure including different elements and locations in all its storys is to involve to enforce ductile modes of failure (e.g. flexure) before brittle failure modes (e.g. shear). The potential regions for plastic hinge formation shall possess high plastic rotational capacities.

Alternatively designed buildings with low dissipation capacity and low ductility (applying only the rules of EN 1992-1 for the seismic design situation and neglecting the specific provisions of EC8) are recommended only in low seismicity cases.

Design concepts

Concrete structures are classified into two ductility classes – **DCM** (medium ductility) and **DCH** (high ductility) – in dependence of the hysteretic dissipation capacity. Both correspond to structures, which are designed, dimensioned and detailed according to specific earthquake resistant provisions. To integrate a concrete building into ductility classes **M** or **H**, specific provisions for all structural elements shall be satisfied. Each class uses different values of the behavior factor **q** (Table 2.5). The selection of the ductility class in a certain country may be found in its National Annex.

Structural types

Concrete buildings shall be classified to one of the following structural types according to their behavior under horizontal seismic actions:

- **Frame Systems** are structural systems, in which vertical and lateral loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65 % of the total shear resistance of the whole structural system.
- **Wall Systems** are structural systems, in which vertical and lateral loads are mainly resisted by vertical structural walls (either coupled or uncoupled). Their shear resistance at the building base exceeds 65 % of the total shear resistance of the whole structural system.
- **Dual systems (frame- or wall-equivalent*)*** are structural systems, in which vertical loads are mainly resisted by spatial frames and lateral loads are resisted partly by the frame system and partly by structural walls, single or coupled (= composed of two or more single walls by adequately ductile beams).
- **Ductile wall systems (coupled or uncoupled)** consist of walls fixed at the base so that the relative rotation of the base with respect to the rest of the system is prevented. They are designed and detailed to dissipate energy in a flexural plastic hinge zone just above its base [1]
- **System of large lightly reinforced walls:** Such walls have large cross-sectional dimensions due to which it is expected to develop limited cracking and inelastic behavior in the seismic design situation. It cannot be designed effectively for energy dissipation through plastic hinging at the base.
- **Inverted Pendulum Systems** are characterized by the fact, that 50% or more of the mass is in the upper third of the structure's height, or the energy dissipation takes place mainly at the base of a single building element.
- **Core Systems** are dual or wall systems without a minimum torsional rigidity, e.g. a structural system consisting of flexible frames combined with walls concentrated near the centre of the building in plan.

Behavior factors for horizontal seismic actions

This representative for energy dissipation capacity, introduced in 2.2.2.4, is derived with the following equation for each design direction:

$$q = q_0 \cdot k_w \geq 1{,}5 \qquad (2.18)$$

q_0 .... is the basic value of the behavior factor. It depends on the type of the structural system and on the regularity in elevation

k_W .... is a factor reflecting the prevailing failure mode in structural systems with walls (EC 8-1, Section 5.2.2.2).

For buildings – regular in elevation – the basic values $\mathbf{q_0}$ for the structural types that have just been discussed are given in Table 2.5. For buildings, which are irregular in elevation, the value of $\mathbf{q_0}$ should be reduced by 20 %.

STRUCTURAL TYPE	DCH	DCM
Frame system, dual system, coupled wall system	4,5 α_u/α_l	3,0 α_u/α_l
Walls system	4,0 α_u/α_l	3.0
Core system	3.0	2.0
Inverted pendulum system	2.0	1.5

Table 2.5: Basic behavior factor values q_0 in systems regular in elevation [1]

As Table 2.5 shows, the EC 8 suggests specified recommendations for the multiplier α_u/α_l in cases, when no evaluation through calculations is carried out.

α_l .... *is the multiplier of the horizontal seismic design action at first attainment of member flexural resistance anywhere in the structure, while all other design actions remain constant.*

α_u .... *is the multiplier of the horizontal seismic design action with all other design actions constant, at formation of plastic hinges in a number of sections sufficient for the development of overall structural instability.*

The maximum value for α_u/α_l used in design is equal to 1,5.

Capacity design rule

This paper is primarily motivated by the necessity of demonstrating the calculation of the structure's reaction due to earthquake action without going into too much explicit detail incorporating design and detailing rules.

Nevertheless the understanding of a fundamental and characteristical rule – the **Capacity Design Rule** – should be given. Elastic approaches to seismic design have the disadvantage, that the problem of non-linear behavior is not addressed directly (Section 2.2.2.4) but assumes that the strengths allocated on the basis of elastic analysis will be adequate once yielding has occurred [9]. This is not necessarily true. In the course of that procedure the design process (= allocation of strengths and ductilities) and the analysis are independent. Normally capacity design is combined with static lateral force method (Section 2.3.2.3). It allocates strength throughout the structure in a rational manner and secures the hierarchy of resistances of the various structural components. By this way the capacity design philosophy obliges the engineer to design the structure in such a way that yielding hinges can only form in predetermined positions. The process stipulates the margin of strength necessary for non-yielding elements to ensure that their behavior remains elastic [9]. The method's name comes from the fact, that – in the yielding condition – the strength developed in a weaker member is related to the capacity of the stronger one. By bringing to mind the need of avoiding the failure-mechanism in Figure 2.10 (left), it is to determine that yielding cannot occur in the columns of a single storey or a multi-storey building. A strong column-weak beam yielding pattern is ensured (Figure 2.13).

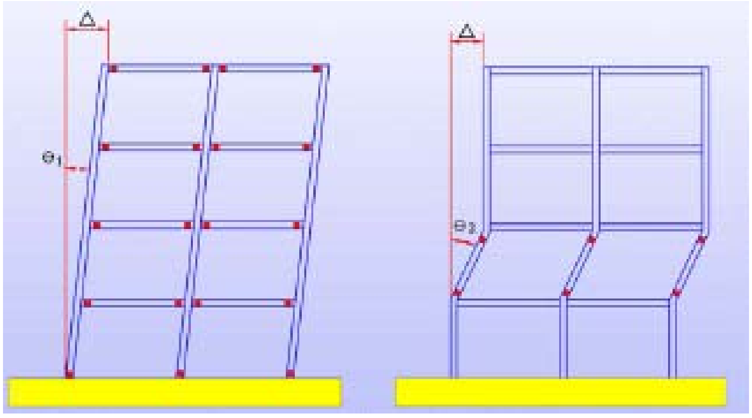


Figure 2.13: Comparison between different failure mechanisms: The recommended Beam-Damage-Mechanism (left); the Column concentrated Damage-Mechanism (not recommended) [11]

Other undesirable failure mechanisms are shear-failure of structural elements, failure of beam-column joints, yielding of foundations or of any element intended to remain elastic. Two factors included in the EC 8 might ensure that the elastic non-yielding elements are sufficiently strong so that the risk of yielding is small. **The first factor** α_{CD} includes the demand, that the sum of design values of the bending moments at the observed joint occurring at the vertical members $\mathbf{M_{Rc}}$ must be bigger than the sum of the design values of the bending moments at the horizontal ones $\mathbf{M_{Rb}}$. The so-called overstrength-effect in the yielding members is also to bear in mind by being integrated into the equilibrium condition. The equation (2.19) shows, how overstrength effects are calculated.

The intended plastic mechanism is analyzed under permanent actions and the level of seismic action at which all intended flexural hinges have developed bending moments equal to an appropriate upper fractile of their flexural resistance, called the overstrength moment $\mathbf{M_0}$.

$$\sum M_{Rc} \geq 1{,}3 \cdot \sum M_{Rb}$$

$$\sum M_{Rb} = M_0 = \gamma_0 \cdot M_{Rd} \tag{2.19}$$

γ_0.................. is the overstrength factor. Its value shall be taken in general as 1,35.

M_{Rd}............... is the design flexural strength of the section – in the selected direction and sense – based on the actual section geometry and reinforcement configuration and quantity according to EC 2.

The second factor δ deals with uncertainty in the distribution of forces in the yielding structure, as the direction of action will change during the earthquake. These two values are combined and applied to the capacity of non-yielding elements.

Capacity design effects for members, which have to remain elastic, need not being taken greater than these resulting from the design seismic combination, equation (2.3), where the design effects $\mathbf{A_{ed}}$ include a ductility factor $\mathbf{q}$ = 1 (elastic).

2.4.2 Steel buildings

This section applies to the design of steel buildings in seismic regions. The following rules are additional to those given in EN 1993-1.

Design concepts

Steel structures designed to resist earthquakes follow two main concepts of ductility:

- Concept a) Dissipative structural behavior
- Concept b) Low-dissipative structural behavior

Concept a) implies the capability of dissipative zones to resist earthquake actions through inelastic behavior. Such structures have to belong to the ductility classes M or H. In theses cases specific requirements (structural type, class of steel sections and rotational capacity of connections) have to be met.

Concept b) implies the calculation of the action effects based on an elastic global analysis without taking into account significant non-linear material behavior. When using the design spectrum defined in 2.2.2.4, the behavior factor is taken equal to 1,5 – 2. The resistance of the members and of the connections should be evaluated based on EN 1993-1-1 without any additional requirement.

Design concept	Behaviour factor q	Required ductility class
Concept b) Low dissipative structure	$\underline{1.5}$ - 2	L (Low)
Concept a) Dissipative structure	$\underline{1.5} - 2 < q < 4$	M (Medium)
Concept a) Dissipative structure	$q \geq 4$	H (High)

Table 2.6: Design concepts, behavior factors and structural ductility classes [1]

The Table above shows the discussed design concepts and their consequence to the ductility classes **L**, **M** and **H**, evaluated with the behavior factor **q** (recommendations are underlined). The ranges for **q** as well as the ductility class may be found in the National Annex.

In cases, when capacity design is performed – as described in the specified section for concrete – a material overstrength factor for steel γ_{ov} is taken into account with a recommended value of 1,25. Other values for partial safety factors are – as always – included in the National Annex.

Structural types

Steel buildings shall be classified to one of the following structural types according to the behavior of the primary resistant structure under seismic actions (the meaning of the following definitions will get clearer after having compared them to Table 2.7 and Table 2.8):

- ***Moment resisting frames*** are structural systems, in which horizontal forces are mainly resisted by members acting in a essentially flexural manner. In these structures the energy-dissipation should mainly take place in plastic hinges in the beams or the beam-column joints by means of cyclic bending.
- ***Frames with concentric bracings*** are structural systems, in which the horizontal forces are mainly resisted by members subjected to axial forces. In these structures the energy-dissipation mainly takes place in the tensile diagonals.
- ***Frames with eccentric bracings*** are structural systems, in which the horizontal forces are mainly resisted by axially loaded members. The eccentricity of the layout leads to an energy-dissipation in seismic links by means of either cyclic bending or cyclic shear.
- ***Inverted Pendulum Systems*** have been defined in the specified section for concrete before. Dissipative zones are located in the columns. This type of structure can be considered a moment resisting frame provided that the earthquake resistant structures possess more than one column in each resisting plane and that the limitation of axial force $\mathbf{N_{Sd} < 0{,}3N_{pl,Rd}}$ is satisfied. $\mathbf{N_{Sd}}$ represents the design axial force, $\mathbf{N_{pl,Rd}}$ represents the design compression resistance – both according to EC 3.
- ***Systems with concrete cores or concrete walls*** are characterized by the fact, that horizontal forces are mainly resisted by these cores or walls.
- ***Moment resisting frames with concentric bracings***
- ***Moment resisting frames with infills***

Design criteria for dissipative structures

Structures with dissipative zones (with an adequate ductility and resistance according to EN 1993-1-1) shall be designed such that yielding, local buckling or other phenomena due to hysteretic behavior do not affect the overall stability of the structure. Dissipative zones may be located in the members or in the connections, if the effects of such connections on the structure's behavior are assessed.

When dissipative zones are located in the members, the non-dissipative parts and the connections of the dissipative parts to the rest of the structure shall have sufficient overstrength to allow the development of cyclic yielding in the dissipative parts [1].

For the opposite scenario with the location of the dissipative zones in the connections, the members shall have sufficient overstrength to allow the development of cyclic yielding in the connections.

Behavior factors for horizontal seismic actions

Appropriate values for that representative for energy dissipation capacity, are given in Table 2.7 and Table 2.8, as long as the regularity requirements discussed in Section 2.3 are met.

Buildings, with a non-regularity in elevation, should be treated with reduced values for **q** by 20%.

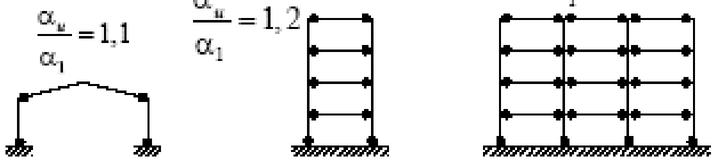
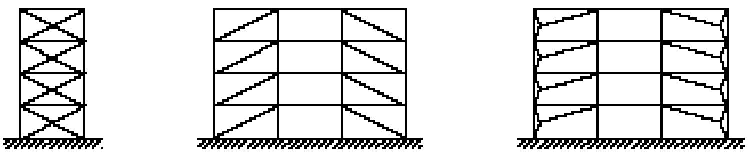
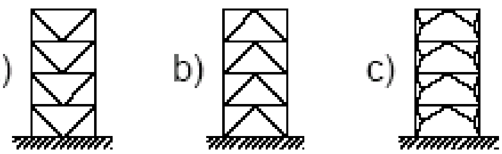
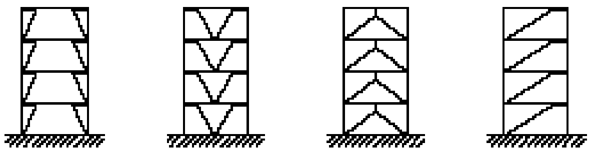
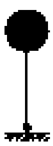
	Ductility Class H	Ductility Class M
a) Moment resisting frame. $\frac{\alpha_u}{\alpha_1}=1,1$ $\frac{\alpha_u}{\alpha_1}=1,2$ $\frac{\alpha_u}{\alpha_1}=1,3$  ▪ Dissipative zones in the beams and bottom of columns	$5\frac{\alpha_u}{\alpha_1}$	4
b) Frame with concentric bracings.		
Diagonal bracings.  Dissipative zones -tension diagonals only-.	4	4
V - bracings. a) b) c)  Dissipative zones (tension & compression diagonals).	2,5	2
c) Frame with eccentric bracings. $\frac{\alpha_u}{\alpha_1}=1,2$  **-** Dissipative zones (bending or shear links).	$5\frac{\alpha_u}{\alpha_1}$	4
d) Inverted pendulum. $\frac{\alpha_u}{\alpha_1}=1$  - Dissipative zones at the column base.  $\frac{\alpha_u}{\alpha_1}=1,1$ - Dissipative zones in columns $N_{Sd}/N_{Pl.Rd}>0,3$	$2\frac{\alpha_u}{\alpha_1}$	2

Table 2.7: Structural types and maximum associated behavior factors q *[1]*

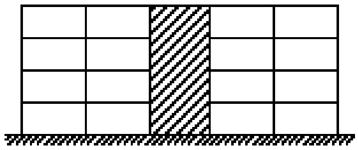
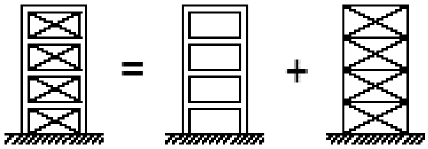
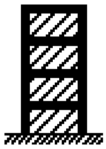
	Ductility Class	
	H	M
e) Structures with concrete cores or concrete walls. 	See section 5.	
f) Moment resisting frame with concentric bracing.  $\frac{\alpha_u}{\alpha_1} = 1,2$ Dissipative zones: in moment frame and in tension diagonals.	$4\frac{\alpha_u}{\alpha_1}$	4
g) Moment resisting frames with infills. 		
Unconnected concrete or masonry infills, in contact with the frame.	2	2
Connected reinforced concrete infills.	See section 7.	
Infills isolated from moment frame: see moment frames.	$5\frac{\alpha_u}{\alpha_1}$	4

Table 2.8: Continuation of Table 2.7 [1]

Table 2.7 and Table 2.8 show proposals for specified recommendations due to the EC 8 for the multiplier α_u/α_l in cases, when no evaluation through calculations is carried out.

α_l .... *is the multiplier of the horizontal seismic design action which corresponds to the point where the most strained cross-section reaches its plastic resistance, while all other design actions remain constant.*

α_u .... *is the multiplier of the horizontal seismic design action with all other design actions constant, at the point of where a number of sections, sufficient for the development of overall structural instability, reach their plastic moment of resistance.*

The maximum value for $\alpha_u/\ \alpha_l$ used in design is equal to 1,6.

2.4.3 Steel concrete composite buildings

This section applies to the design of composite steel concrete buildings in seismic regions. The following rules are additional to those given in EN 1994-1.

Design concepts

Composite structures designed to resist earthquakes follow three main concepts of ductility:

- Concept a) Dissipative structural behavior with composite dissipative zones
- Concept b) Dissipative structural behavior with steel dissipative zones
- Concept c) Low dissipative structural behavior

Design concept	Behaviour factor q	Required ductility class
Concept c) Low dissipative structure	<u>1.5</u> - 2	L (Low)
Concept a) or b) Dissipative structure	<u>1.5</u> - 2< q < 4	M (Medium)
Concept a) or b) Dissipative structure	q ≥ 4	H (High)

Table 2.9: Design concepts, behavior factors and structural ductility classes [1]

The Table above shows the discussed design concepts and their consequence to the ductility classes **L**, **M** and **H**, evaluated with the behavior factor **q** (recommendations are underlined). The ranges for **q** as well as the ductility class may be found in the National Annex.

The *concepts a) and b)* imply the capability of dissipative zones to resist earthquake actions through inelastic behavior. In such cases the design response spectrum defined in 2.2.2.4 is used (the behavior factor is automatically greater than 1,5 and depends on the structural type).

Concept b) implies structures taking no advantage of composite behavior in dissipative zones. Therefore the application of concept b) is conditioned by a strict compliance to measures that prevent involvement of the concrete in the resistance of dissipative zones. The composite structure is designed according to EN 1994-1 under non-seismic loads and according to the section before (steel buildings) to resist earthquake action. The explicit measures preventing involvement of the concrete are defined in EC 8-1; Section 7.7.5.

Concept c) implies the calculation of the action effects based on an elastic global analysis without taking into account significant non-linear material behavior. This concept does consider the reduction in moment of inertia (which is in charge for the flexural rigidity when it is combined with the elasticity modulus) due to the cracking of concrete in part of the beam spans. When using the design spectrum defined in 2.2.2.4, the behavior factor is taken equal to 1,5. The resistance of the members and of the connections should be evaluated based on EN 1993-1-1 and EN 1994-1-1 without any additional requirements.

Concept a) implies the capability of dissipative zones to resist earthquake actions through inelastic behavior. The EC 8 defines specific criteria in order to aim at the development of reliable local plastic mechanisms and of reliable global plastic mechanisms dissipating as much energy as possible under the design earthquake action. Such structures have to belong to the ductility classes M or H. In theses cases specific requirements (structural type, class of steel sections and rotational capacity of connections and detailing) have to be met.

In cases, when capacity design is performed – as described in the specified section for concrete – a material overstrength factor for steel γ_{ov} is taken into account with a recommended value of 1,25. Other values for partial safety factors are – as always- included in the National Annex.

Structural types

Composite steel concrete buildings shall be classified to one of the following structural types according to the behavior of the primary resistant structure under seismic actions. Moment frames, concentrically braced frames, frames with eccentric bracings and inverted pendulum structures have already been defined in the previous sections and determined with their value for **q** in Table 2.7 and Table 2.8. Table 2.10 shows composite structural systems behaving like walls and composite steel plate shear walls.

- ***Moment resisting frames*** in the actual context follow the same definition and limitations as in Section 2.4.2, but right here beams and columns may be either steel or composite.
- ***Composite frames with concentric bracings*** in the actual context follow the same definition and limitations as in Section 2.4.2, but right here beams and columns may be either structural steel or composite structural steel. Braces shall be made of structural steel.
- ***Composite frames with eccentric bracings*** in the actual context follow the same definition, configuration and limitations as in Section 2.4.2. Members which do not contain the links may be either structural steel or composite structural steel. Other than for the slab, the links shall be made of structural steel. Dissipative action occurs only through yielding in shear of these links.
- ***Inverted Pendulum Systems*** in the actual context follow the same definition and limitations as in Section 2.4.2.
- ***Composite structural systems behaving essentially as reinforced concrete walls*** are listed and determined in Table 2.10. These composite systems may belong to one of the following types:

  Type 1 fits with a steel or composite frame working together with concrete infill panels connected to the steel structure.

  Type 2 are reinforced concrete walls with encased steel sections (connected to the steel structure) used as vertical edge reinforcement.

  Type 3 are steel or composite beams, used to couple two or more reinforced concrete or composite walls.

  In all introduced types, energy-dissipation takes place in the vertical steel sections and in the vertical reinforcements of the walls. In Type 3, energy dissipation may also take place in the coupling beams.

  In cases, when the wall elements are not connected to the steel structure, Section 2.4.2 applies.

- ***Composite steel plate shear walls*** are conceived by a vertical steel plate continuous over the height of the building with reinforced concrete encasement on one or both sides of the plate and structural steel or composite boundary members.

Design criteria for dissipative structures

Structures with dissipative zones (with an adequate ductility and resistance according to EN 1994-1-1) shall be designed such that yielding, local buckling or other phenomena due to hysteretic behavior do not affect the overall stability of the structure. Dissipative zones may be located in the members or in the connections, if the effects of such connections on the structure's behavior are assessed.

When dissipative zones are located in the members, the non-dissipative parts and the connections of the dissipative parts to the rest of the structure shall have sufficient overstrength to allow the development of cyclic yielding in the dissipative parts [1].

For the opposite scenario with the location of the dissipative zones in the connections, the members shall have sufficient overstrength to allow the development of cyclic yielding in the connections.

Behavior factors for horizontal seismic actions

Appropriate values for that representative for energy-dissipation capacity, are given in Table 2.7, Table 2.8 and Table 2.10, as long as the regularity requirements discussed in Section 2.3 are met.

Buildings, with a non-regularity in elevation, should be treated with reduced values for q by 20%.

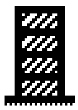
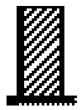
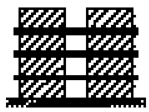
	Ductility Class H	Ductility Class M
e) Composite structural systems.		
Composite walls. $\frac{\alpha_u}{\alpha_1} \approx 1{,}1$ TYPE 1  TYPE 2  Steel or composite moment frame with connected concrete infill panels. Concrete walls reinforced by connected encased vertical steel sections.	$4\frac{\alpha_u}{\alpha_1}$	$3\frac{\alpha_u}{\alpha_1}$
Composite or concrete walls coupled by steel or composite beams. TYPE 3 	$4{,}5\frac{\alpha_u}{\alpha_1}$	$3\frac{\alpha_u}{\alpha_1}$
f) Composite steel plate shear walls. $\frac{\alpha_u}{\alpha_1} \approx 1{,}2$	$4\frac{\alpha_u}{\alpha_1}$	$3\frac{\alpha_u}{\alpha_1}$

Table 2.10: Continuation of Tables 4-3 and 4-4 [1]

Table 2.7, Table 2.8 and Table 2.10 show proposals for specified recommendations due to the EC 8 for the multiplier α_u/α_l in cases, when no evaluation through calculations is carried out.

The maximum value for α_u/α_l used in design is equal to 1,6.

2.4.4 Masonry buildings

This section applies to the design of unreinforced, confined and reinforced masonry in seismic regions. The following rules are additional to those given in EN 1996.

Materials and bonding patterns

- **Masonry units** should have sufficient robustness in order to avoid local brittle failure (see EC 8-1, Section 9 and EN 1996-1).
- **Minimum strength of masonry units** is, except for cases of low seismicity, the normalized compressive strength of masonry units derived in EN 772-1 and should not fall below the minimum values defined in the Country's National Annex.
- **Minimum strength of mortar** is generally required. It exceeds the minimum specified in EN 1996 and is also defined in the Country's National Annex.
- **Masonry bond:** Except in cases of low seismicity and unless mechanical interlocking between masonry units is provided along perpend joints, such joints shall be fully filled with mortar [1].

Types of construction and behavior factors

Masonry buildings shall be classified to one of the following types of construction in dependence on the masonry type used for the seismic resistant elements:

a) unreinforced masonry construction,
b) confined masonry construction,
c) reinforced masonry construction,
d) construction with industrially produced reinforced masonry systems.

The low tensile strength and low ductility results in the fact, that ***unreinforced masonry***, that follows the provisions of EN 1996 alone, is considered to offer low-dissipation capacity (DCL) – its use is recommended only in low seismicity cases. Even if unreinforced masonry satisfies the provisions of the Eurocode 8, it may not be used in cases of exceedance of the design ground acceleration $\mathbf{a_g.S}$ over 0,15 g (see 2.2.2.4).

The Table below shows the discussed design concepts a) to c) and their consequence to the ductility classes **L**, **M** and **H**, evaluated with ranges for the behavior factor **q** (recommendations are underlined again).

Type of construction	Behaviour factor *q*
Unreinforced masonry according to EN 1996 alone (recommended only for low seismicity cases).	<u>1,5</u>
Unreinforced masonry according to EN 1998-1	<u>1,5</u> - 2,0
Confined masonry	<u>2,0</u> – 2,5
Reinforced masonry	<u>2,5</u> - 3,0

Table 2.11: Types of construction and behavior factor [1]

The usage of ***industrially produced reinforced masonry system*** is related with different values of the behavior factor **q**. They are determined in every National Annex and depend on the results of certain ductility tests.

Design criteria

Masonry buildings shall be composed of floors and walls, connected in the two horizontal directions and in the vertical. This connections shall be provided by steel ties or reinforced concrete ring beams. Every type of floor, that provides the general requirements of continuity and effective diaphragm action, may be used. Shear walls are to position at least in two horizontal directions.

Further information, requirements, detailing rules, etc. for masonry buildings and all other introduced materials are discussed in the pertinent sections of EC 8.

2.4.5 Final comparative remarks

This short section's intention is to make some conclusive statements that might help to distinguish between a possible application of the introduced four main building materials.

After having taken into account the different ductility classes, the structural type-dependent key figures $\mathbf{q_0}$, α_u/α_l and the complementing specifically defined material parameters it can be said:

In general *concrete* buildings are slightly less ductile than *steel* buildings (*steel* and *steel-concrete composite* buildings are approximately equal to treat in seismical manners).

Expectedly the EC 8 has assigned *timber* to have a quantitative smaller ability of energy-dissipation than materials that have already been listed in this section. Masonry has the lowest reliability to seismic action.

2.5 Fundamentals of dynamics in civil engineering [12]

Every load-bearing structure not only vibrates due to dynamic superimposed loads but also a "quasi stationary" structure reacts on excitations always present in nature by vibrations. These so-called ambient excitations have the properties of white noise in the statistic average – all relevant frequencies are represented in the response spectrum with almost equal energy content. The minor vibrations a structure shows due to these ambient excitations can be registered by modern highly sensitive acceleration sensors.

Dynamics, i.e. the science of movements under the influence of forces, is often not so familiar to the civil engineer, because he normally uses statistic considerations for the solution of his tasks. In building design most dynamic problems (earthquakes, wind, waves, etc.) are usually treated by means of static substitute procedures (for example multiplication of static equivalent loads with factors). With this procedure the maximum member forces and deformations occurring in a structure due to dynamic influences can be approximately recorded, the vibration behavior itself can, however, only be modeled by a dynamic analysis.

2.5.1 General Survey on Dynamic Calculation Methods

The response of a structure to dynamic influences is determined by the kind of influence and by the properties of the structure itself.

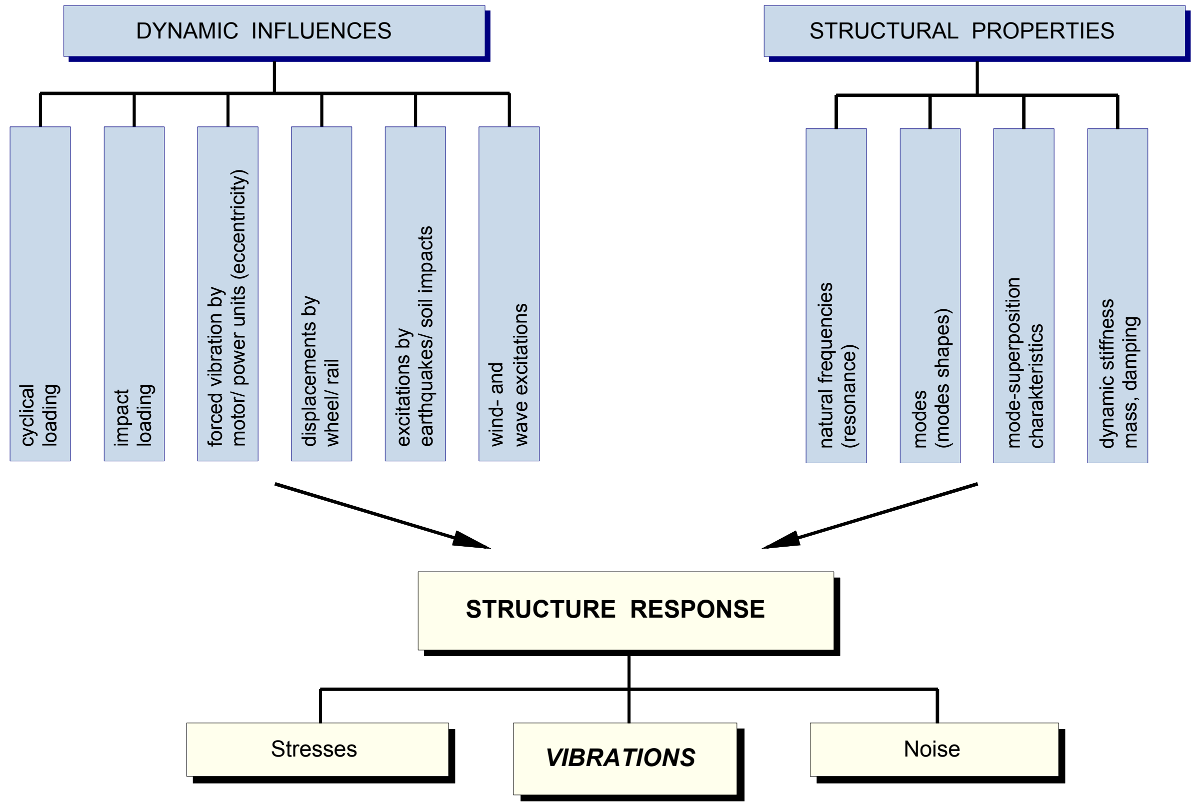


Figure 2.14: Dynamic influences and structural properties

Dynamic influences can occur as variable (operating stress) and as extraordinary events (earthquakes) as defined by the ON B 4040. The dynamic properties of the structure can be described by eigenfrequencies, mode shapes and transmission characteristics – they are on the other hand determined by stiffness, mass and damping. The response of the structure consists of stresses (member forces) and vibrations (oscillations = temporally variable modifications of form), only the latter, however, can be directly measured.

The following figure gives a general overview on the currently usual dynamic calculation methods (without claim to completeness). The most important step, which forms the basis of all calculation methods, is the correct establishment of models. For this purpose the stiffness and masses as well as the bearings of the structures have to be registered sufficiently accurately. It is very difficult to calculate the influence of damping but empirical values and measuring results can serve as a basis.

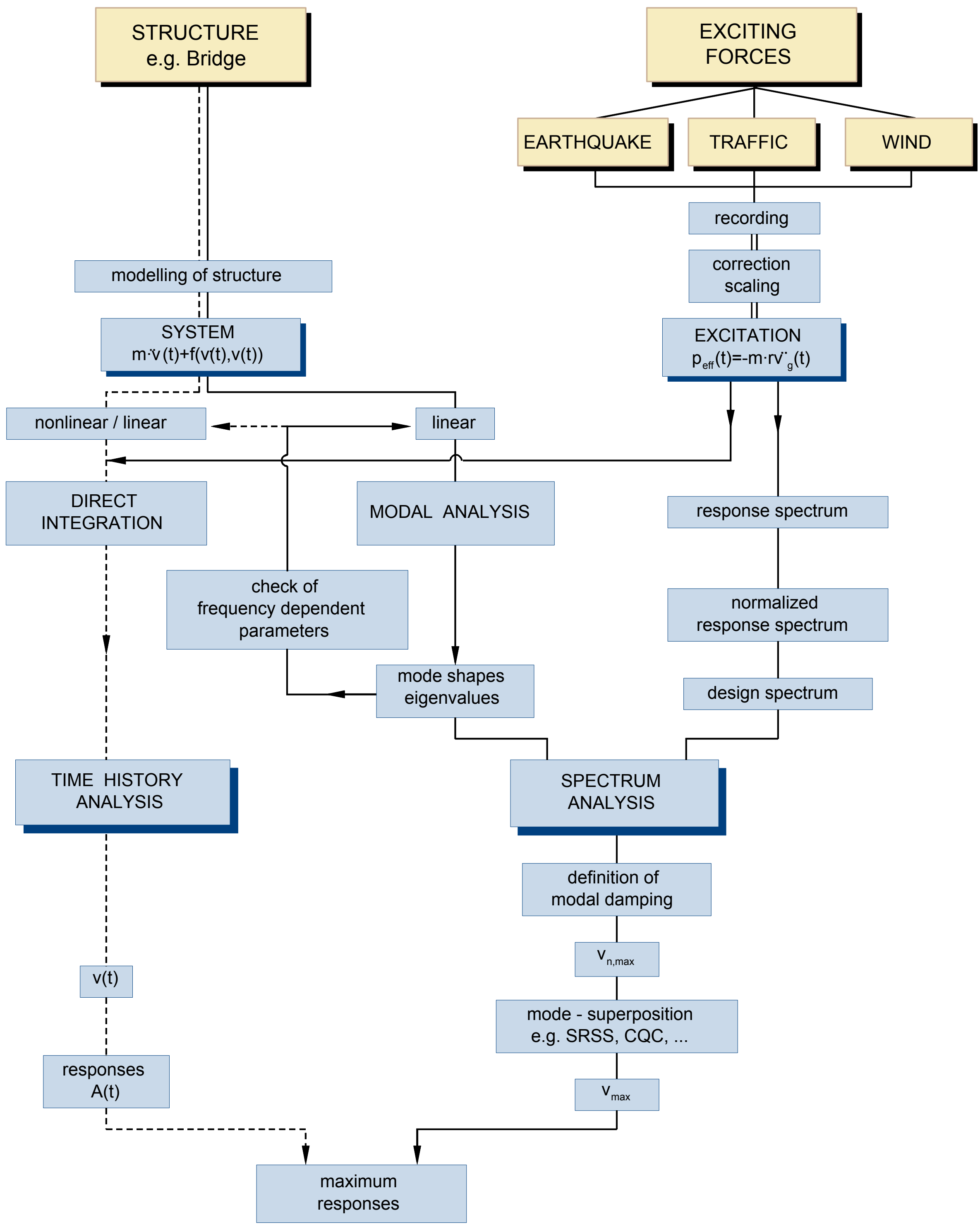


Figure 2.15: Dynamic calculation methods (according to Böhler)

The dynamic calculation methods can basically be divided into two groups: Linear and general (linear and non-linear) methods, where non-linearities can be caused by the structure (for example marginal conditions) or by the material (material laws). As general dynamic calculation methods in the form of non-linear time-history-analyses for the determination of structural responses are very time-consuming, usually dynamic calculations by means of the response spectrum method are linearly carried out. Here in a first step the eigenfrequencies and mode shapes are determined by modal analysis. The maximum structural response to an external influence is received by superposition of the mode shapes multiplied with the spectral values of the eigenfrequencies.

$$\boxed{[m]\cdot[\dot{x}]+[r]\cdot[\dot{x}]+[c]\cdot[x]=[F(t)]}$$

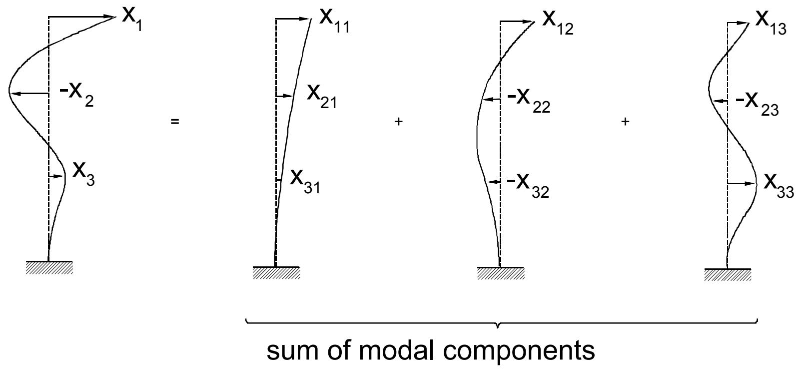
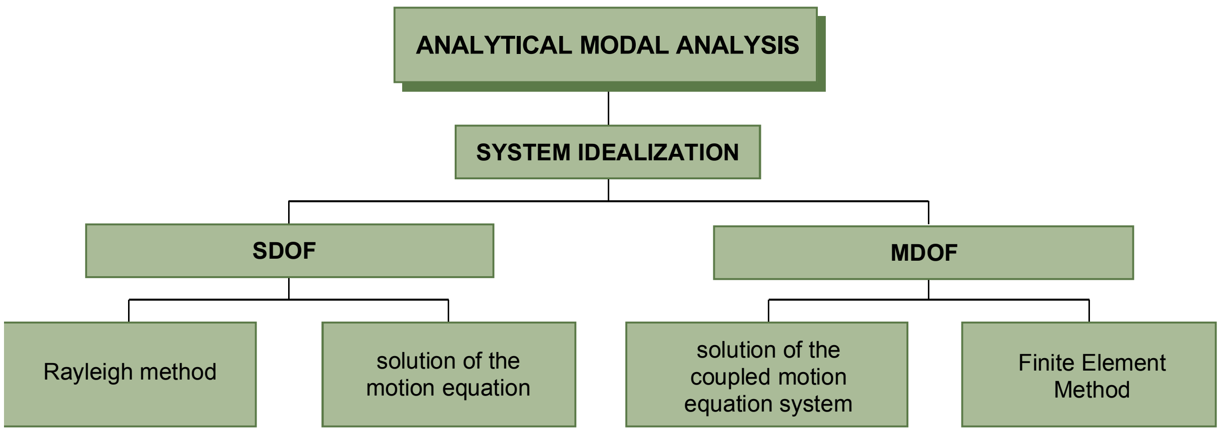


Figure 2.16: Composition of an oscillation from mode shapes

2.5.2 Short Description of Analytical Modal Analysis

Modal analysis, i.e. the determination of eigenfrequencies and mode shapes (modes) of a structure, can be carried out by means of different methods.



$$w(x,t)=\left(\sum_{i=1}^{n} c_i\cdot x^i + c_o\right)\cdot f(t)$$

$$T=\frac{1}{2}\int_o^l \dot{w}^2\,dm$$

$$J_i=\frac{1}{2}\int_o^l (w''\cdot EI)\cdot\frac{1}{EI}dx$$

$$T+U_i=\text{const}$$

$\Rightarrow \omega_e$ ... eigenkreis frequency

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

$$x(t)=x_H(t)+x_P(t)$$

$\omega_e=\sqrt{\frac{c}{m}}$ ... undamped

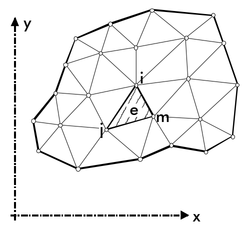
$\omega'_e=\frac{-r\pm\sqrt{r^2-4cm}}{2m}$ ... damped

$$[m]\cdot[\ddot{x}(t)]+[r]\cdot[\dot{x}(t)]+[c]\cdot[x(t)]=[F(t)]$$

$[r]=a[m]+b[c]$ ... Rayleigh damping

$\det([c]-\omega_i^2[m])$ ... eigenvalue problem

$\Rightarrow \omega_i$ ... eigenfrequencies



$[F]=[K]\cdot[a]$

Figure 2.17: Method of analytical modal analysis

A prerequisite for all methods is a linear behavior of the structure. Energetic approaches like for example the Rayleigh Method can be principally applied to SDOF and MDOF systems, an exact solution is, however, generally only possible in the first case. The direct solution of the equation of motion is, however, always possible if the damping matrix is either negligible or represented as linear combination of the mass and stiffness matrix.

In practice the Finite Element Method (FEM) is universally used today for numerical treatment of beam and shell structures. FEM is furthermore used for non-linear problems for the calculation of the structural response to general stresses.

2.5.3 Equation of Motion of Linear Structures

As eigenfrequencies and mode shapes are structural properties independent from stress (Vibrational Signature), it is only natural to use them for the assessment of the maintenance condition of structures. The basis for its determination is the general equation of motion of the structure that has to be examined.

Newton formulated the necessary basic laws in three axioms already in the 17th century:

(1) If all forces affecting a body are in equilibrium, the following applies:
$a(t) = 0$; $v(t) = \text{const.}$

(2) The temporal force influence is proportional to the modification of impulse:
$F(t) \cdot dt = D(m \cdot v(t))$

(3) Actio = Reactio

with: *a* .... *acceleration*
v .... *velocity*
F .... *force (influence)*
t .... *time*
m .... *mass*

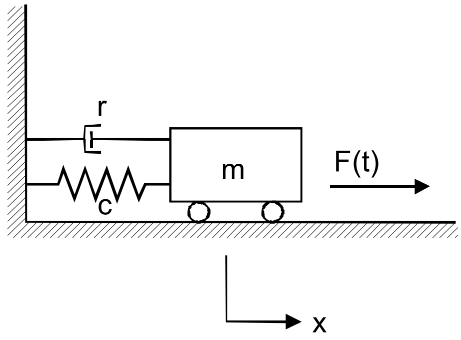
The 2nd axiom can be represented in the following formula by transformation and consideration of the critical value $dt \rightarrow 0$: $F(t) = m \cdot a(t)$. In d'Alembert's style the equilibrium condition can be formulated $m \cdot a(t) - F(t) = 0$.

2.5.3.1 SDOF System

If d'Alembert's equilibrium consideration is applied to an SDOF system considering a damping **r** proportional to velocity and a spring constant **c**, you obtain the equation of motion for a forced damped vibration. The latter has the form of a linear inhomogeneous differential equation of the second order with constant coefficients and can be therefore solved by the simple statement $x(t) = x_h(t) + x_p(t)$ in the case of a harmonic excitation.

The solution $x_h(t)$ of Euler's homogeneous differential equation is obtained by an exponential statement $e^{\lambda t}$, which describes the so-called transient effect (flowing back process to the static equilibrium condition). The particular solution is – dependent on the form of the disturbance (influence) F(t) – a constant or a harmonic function of t corresponding to the disturbance.

If F(t) is an arbitrary (non harmonic) influence, the solution x(t) can be generally represented by a so-called convolution or Duhamel integral.

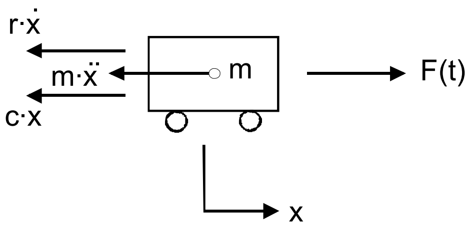


m ... mass

c ... spring stiffness (linear)

r ... damping
(velocity proportional)

Equilibrium of force at t



$$\dot{x} = \frac{dx}{dt}$$

$$\ddot{x} = \frac{d^2x}{dt^2}$$

$$\boxed{m \cdot \ddot{x} + r \cdot \dot{x} + c \cdot x = F(t)}$$

Equation of motion of a forced damped vibration of a SDOF-System

$$\omega = \sqrt{\frac{c}{m}}$$ ... natural cyclic frequency (undamped)

$$D = \frac{r}{2\sqrt{m \cdot c}}$$ ... damping according to Lehr

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

Transformed equation of motion

solution: $x(t) = x_h(t) + x_p(t)$

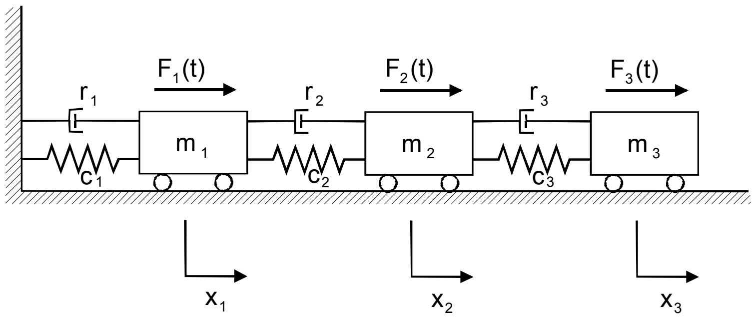
$x_h(t)$ ... solution of a homogeneous equation

$x_p(t)$ ... particular solution

Figure 2.18: Equation of motion of a SDOF system

2.5.3.2 MDOF System

The equation of motion can be established for a system with multiple degrees of freedom with an analogous procedure. You obtain, however, a differential equation system linked via stiffness matrix, which can no longer be solved by a simple statement like in the case of the SDOF system. The mass matrix [m] of the MDOF system meets the criteria of a positively definite diagonal matrix. The stiffness matrix [c] is symmetrical and positively definite according to the Maxwell-Betti theorem.



$$[m] = \begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} \qquad \text{... mass matrix}$$

$$[c] = \begin{bmatrix} c_1 + c_2 & -c_2 & 0 \\ -c_2 & c_2 + c_3 & -c_3 \\ 0 & -c_3 & c_3 \end{bmatrix} \qquad \text{... stiffness matrix}$$

$$[r] = a \cdot [m] + b \cdot [c] \qquad \text{... damping matrix (proportional)}$$

$$[x] = \begin{bmatrix} x_1 \\ x_2 \\ x_3 \end{bmatrix} \qquad [\dot{x}] = \begin{bmatrix} \dot{x}_1 \\ \dot{x}_2 \\ \dot{x}_3 \end{bmatrix} \qquad [\ddot{x}] = \begin{bmatrix} \ddot{x}_1 \\ \ddot{x}_2 \\ \ddot{x}_3 \end{bmatrix}$$

$$[F(t)] = \begin{bmatrix} F_1(t) \\ F_2(t) \\ F_3(t) \end{bmatrix}$$

$$\boxed{[m] \cdot [\ddot{x}] + [r] \cdot [\dot{x}] + [c] \cdot [x] = [F(t)]}$$

Equation of motion for forced damped vibration of the MDOF-System

Figure 2.19: Equation of motion of a MDOF system

The solution of the equation of motion is carried out in three steps:

1. Decoupling of the differential equation system (return to the SDOF systems)
2. Solution of the decoupled differential equations
3. Superposition of the individual solutions to the total solution.

The decoupling process is done by determination of the eigenfrequencies and mode shapes. They are obtained by solving the □igenvalues problems $([c] - \omega_i^2 \cdot [m]) \cdot [a_i] = 0$. This homogeneous linear equation has a non-trivial solution only if the denominator determinant of the system det $([c] - \omega_i^2 \cdot [m])$ disappears. For a system with n degrees of freedom (masses) n eigenfrequencies and eigenvectors (mode shapes, modes) have to exist.

The decoupled differential equation system, which is obtained by multiplying the coupled differential equation system with the modal matrix [a] (composed from modal forms) and its transpose $[a]^T$ due to orthogonality $[a_j]^T \cdot [m] \cdot [a_i] = 0$ and $[a_j]^T \cdot [c] \cdot [a_i] = 0$ for $j \neq i$, can be solved like n SDOF systems.

The total solution of the MDOF system is obtained by superposition of the individual solutions $Y_i(t)$, i = 1,… n to $[x(t)] = [a] \cdot [Y(t)]$.

2.5.3.3 Influence of Damping

In an damped system a complete decoupling is only possible if the damping matrix is proportional to the mass and stiffness matrix. This damping form is also called Rayleigh damping.

For the special case a = 0 – i.e. the damping matrix is only proportional to the stiffness matrix (also called relative damping) – higher eigenfrequencies are damped more quickly. In case of b = 0 – i.e. proportionality only to the mass matrix (absolute damping) – lower eigenfrequencies are, however, damped more quickly.

The condition $[r] = a \cdot [m] + b \cdot [c]$ is a sufficient but not absolutely necessary criterion for decoupling the equation of motion. In a general case the damping matrix cannot be diagonalised simultaneously with the mass and stiffness matrix, in slightly damped systems, as they are mostly existing in the building trade, non-diagonal terms can, however, be neglected. As it is usually quite difficult to establish the damping of the individual eigenfrequencies, a constant modal damping ratio of $\xi_I = \xi$ is usually assumed.

Practice shows that these assumptions are subject to high variation. The current trend is to replace assumptions by actual measurements taken from the structure after erection to verify the assumption. A major difficulty is the lack of clear definition which damping properties shall be determined. Also the fact, that damping is non-linear in dependence of the amplitude of the mode, is neglected. Results from monitoring projects suggest that a normal system damping for structures should be on the save side. Nevertheless a numerous structures have been measured with damping values below 1%. In this respect also the relation between the expected input level and the assumed damping value has to be considered. The future practice will be a combination of assumed values which are later on proven by measurements.

2.6 A simplified practical demonstration for a building

2.6.1 Introduction

The following section's emphasis is to demonstrate the application of the engineer's two main methods of earthquake analysis to a certain structure taken from [13]. Figure 2.20 shows the structure as a so-called moment resistive frame with its input-data for dynamical analysis converted into European units. This frame is going to be excited in the global X-direction (Figure 2.20).

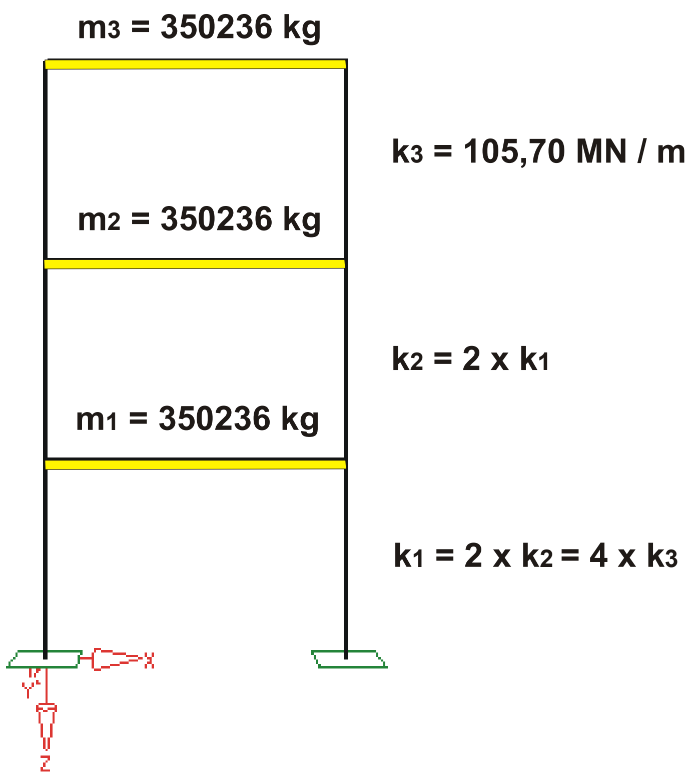


Figure 2.20: Moment-resistive frame with its key data of stiffness and participating mass

2.6.2 Modeling

The initial relation representing the bending stiffness due to horizontal forces is determined by equation (2.20).

$$k = 2 \cdot \frac{12 \cdot EI}{l^3} \quad \text{with} \quad I = \frac{d \cdot h^3}{12} \tag{2.20}$$

k .... *is the frame's stiffness per storey.*
E .... *modulus of elasticity*
I .... *is the column's cross-sectional moment of inertia*
L .... *is the height between the floors*
D .... *is the term of the column's cross section in the global Y-direction*
H .... *is the term of the column's cross section in the direction of vibration*

By means of iteration of equation (2.21) the unknown values for d are calculated and can be identified in Figure 2.21. To create a logical, possible structure corresponding with this example the obtained values are divided by 10. This leads to a distribution of the stiffness properties among 10 identical frames visualized in Figure 2.22.

$$d = \frac{k \cdot l^3}{2 \cdot E \cdot h^3 \cdot 10} \tag{2.21}$$

l [m]	E [MN/m]	h [m]	k3 =	105.7 MN/m
4	35000	0.315		
d =	30.92	[cm]		
⇒ Dimensions d / h = 31 / 31.5				

l [m]	E [MN/m]	h [m]	k2 =	210.14 MN/m
4	35000	0.37		
d =	37.93	[cm]		
⇒ Dimensions d / h = 38 / 37				

l [m]	E [MN/m]	h [m]	k1 =	420.28 MN/m
4	40000	0.425		
d =	43.80	[cm]		
⇒ Dimensions d / h = 44 / 42.5				

Table 2.12: Determination of the column's cross-sectional dimensions in every storey

The allocation of the masses given in Figure 2.20 leading to inertial effects corresponds with equation (2.2) including reasonable values for the permanent loads $\mathbf{G}_{k,j}$ and the imposed loads $\mathbf{Q}_{k,I}$ considering the combination coefficient ψ_E (=$\varphi \cdot \psi_2$) with ψ_2 = 0,3 (shopping area) and $\varphi = 1$ (Category D).

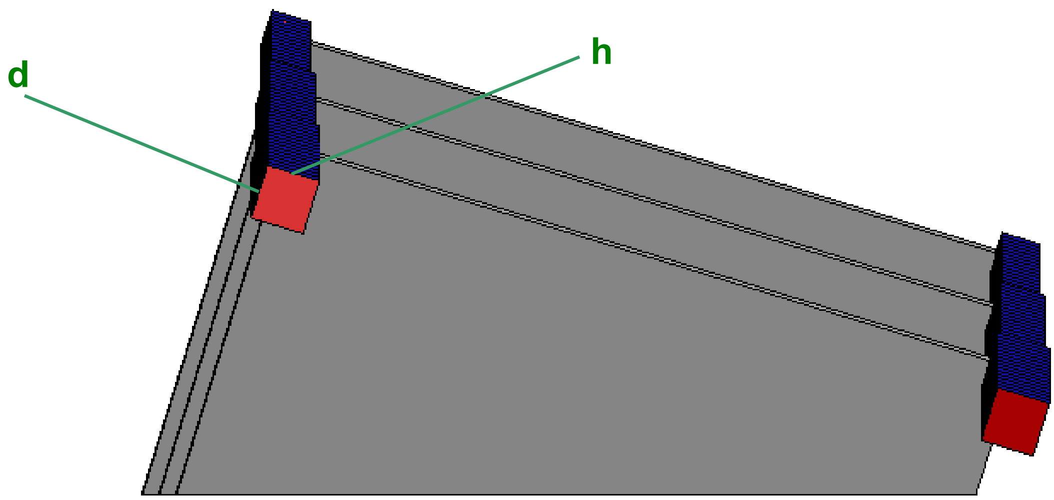


Figure 2.21: Clarification of the nomenclature of the column's cross sections

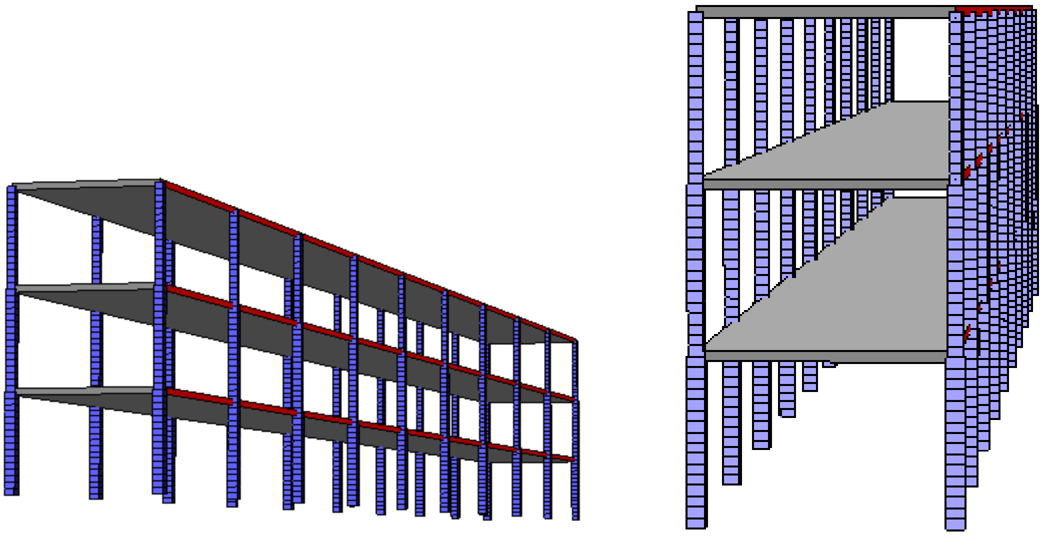


Figure 2.22: A possible building structure equivalent to the discretized Clough-Penzien-Frame

2.6.3 Dynamical analysis

The following Figure shows, how a structure like the one in Figure 2.22 could normally be simplified for a better, more rational mechanical handling. The structure – consisting of ten similar frames, connected by floor slabs – is going to be represented by a three degrees of freedom system. That means that the motion of this structure will be defined by the horizontal displacement-amplitudes of certain selected points (the floors) in the structure. The moment-resisting frame is determined by its already defined stiffness properties. The allocation of masses is realized by concentrating half of the incorporated weight per floor in every beam-column joint (the so-called lumped masses can move only in a single fixed direction). By this way all relevant internal force variables can be obtained as residual terms before being distributed to all the several parts of the structure afterwards. The following demonstration will deal with the structure represented just by that one moment-resisting frame. The concluding distribution will be discussed but not shown in detail.

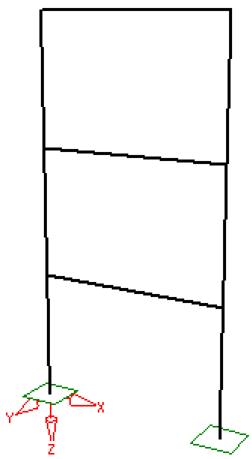


Figure 2.23: The simplified mechanical model used for further calculation

The further calculation from the mechanical model to the dynamical characteristics is based on dynamical basics included in *[14]*. The procedure is going to be accomplished without an exhaustive guideline through the used theory.

The following sections imply the assumption that the same free-field motion acts simultaneously at all support points of the structure with its foundation. As rotational motions are neglected, this assumption is equivalent to considering the foundation-soil or –rock to be rigid. When considering input motions at the base of a structure, it should be recognized that the actual structure-base motions during an earthquake may be significantly different from the corresponding free-field motions that would have occurred without the structure being present [13]. This “soil-structure interaction” effect is negligible in our case presuming a relatively stiff foundation and a relatively flexible structure.

Eigenvalue problem of a free vibrating structure:

It is assumed, that the free-vibration motion is simple harmonic and undamped, which may be expressed for the particular example – the three degree of freedom structure – with the following equilibrium condition:

$$\underline{\mathbf{m}} \cdot \ddot{\vec{x}} + \underline{\mathbf{k}} \cdot \vec{x} = \vec{0} \tag{2.22}$$

- **Mass matrix** [kg]

$$\underline{\mathbf{m}} = \begin{pmatrix} m_{33} & m_{32} & m_{31} \\ m_{23} & m_{22} & m_{21} \\ m_{13} & m_{12} & m_{11} \end{pmatrix} = m \cdot \begin{pmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{pmatrix} \qquad m = 350236 \tag{2.23}$$

The coefficients $\mathbf{m_{ij}}$ are defined as the force corresponding to coordinate **I** due to unit acceleration of coordinate **j**.

- **Stiffness matrix** [N / m]

$$\underline{\mathbf{k}} = \begin{pmatrix} k_{33} & k_{32} & k_{31} \\ k_{23} & k_{22} & k_{21} \\ k_{13} & k_{12} & k_{11} \end{pmatrix} = k \cdot \begin{pmatrix} 1 & -1 & 0 \\ -1 & 3 & -2 \\ 0 & -2 & 6 \end{pmatrix} \qquad k = 105{,}7 \cdot 10^6 \tag{2.24}$$

The coefficients $\mathbf{k_{ij}}$ are defined as the force corresponding to coordinate / due to unit displacement of coordinate **j**.

The expression beneath represents the displaced shape of the structure, which does not change with time, only the amplitude varies.

$$\vec{x} = \vec{\Phi} \cdot \cos\omega \cdot t \tag{2.25}$$

The combination of all equations above results in the so-called frequency equation. Its solution leads to the characteristical ***free-vibration frequencies*** ω. The corresponding vectors of displacement amplitudes – called ***eigenvectors*** or ***mode shapes*** – are usually expressed in dimensionless form by dividing all the components by the reference component (the largest one).

- **Vector of circular frequency** [rad / sec]

$$\underline{\omega} = \begin{pmatrix} 11{,}651 \\ 27{,}460 \\ 45{,}937 \end{pmatrix} \qquad (2.26)$$

- **Vector of natural frequency** [Hz]

$$\underline{f} = \begin{pmatrix} 1{,}854 \\ 4{,}370 \\ 7{,}311 \end{pmatrix} \qquad (2.27)$$

- **Vectors of modal shapes** [ ]

$$\underline{\Phi_1} = \begin{pmatrix} 1 \\ 0{,}5475 \\ 0{,}1974 \end{pmatrix} \quad \underline{\Phi_2} = \begin{pmatrix} -0{,}6607 \\ 1 \\ 0{,}5736 \end{pmatrix} \quad \underline{\Phi_3} = \begin{pmatrix} 0{,}0857 \\ -0{,}5170 \\ 1 \end{pmatrix} \qquad (2.28)$$

- **Matrix of the three modal shapes** [ ]

$$\underline{\mathbf{\Phi}} = \begin{pmatrix} 1 & -0{,}6607 & 0{,}0857 \\ 0{,}5475 & 1 & -0{,}5170 \\ 0{,}1974 & 0{,}5736 & 1 \end{pmatrix} \qquad (2.29)$$

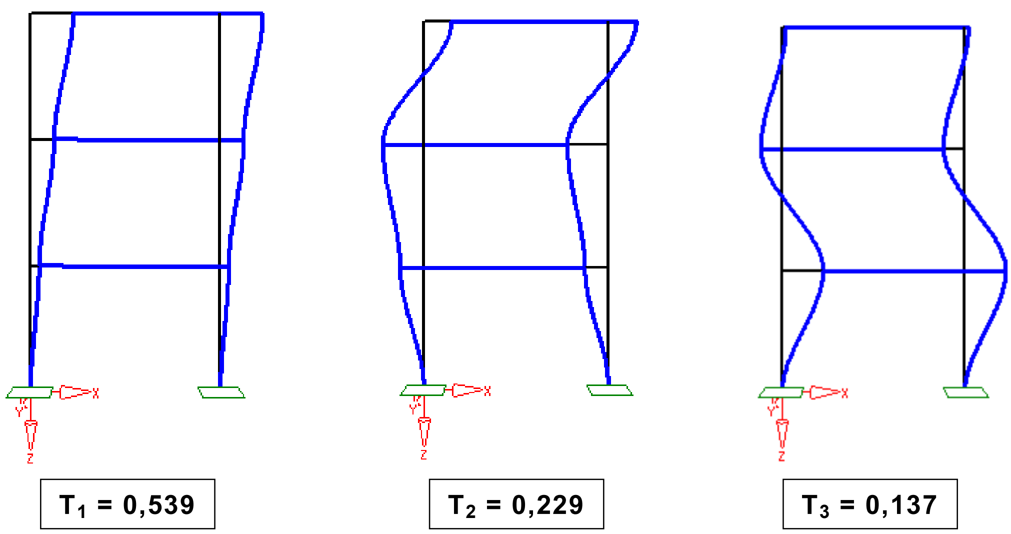


Figure 2.24: Visuals for the first three vectors of the Eigenmodes *with the appropriate period of vibration* $\mathbf{T_i}$

2.6.4 Earthquake analysis

2.6.4.1 Response spectra

In Section 2.2.2.4 the significance of response spectra as decision support for estimating a structure's response in earthquake cases has already been discussed. For reasons of completeness this whole demonstration will be carried out with an elastic and a design response spectrum to show the differences in results and further applications. The input parameters and the descriptive formulas (depending on the period of vibration **T**) for Figure 2.25 are given in equation (2.30):

The chosen location is *Nassfeld – Carinthia* in Austria. The design ground acceleration according to equation (2.1) is determined with $\mathbf{a_g}$ =1,34 m/s^2 (**k** = 1 as well as the importance factor γ_I for ordinary buildings). It is assumed, that the earthquakes that contribute most to the seismic hazard have a surface-wave magnitude $\mathbf{M_s}$ not greater than 5,5. This leads to the adoption of response spectrum Type 2 – Ground type A (rock or rock-like geological formation).

Its describing parameters according to section 2.2.2.4 are **S = 1,0; $\mathbf{T_B}$ = 0,05; $\mathbf{T_C}$ = 0,25** and **$\mathbf{T_D}$ = 1,2.** The appropriate damping correction factor **η = 1** and the behavior factor for DCM **q = 3,6** (according to equation 2.18) and Table 2.5).

$0 \le T \le T_B$	$\mathbf{S_e}(T) = a_g \cdot S \cdot (1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)$	$\mathbf{S_d}(T) = a_g \cdot S \cdot (1 + \frac{T}{T_B} \cdot (\frac{2{,}5}{q} - 1)$
$T_B \le T \le T_C$	$\mathbf{S_e}(T) = a_g \cdot S \cdot \eta \cdot 2{,}5$	$\mathbf{S_d}(T) = a_g \cdot S \cdot \frac{2{,}5}{q}$
$T_C \le T \le T_D$	$\mathbf{S_e}(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \cdot \frac{T_C}{T}$	$\mathbf{S_d}(T) = a_g \cdot S \cdot \frac{T_C}{T} \cdot \frac{2{,}5}{q}$
		$\mathbf{S_d}(T) \ge a_g \cdot 0{,}2 = \mathbf{0{,}268}$
$T_D \le T \le 4\,\mathrm{sec}$	$\mathbf{S_e}(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \cdot \frac{T_C \cdot T_D}{T^2}$	$\mathbf{S_d}(T) \ge a_g \cdot 0{,}2 = \mathbf{0{,}268}$

(2.30)

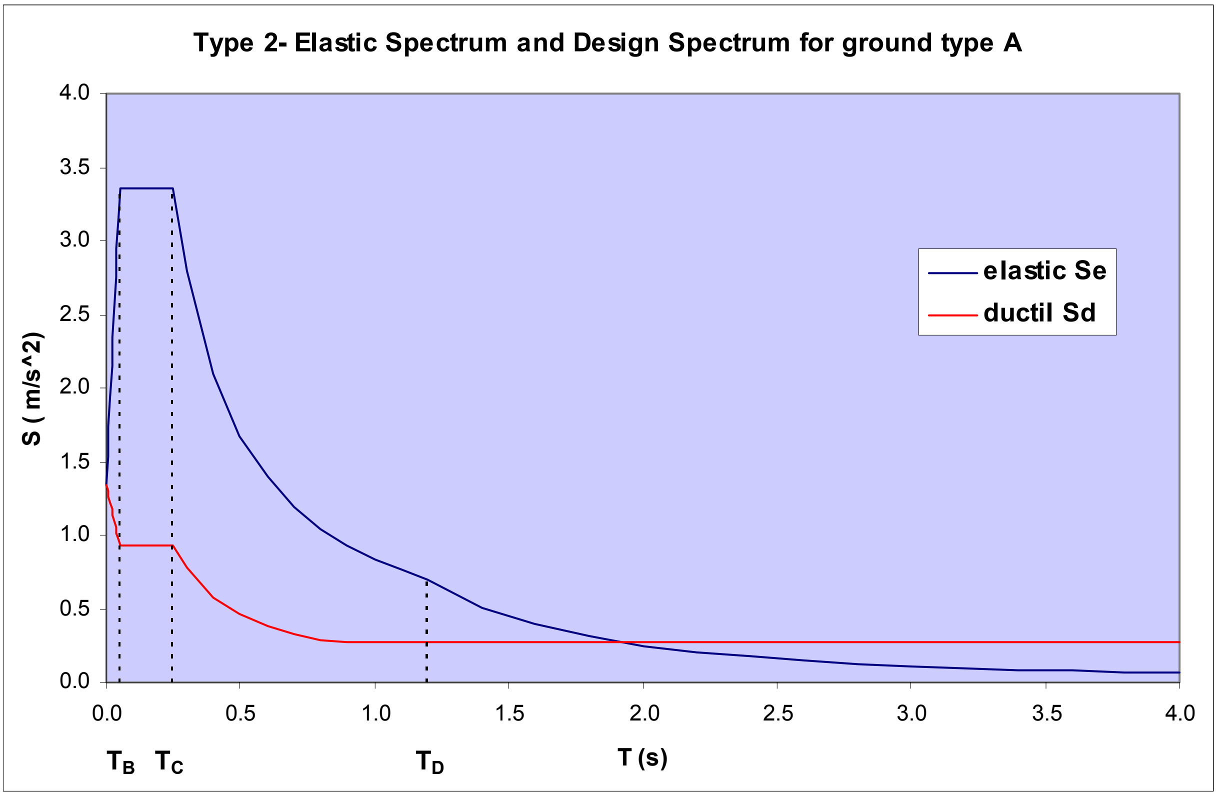


Figure 2.25: Generated Response Spectra for earthquake analysis

2.6.4.2 The Lateral Force Method

The fundamental period of vibration $\mathbf{T_1}$ fulfils the condition in equation (2.6) which allows the application of the Lateral Force Method. In compliance with section 2.6.4.1 and the equation (2.30) both relevant ordinates of the response spectrum can be ascertained:

$$\boldsymbol{S_e} = 0{,}1584 \qquad \boldsymbol{S_d} = 0{,}044$$

The total mass **m** of the structure is the sum of the three masses from Figure 2.20.

$$\boldsymbol{m} = 1050709 \ kg$$

Equation (2.7) leads to the seismic base shear forces $\mathbf{F_{bi}}$ (λ = 1,0) – shown with eq. (2.31). They are distributed as defined in equation (2.9) afterwards.

$$\mathbf{F}_{\mathbf{b}e} = \mathbf{S}_e(T_1) \cdot m \cdot \lambda = \mathbf{1664{,}2 \ kN}$$

$$\mathbf{F}_{\mathbf{b}d} = \mathbf{S}_d(T_1) \cdot m \cdot \lambda = \mathbf{462{,}3 \ kN} \qquad (2.31)$$

The following formula shows an example, how every single component of equation (2.33) – the lateral force per story – is calculated:

$$\mathrm{F_{3e}} = \mathrm{F_{be}} \cdot \frac{\mathrm{s_i} \cdot \mathrm{m_i}}{\sum \mathrm{s_j} \cdot \mathrm{m_j}} = 1664{,}2 \cdot \frac{1 \cdot 3502{,}4}{(1 \cdot 3502{,}4 + 0{,}5475 \cdot 3502{,}4 + 0{,}1974 \cdot 3502{,}4)} \qquad (2.32)$$

$$F_{3e} = 954{,}81\ kN \ldots\ldots\ldots F_{3d} = 264{,}95\ kN$$

$$F_{2e} = 522{,}12\ kN \ldots\ldots\ldots F_{2d} = 145{,}03\ kN$$

$$F_{1e} = \ \ 94{,}14\ kN \ldots\ldots\ldots F_{1d} = \ \ 52{,}30\ kN \qquad (2.33)$$

The utilized Finite Element-Software RSTAB [17], used for further engineering calculations, distributes the story-associated forces automatically to the main nodes of every floor (lumped masses).

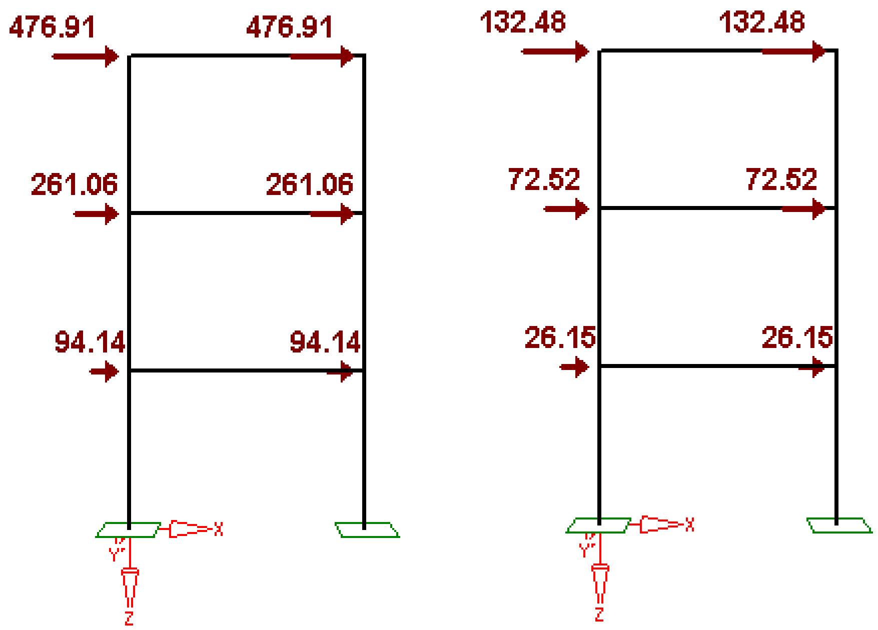


Figure 2.26: Quantified Forces [kN] as representatives of an elastic (left) or a ductile (right) answer of the structure due to the fundamental period of vibration

2.6.4.3 The Modal Response Spectrum Method

The calculated periods of vibration listed in Figure 2.24 fulfill the condition in equation (2.12) as well as the condition formulated in [2] (criterion that permits to disregard the CQC-Method), which allows the application of the Modal Response Spectrum Method.

Initial relationship: Equation of motion due to forced vibration

$$\underline{\mathbf{m}} \cdot \ddot{\vec{x}} + \underline{\mathbf{k}} \cdot \vec{x} = -\ddot{w}_g \cdot \underline{\mathbf{m}} \cdot \vec{e} \qquad \textit{with} \qquad \vec{e} = \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix} \qquad (2.34)$$

$\ddot{w}_g$ .... *represents the free-field input acceleration applied at the base of the structure*

$\vec{e}$ .... *influence coefficient vector, which represents the displacements resulting from a unit support displacement*

The already mentioned solution method for MDOF-Systems in Section 2.5.3.2 by decoupling the differential equation system (equation 2.35)

$$\vec{x} = \vec{\Phi}_1 \cdot q_1 + \vec{\Phi}_2 \cdot q_2 + ... + \vec{\Phi}_N \cdot q_N = \sum_{j=1}^{N} \vec{\Phi}_k \cdot q_k \tag{2.35}$$

leads to

$$\vec{F}_{S\,\mathrm{e},j} = \underline{\mathbf{m}} \cdot \vec{\Phi}_j \cdot \frac{L_j^*}{m_j^*} S_{\mathrm{e},j} \tag{2.36}$$

$\vec{F}_{S\,\mathrm{e},j}$ .... *Vector of modal maximum forces (equivalent to the forces associated with the structure's relative displacements)*

q_k .... *generalized coordinates representing the amplitudes of the specified set of displacement patterns*

$\underline{\mathbf{m}}$ .... *mass matrix*

$\vec{\Phi}_j$ .... *Vector of modal shape j*

L_j^* .... *is the earthquake excitation factor representing the extent to which the earthquake motion tends to excite a response in the assumed shape* $\vec{\Phi}_j$

m_j^* .... *modal mass (a constant that depends on the mode shape and the mass distribution) as a consequence of decoupling the three degree of freedom system in three "quasi one degree of freedom"-systems*

$S_{\mathrm{e},j}$ .... *value of the ground acceleration corresponding with the vibration period of the modal shape j – taken from the response spectrum*

As the generalized mass and the other properties associated with a certain degree of freedom have been evaluated, the structure may be analyzed in exactly the same way as a real SDOF system for each considered mode shape.

The modal participation factor $\mathbf{L^*_j/m^*_j}$ depends on the interaction of the mode shape with the spatial distribution of the external load.

$$\frac{L_1^{*2}}{m_1^*} + \frac{L_2^{*2}}{m_2^*} + ... + \frac{L_N^{*2}}{m_N^*} = \sum_{j=1}^{N} \frac{L_j^{*2}}{m_j^*} = \boldsymbol{m} \tag{2.37}$$

The quantity L^{*2}_j/m^*_j has the dimensions of mass and is known as the ***effective modal mass*** of the structure, because it can be interpreted as the part of the total mass responding to the earthquake in each mode [13]. Section 2.6.3 described how the structure in Figure 2.22 was transformed to a mechanical model. The chosen way of discretization to a three degree of freedom system with lumped masses in combination with equation (2.37) automatically fulfills the introductive postulated preconditions of section 2.3.3 by leading to the total mass **m**.

The distribution of the effective forces for every mode shape have been determined with equation (2.36) and are listed in Table 2.13.

$\vec{F}_{Se,j}$			
Mass i	j = 1	j = 2	j = 3
3	709.4	- 400.8	44.9
2	388.4	606.7	- 270.6
1	140.0	348.0	523.5

$\vec{F}_{Sd,j}$			
Mass i	j = 1	j = 2	j = 3
3	709.4	- 400.8	44.9
2	388.4	606.7	- 270.6
1	140.0	348.0	523.5

*Table 2.13: Distribution of the effective forces **[kN]** for every mode shape due to the elastic response spectrum (left) and the design response spectrum (right)*

One fact is implicitly to be considered. The vectors of maximum modal forces shown above are not going to occur at the same time. By conventional methods of statics the internal stress resultants like the bending moment and shear forces are computed for every mode shape j. Equilibrium in forces is given within a certain mode but not in the SRSS-superposed residual forces. That means that in accordance with equation (2.13), all relevant total values of forces and displacements for elastic and ductile behavior are calculated independently of the others. The analysis of the curve representing the distributed SRSS-superposed total shear forces leads to the determination of resulting, horizontal forces of the whole Response Spectrum Method-Procedure (Figure 2.27). These terms are to be compared with Figure 2.26.

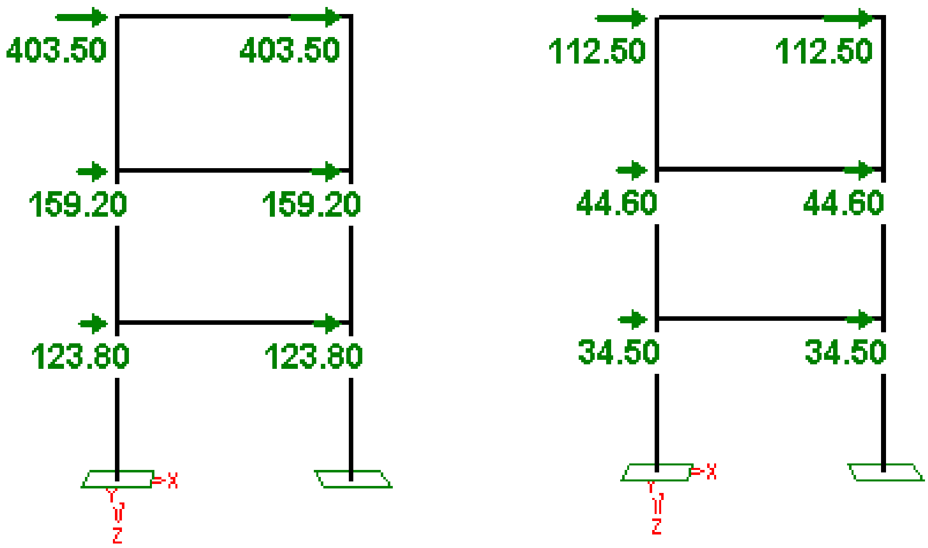


Figure 2.27: Quantified Forces [kN] as representatives of an elastic (left) or a ductile (right) answer of the structure due to the superimposed effects of the first three periods of vibration.

Annotation to these two normally practically applied analysis methods:

Up to now the *Lateral Force Method* and the *Modal Response Spectrum Method* have been demonstrated to their output, represented by horizontal forces – applied as the structure's loading. This part wants to discuss and evaluate both methods and show what terms are of explicit relevance for the further design process.

It was shown, that the excitation of the analyzed frame with its certain mass and stiffness properties results in responses in higher shape forms, which are not to be undervalued (Table 2.13). The relative high stiffness causes low periods of vibration. This leads to the fact, that both output values from the response spectrum due to the second and third period of vibration are the upper limit ones. These facts in conjunction with the calculated modal participation factors $\mathbf{L^*_j/m^*_j}$ cause values of modal maximum forces, which contribute to the superimposition of stress resultants of the Modal Response Spectrum Method.

Table 2.15 shows the contribution of the second and third mode to the total value of representatives to earthquake action of the Response Spectra Method. It demonstrates clearly the fact, why structures like these with such properties of mass and stiffness can conservatively be calculated with the Lateral Force Procedure – only in dependence of the first period of vibration (Table 2.14).

Representative Forces [kN]	***L F M***	***R S M***	**increase in relation to *R S M [%]***
3rd Floor	476.9	403.5	18.2
2nd Floor	261.1	159.2	64.0
1st Floor	94.1	123.8	-24.0
Reaction Forces the foundation[kN]			
Horizontal	832.1	694.2	19.9
Vertical	2165.6	1584.4	36.7
Selected bending moments [kNm]			
Clamped support	-1691.0	-1400.0	20.8
3rd Floor - Beam	953.8	807.5	18.1
1st Floor - Beam	3115.9	2407.7	29.4

*Table 2.14: Relation between certain representatives of the Lateral Force Method (*LFM*) and the Response Spectra Method (*RSM*)*

		Reaction Forces [kN]	**Percentage of the SRSS - Representative**
1st Mode	**H**	618.9	79.5
	V	1583.5	99.9
2nd Mode	**H**	277.0	15.9
	V	47.3	0.1
3rd Mode	**H**	148.9	4.6
	V	21.4	0.0

Table 2.15: Contribution of the individual modes to the total value of the RSM

The Lateral force method determines the mathematical reduction to a SDOF (Single Degree of Freedom) – system with the default of only one deformation shape. Regularity of stiffness and mass properties afford the usage of this method leading to an imprecise but acceptable approximation of the true dynamic behavior with conservative results.

For reasons of cost-effectiveness and a more detailed mechanical discretization of the structure the following section, which deals with the combination of gravity and lateral loads, implies the response spectrum method.

2.6.4.4 Consequences for design

Combination of actions for seismic design situations:

$$E_d = \ldots \sum_{j \geq 1} G_{k,j} + \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \; \ldots\ldots \pm \ldots\ldots A_{Ed} \tag{2.38}$$

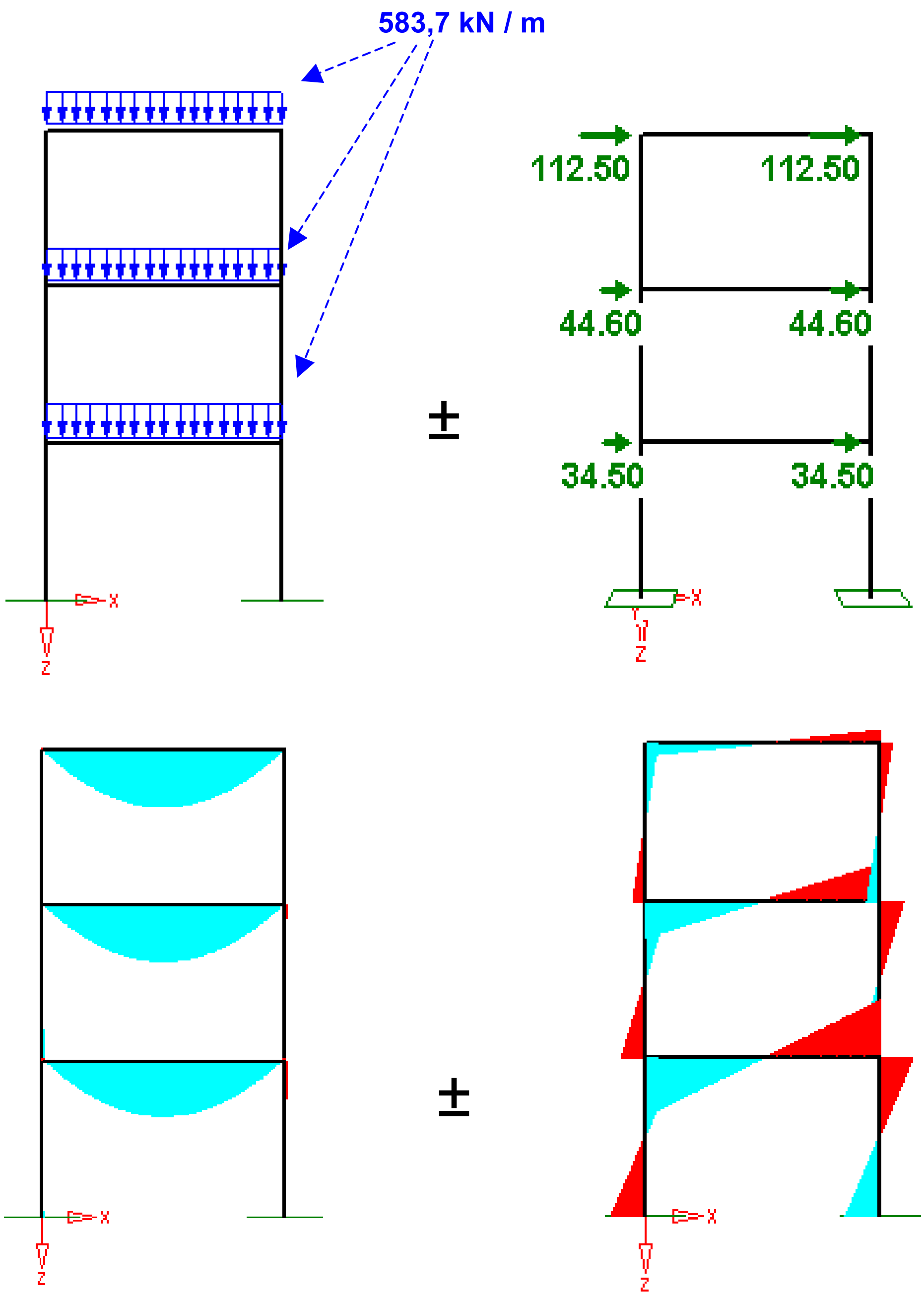


Figure 2.28: Comparison of bending moments due to gravity loads => Max = 2587,5 kNm, Min = -170,8 kNm and earthquake forces => Max = 671,8 kNm, Min = -671,8 kNm

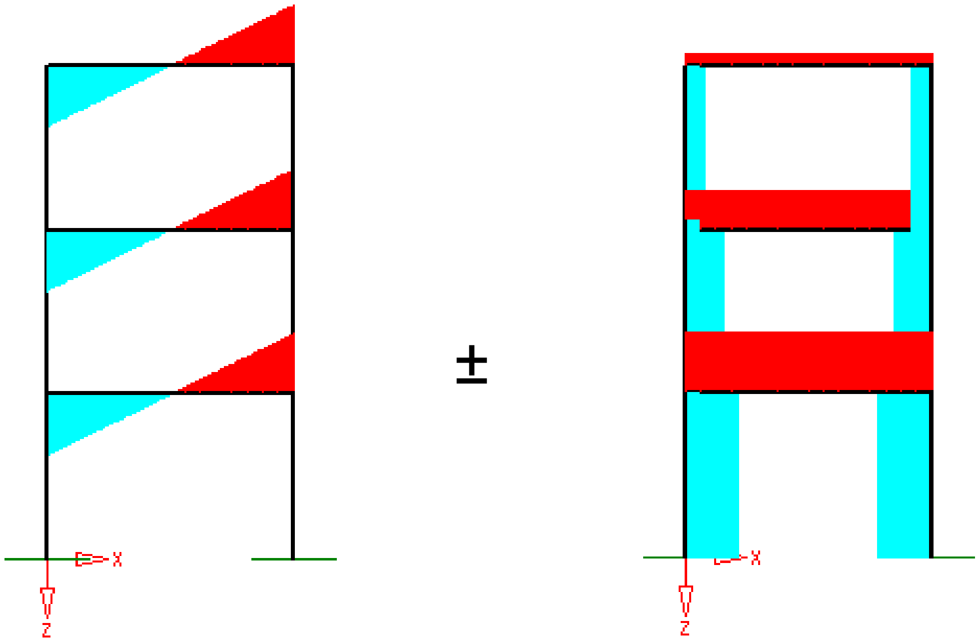


Figure 2.29: Comparison of shear forces due to gravity loads => Max = 1751,1 kN Min = -1751,1 kN and earthquake forces => Max = 224kN Min = -224 kN

The author's presumption of ductility class M for the whole structure demands the following procedure of design for the essential members:

The bending- and shear-resistance of beams has to be designed according to equation (2.38) with A_{ed} based on the so-called design spectrum. The bending- and shear-resistance for columns has to be also designed according to equation (2.38) with an additional implementation of the capacity design method.

It is to keep in mind that – for reasons of a possibly low seismicity – the following combination ("quasi-permanent occurring actions") leads to the proper internal force variables that have to be used to design certain members. In the present case the result of equation (2.39) with a total gravity load of 831,8 kN / m increased by partial safety factors $\gamma_{G,j}$ causes larger values of internal forces for the beams than equation (2.38).

Combination of actions for persistent or transient design situations (fundamental combination):

$$E_d = \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \text{"+"} \gamma_{Q,1} \cdot Q_{k,1} \tag{2.39}$$

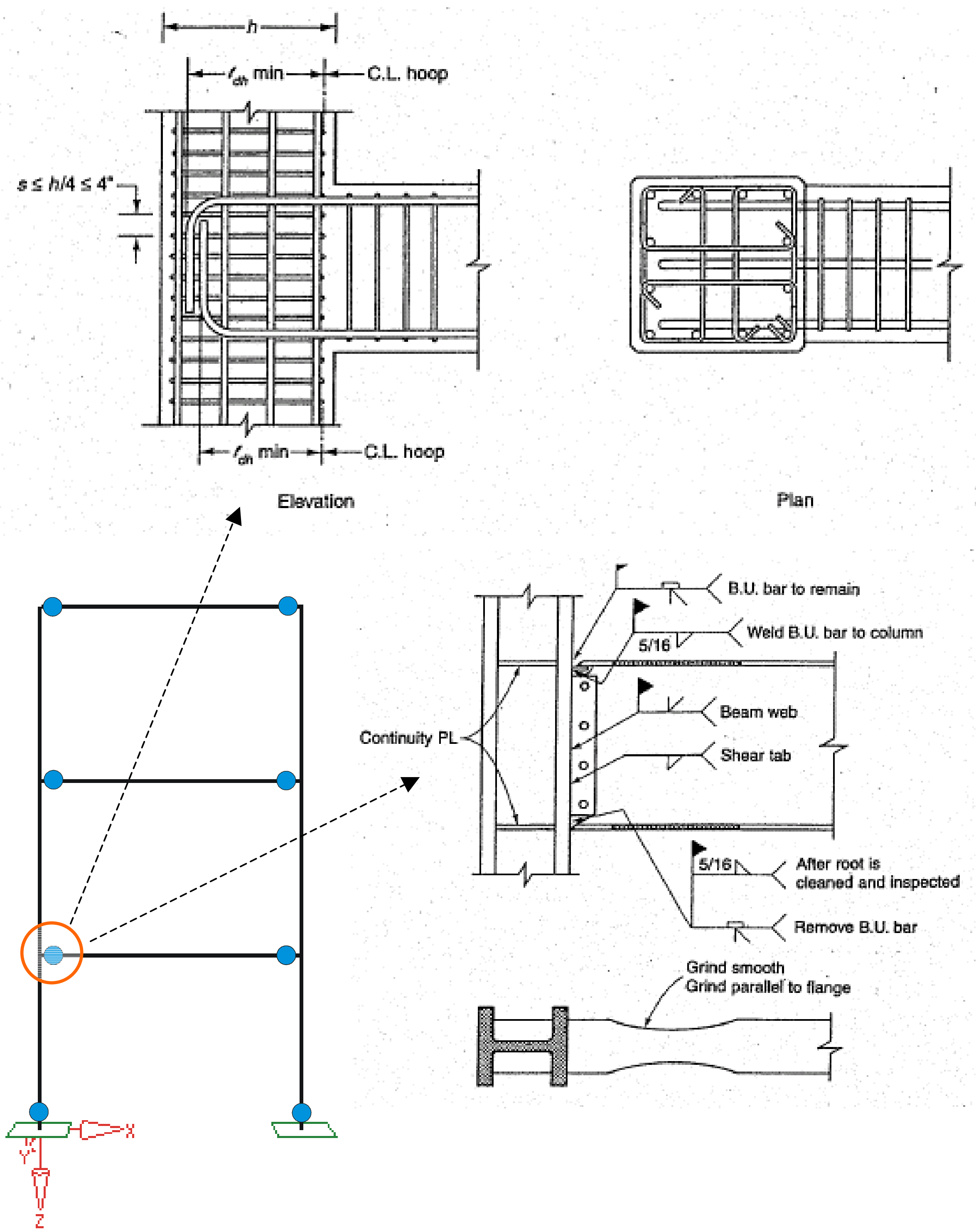


Figure 2.30: Clarification of the Capacity Design Method with Details for Concrete Structures [16] and Steel Structures [16]

2.6.4.5 Conclusive remarks to the whole demonstration

When observing Figure 2.24 the reader will find out, that the visuals for modal analysis are typical ones following the mechanical expectations of a shear resisting frame, as this demonstration traces back to such a structure taken from [13]. For those reasons the stiffness properties of the slabs were determined with higher values than usual.

Nevertheless the author dealed with rules for moment resisting frames in the following. If that concept would have been realized from the very beginning, a marginal decrease of stiffness had been obtained, leading to bigger periods of vibration for the response spectrum and thereby to smaller forces representing seismic action. As that didn't happen, the concept includes a kind of inner reserve because of having been designed for a bit bigger internal force variables.

Inasmuch no mechanical mistake is implemented. As already having been written, the capacity design method with desired plastic hinges localized in the beams is always to be aimed at – as a reaction to experiences made in the past years with accepted column failure mechanisms- for reasons of the structure's global safety and cost effectiveness.

Finally – for reasons of safety again, it is to be kept in mind that redundant structures have the quality to rearrange internal forces. In cases, when the earthquake forces are higher than those due to the design response spectrum, the rearrangement will take place from the intended plastic hinges to the members that must remain their elasticity. This is the reason, why the upper limit of a calculated resistance for these members is based on the elastic response spectrum ignoring the behavior factor **q**.

2.7 Dynamic Time – History Analysis of Buildings due to Seismic Actions (Quick Reference)

2.7.1 Introduction

This chapter discusses the application of the dynamic time-history analysis as an alternative approach to the commonly used techniques in the structural seismic design: lateral force and response spectrum method respectively. First, some theoretical backgrounds and useful techniques for the numerical evaluation of dynamic response are considered. Next, the requirements of the "EUROCODE 8" regarding the practical earthquake resistance design are presented as well. Finally, the entire algorithm is exemplified by means of a three storey reinforced concrete frame.

In general the seismic analysis deals with the prediction of structural response due to earthquake excitation. However, for the designing purpose only the maximum values are required. Thus, using the above mentioned non-time-history methods one obtains the corresponding maximum response levels without investigations of time varying effects. Additionally a linear elastic structural behavior is assumed. In many cases, however, dynamic time history analysis shall be carried out in predicting maximum structural response. One of the important reasons for having to do this is the fact that under maximum probable earthquake conditions most structures experience damage. That means, such structures behave in a nonlinear manner. In other cases, the extreme complexities in the structural geometries cause difficulty in combining modal contributions to response. That requires as well dynamic time history analysis. Modeling, containing critical frequency dependent parameters, is another case.

2.7.2 Numerical techniques used in the dynamic time-history analysis

Analytical solution for the structural response is usually not possible if the excitation varies arbitrary with the time or if the system is nonlinear. For this purpose numerical time-stepping methods for integration of differential equations should be applied. Next, two time-integration techniques are briefly presented:

- **Duhamel's convolution integral**

Consider the equation of motion of a Single-Degree-Of-Freedom (SDOF)

$$m\ddot{w}(t)+c\dot{w}(t)+kw(t)=p(t). \tag{2.40}$$

Next, the analytical solution of this equation is presented subjected to the initial conditions

$$w(0)=0,\ \dot{w}(0)=0. \tag{2.41}$$

In developing the general solution, **p(t)** is represented as a sequence of impulses of infinitesimal duration, and the response of the system with respect to **p(t)** is the sum of the responses to individual impulses. These individual responses can conveniently be written in terms of the response of the system to a unit impulse as shown in Figure 2.31. The response for a viscously damped SDOF system is given by the so called *Duhamel's convolution integral*:

$$w(t)=\frac{1}{m\omega_D}\int_0^t p(\tau)e^{-\zeta\omega_n(t-\tau)}\sin\left[\omega_D(t-\tau)\right]d\tau, \tag{2.42}$$

where $\omega_D=\omega_n\sqrt{1-\zeta^2}$ is the natural circular frequency of the damped vibration, and τ is the time instant, at which an impulse is starting. Note that in this result "at rest" initial conditions, Equation (2.41), are assumed.

The influence of initial displacement and velocity is given by the resulting free vibration response

$$w(t)=e^{-\zeta\omega_n t}\left[w(0)\cos(\omega_D t)+\frac{\dot{w}(0)+\zeta\omega_n w(0)}{\omega_D}\sin(\omega_D t)\right], \tag{2.43}$$

which should be added to Equation (2.42). Note that Duhamel's integral provides a general result for evaluating the response of a linear SDOF system to arbitrary time-varying force.

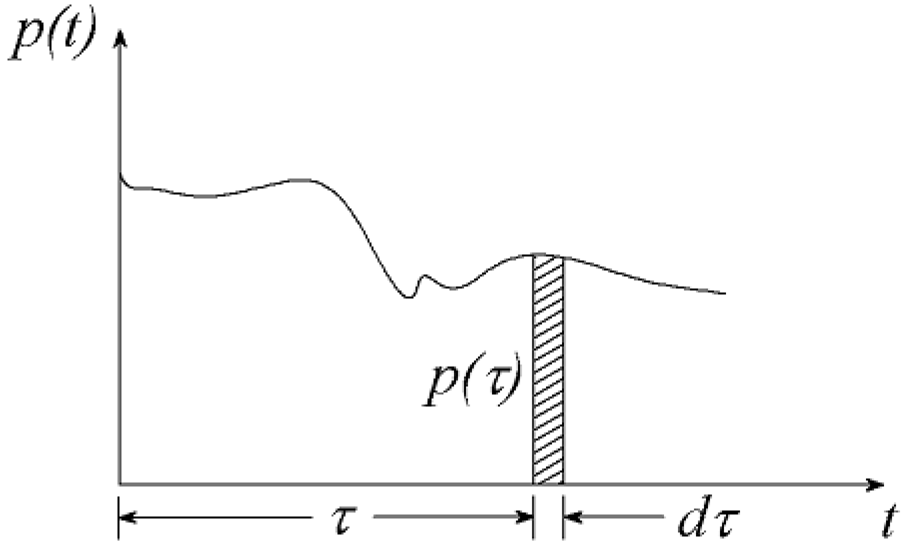


Figure 2.31: Derivation of the Duhamel's integral (undamped)

- **Newmark's method**

In 1959, N. M. Newmark developed a family of time-stepping methods based on the following equations (for MDOF systems):

$$\begin{aligned}
&\underset{\approx}{\mathrm{m}}\ddot{\underset{\sim}{w}}_{n+1} + \underset{\approx}{\mathrm{c}}\dot{\underset{\sim}{w}}_{n+1} + \underset{\approx}{\mathrm{k}}\underset{\sim}{w}_{n+1} = \underset{\sim}{p}_{n+1} \ , \\
&\underset{\sim}{w}_{n+1} = \underset{\sim}{w}_n + \Delta t\dot{\underset{\sim}{w}}_n + \frac{\Delta t^2}{2}\left[\left(1-2\beta\right)\ddot{\underset{\sim}{w}}_n + 2\beta\ddot{\underset{\sim}{w}}_{n+1}\right] \ , \\
&\dot{\underset{\sim}{w}}_{n+1} = \dot{\underset{\sim}{w}}_n + \Delta t\left[\left(1-\gamma\right)\ddot{\underset{\sim}{w}}_n + \gamma\ddot{\underset{\sim}{w}}_{n+1}\right] \ , \\
&\text{where } \ n = 0,\ 1,\ 2,\ \ldots\ .
\end{aligned} \qquad (2.44)$$

The basic assumption in Equation (2.44) is, that $\underset{\sim}{w}_n, \dot{\underset{\sim}{w}}_n, \text{and}\, \ddot{\underset{\sim}{w}}_n$ are known from the previous time-step. Thus, the unknowns $\underset{\sim}{w}_{n+1}, \dot{\underset{\sim}{w}}_{n+1}, \text{and}\, \ddot{\underset{\sim}{w}}_{n+1}$ can be computed at time **i+1**. For non-linear systems iteration is required to implement these computations because the unknown $\ddot{\underset{\sim}{w}}_{n+1}$ appears in the right hand-side of Equation (2.44). The parameters β and γ define the variation of the acceleration over a time step and determine the stability and accuracy characteristics of the method. For $\beta = 1/4$ and $\gamma = 1/2$ the method becomes *unconditionally stable*. In other words, the technique leads to bounded solutions regardless of the time-step length. However, the method is accurate only if the time-step is small enough, e.g. $\Delta t = 0.01 \div 0.02$ sec, which is typical in the earthquake engineering.

2.7.3 Application of dynamic time-history analysis with respect to "EUROCODE 8"

According to "EUROCODE 8" the seismic action may also be represented alternatively by means of ground acceleration time-histories. For this purpose both artificial and recorded or simulated accelerograms may be used.

If artificial accelerograms are used:

- These shall be generated so as to match the elastic response spectrum for 5% viscous damping ($\zeta = 5\%$);
- The duration of the artificial ground motion shall be consistent with the magnitude and the other relevant futures of the seismic event underlying the establishment of a_g;
- The stationary part of the accelerograms should be equal to 10sec;
- A minimum of 3 accelerograms has to be used;
- The mean of the zero period acceleration spectral response values is not smaller than the corresponding value of the elastic response spectrum for the site in question;
- No value of the mean 5% damping elastic spectrum is less than 90% of the corresponding value of the 5% damping elastic response spectrum.

For recorded or simulated accelerograms the following criteria shall be satisfied:

- The samples used for the physical simulation of source and travel mechanisms are adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site. Additionally the accelerogram values shall be scaled to corresponding values for the zone under consideration;
- The suite of recorded or simulated accelerograms to be used shall satisfy the last three requirements for artificial generated accelerograms listed above.

2.7.4 Numerical example

In this section the considerations presented above are visualized by means of an example, namely a 2D three storey frame subjected to an artificial ground motion (Figure 2.32). The structure of consideration is assumed to be made of reinforced concrete (Table 2.16). In accordance with EC8 5% viscous damping is assumed as well. The structural mass is lumped at the cross points of the floor levels with the frame columns. Two dynamic time-history analyses are provided with respect to the structural behavior, i.e. linear elastic versus inelastic. The obtained internal forces, e.g. shears and bending moments, are compared with the corresponding values provided by means of response spectrum analysis.

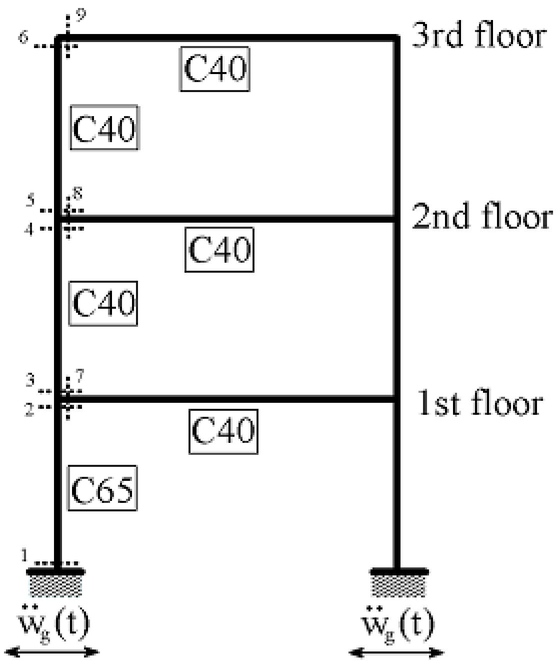


Figure 2.32: Structure under consideration

Using stationary simulation, i.e. superposition of sinus waves, ten artificial motions are generated, but in order to simplify matters, only one artificial record (Figure 2.33) is used in this numerical study. Its displacement, velocity, and acceleration elastic response spectra are depicted in Figure 2.34, whereas Figure 2.35 compares the values of the acceleration response spectrum with the limits specified by “EUROCODE 8”.

	C40	C65	Reinforcement
Young’s Modulus [N/m^2]	35.0E+09	40.0E+09	2.0E+11
Compressive resp. Yield Strength [N/m^2]	4.0E+07	6.5E+07	5.0E+08
Tensile Strength [N/m^2]	3.5E+06	4.5E+06	-
Strain at peak stress	0.0023	0.00265	-

Table 2.16: Material properties for the reinforced concrete

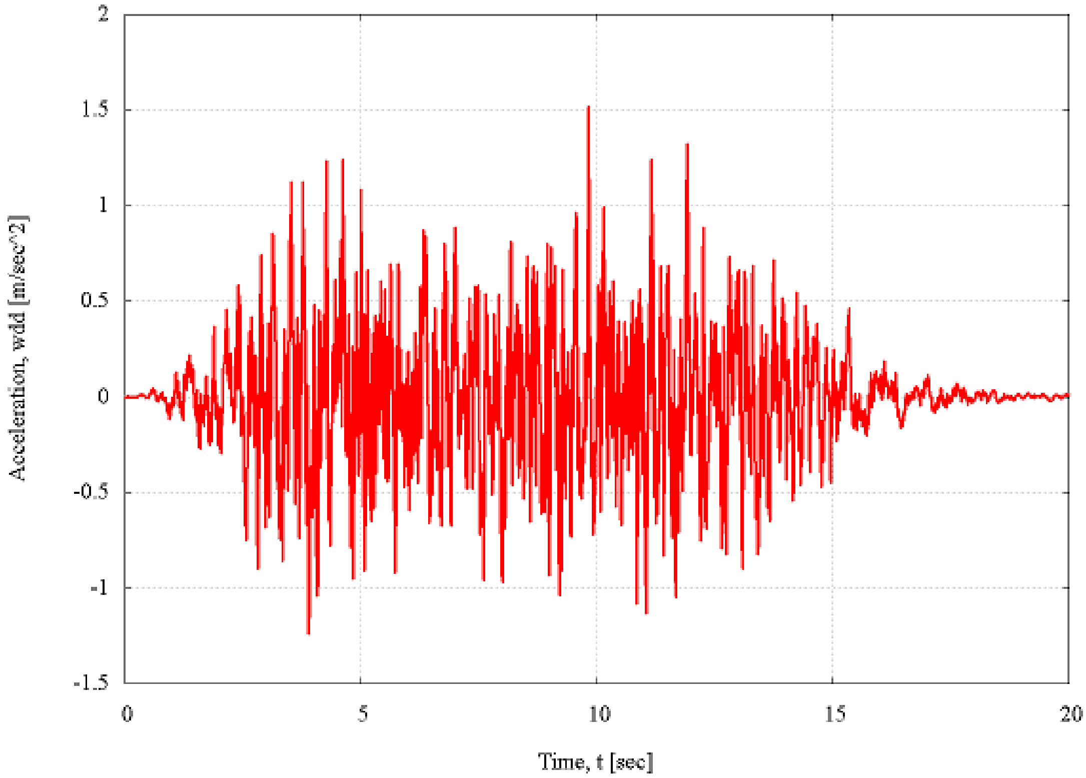


Figure 2.33: Artificial acceleration time-history

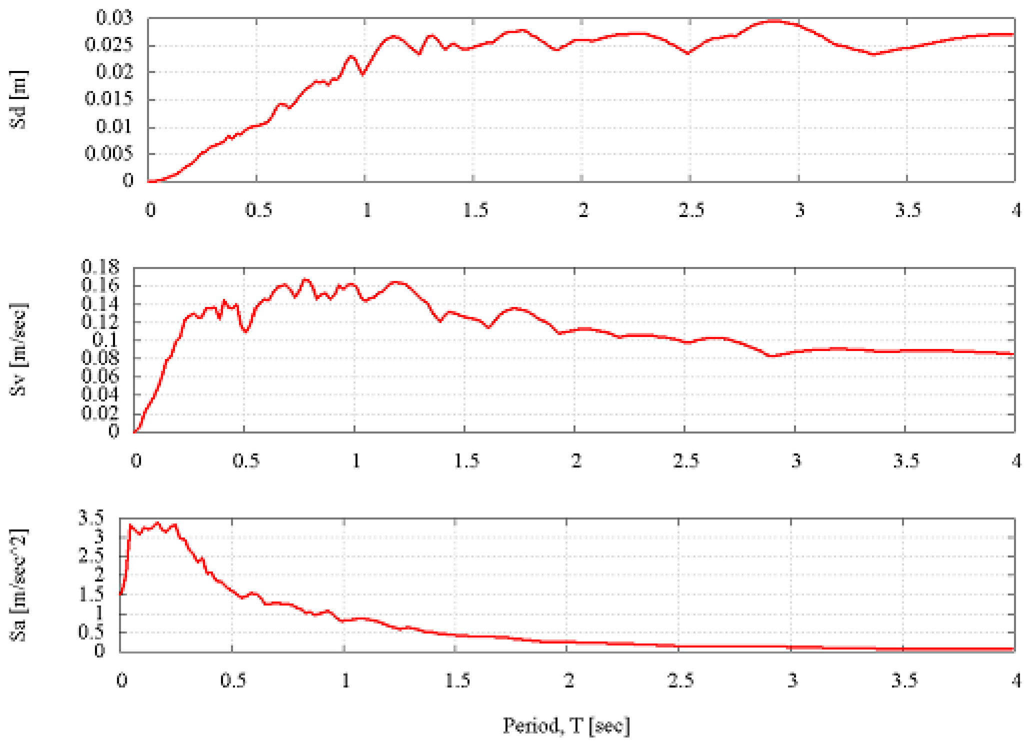


Figure 2.34: Response spectra of the artificial generated seismic action

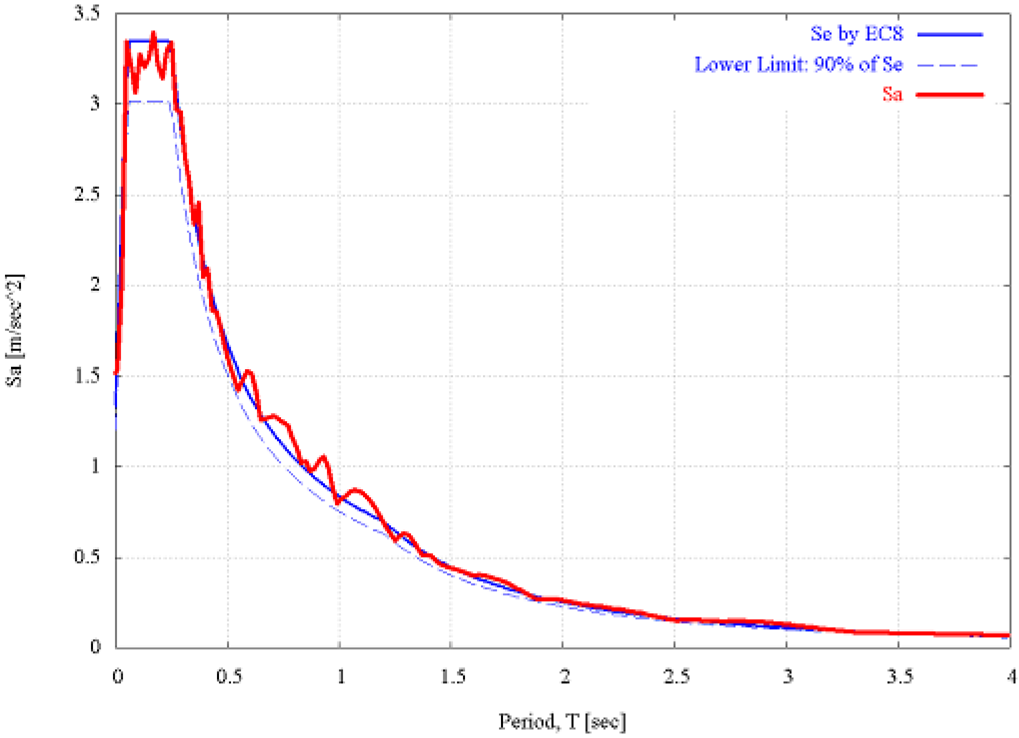


Figure 2.35: Acceleration response spectrum versus EC8 limits

- **Elastic structural response**

In this case linear elastic material properties are assumed. Thus, no reinforcement steel is considered in the numerical model. The obtained structural response for each floor is depicted in Figure 2.36 to Figure 2.39. Next, the corresponding internal forces can be calculated by means of simple static for each discrete time instant. Figure 2.40 and Figure 2.41 show the internal moment and shear force at the discrete point #1 (according to Figure 2.32). Additionally, the values provided by elastic response spectrum analysis are plotted as logical limits to the time-history analysis. As expected, the maximum time-history values are within these limits. Applying the half-power bandwidth technique on the acceleration response of the first floor the structural damping ratio can be evaluated as $\zeta = 0.048 \approx 0.050$.

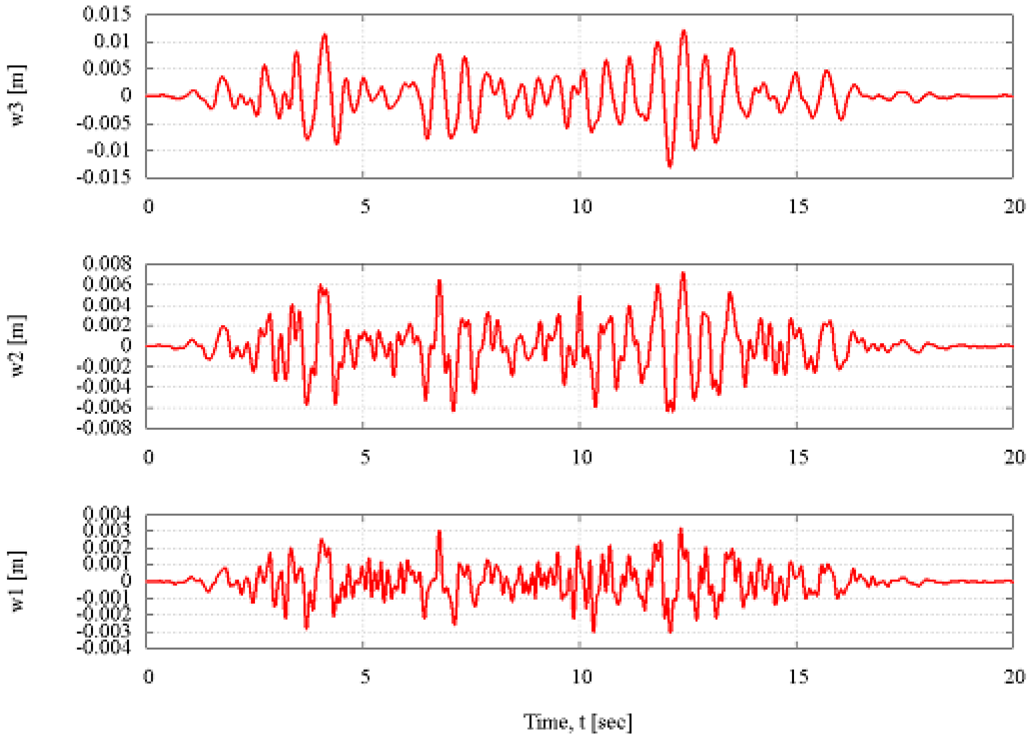


Figure 2.36: Displacement time-history

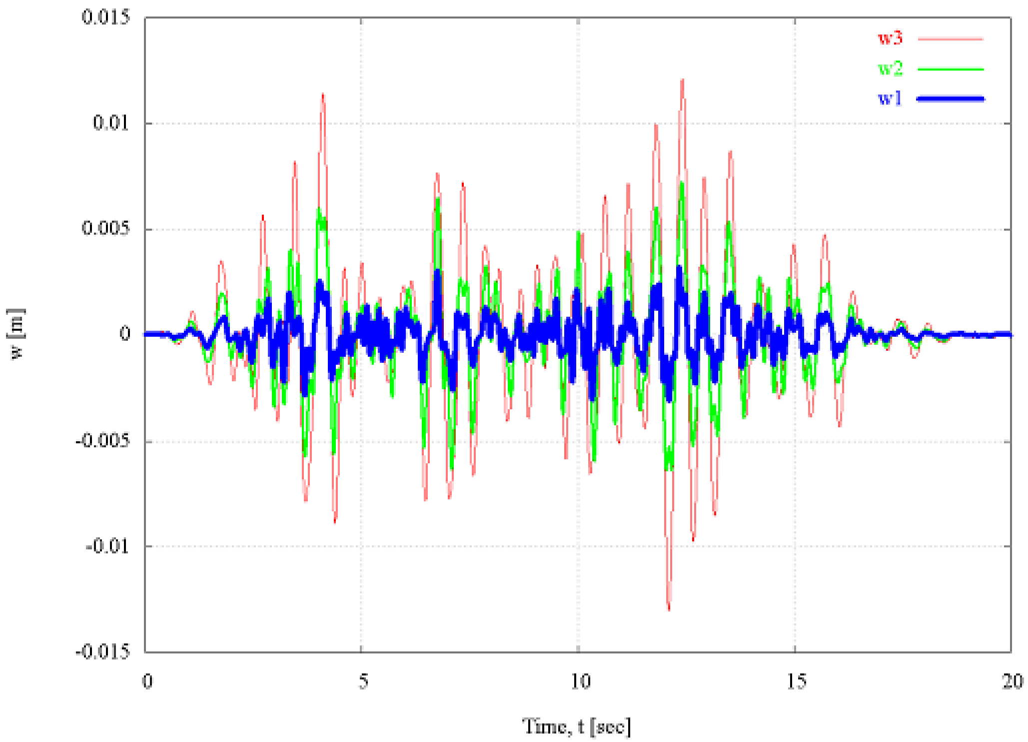


Figure 2.37: Comparison between the floor displacement responses

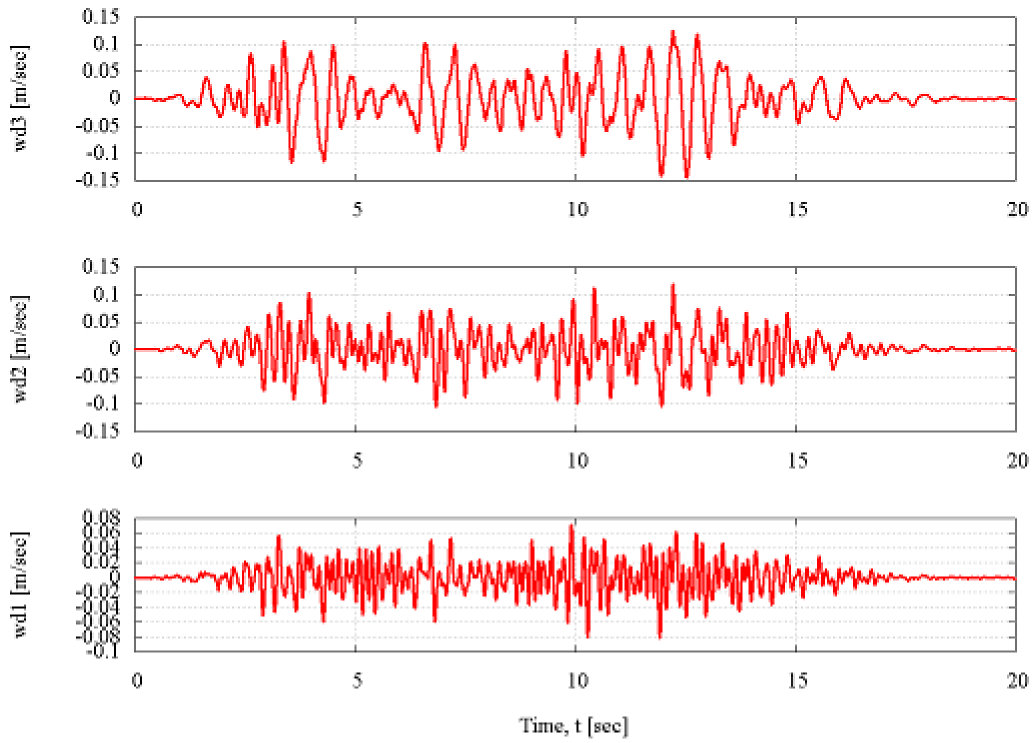


Figure 2.38: Velocity time-history

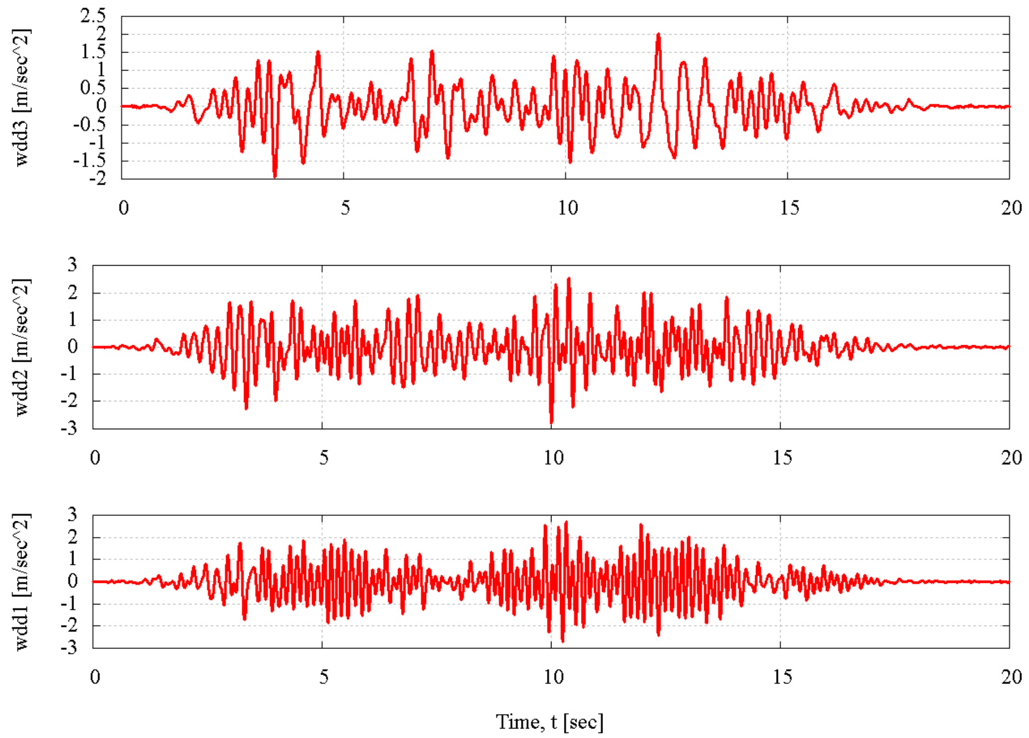


Figure 2.39: Acceleration time-history

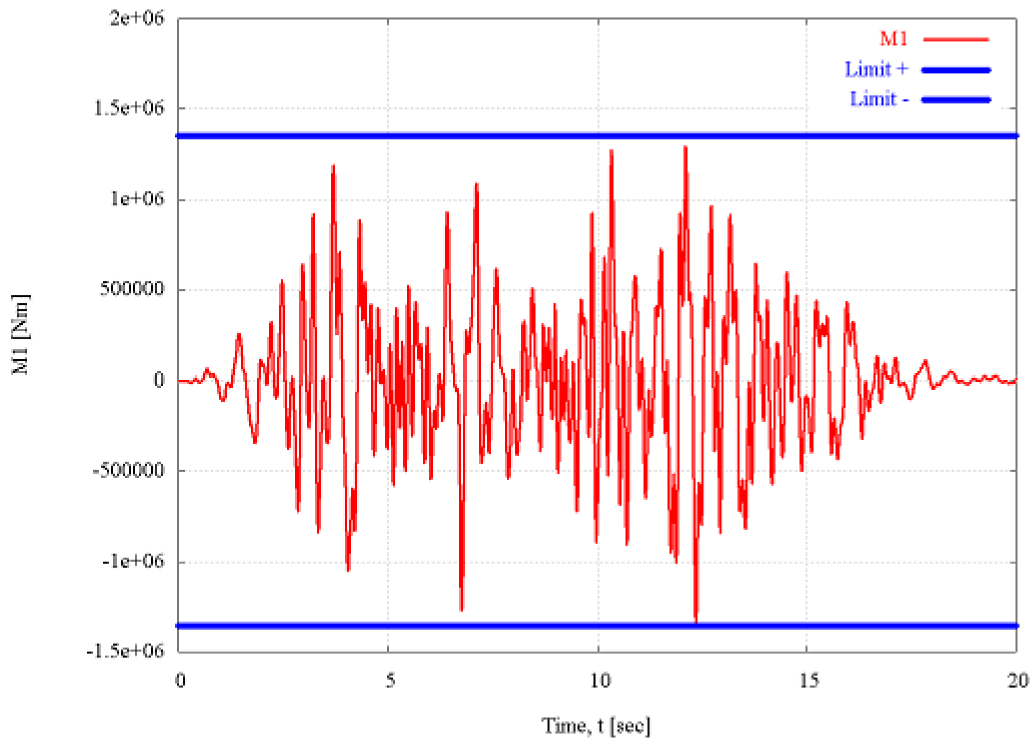


Figure 2.40: Internal moment time-history at discrete point #1

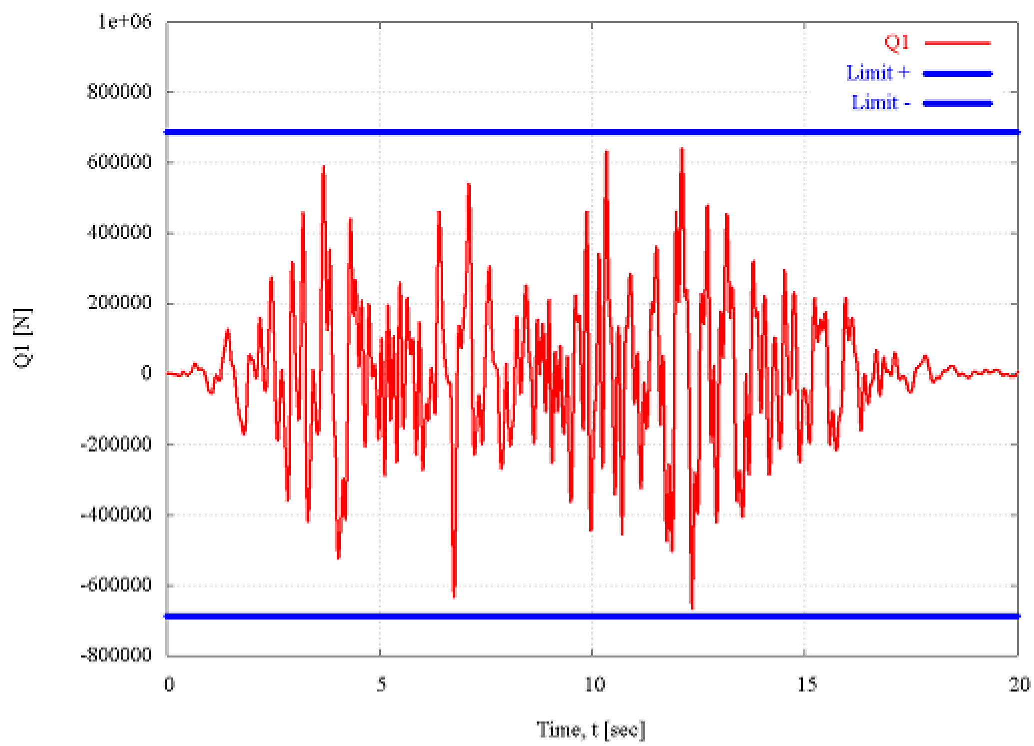


Figure 2.41: Internal shear force time-history at discrete point #1

- **Inelastic structural response**

For the purpose of inelastic study the structure is assumed to be lightly reinforced and nonlinear material models are implemented in the analysis for both concrete and reinforcement steel. Analog to the previous subsection, first the obtained structural displacement, velocity, and acceleration time-histories are illustrated in Figure 2.42 to Figure 2.45, and then the internal moment and shear force respectively are depicted in Figure 2.46 and Figure 2.47. Here it should be mentioned that the values lying outside the limits are caused by redistribution of internal forces. In other words, first of all the concrete at point #4 is cracked (see the first green line in Figure 2.46 and Figure 2.47 respectively), then the remaining columns are cracked one after other. Meanwhile the reinforcement in points #5, #6, #4, and #3 reached its yield strength as well. Finally first the concrete and immediately afterwards the steel in discrete point #1 are damaged and reach the yield strength respectively. As a consequence of this the internal forces become smaller than the limits obtained by plastic response spectrum analysis. The completed damage time-history is listed in Table 2.17. The reinforcement steel stress-strain relation for discrete points #1, #3, and #5 is shown exemplarily in Figure 2.48 to Figure 2.50. Note, the structural damping ratio calculated by means of the half-power bandwidth method has increased to $\zeta = 0.277$. This fact shows the “positive” influence of damage effects on the structural dynamic time-history response, namely increased energy dissipation.

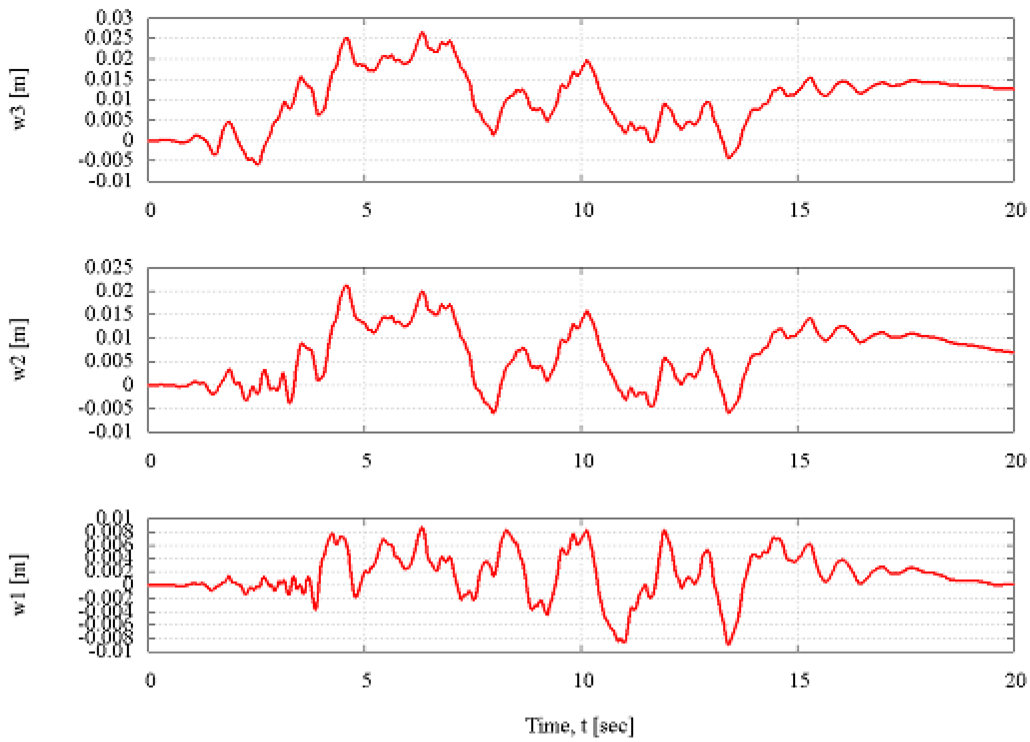


Figure 2.42: Displacement time-history

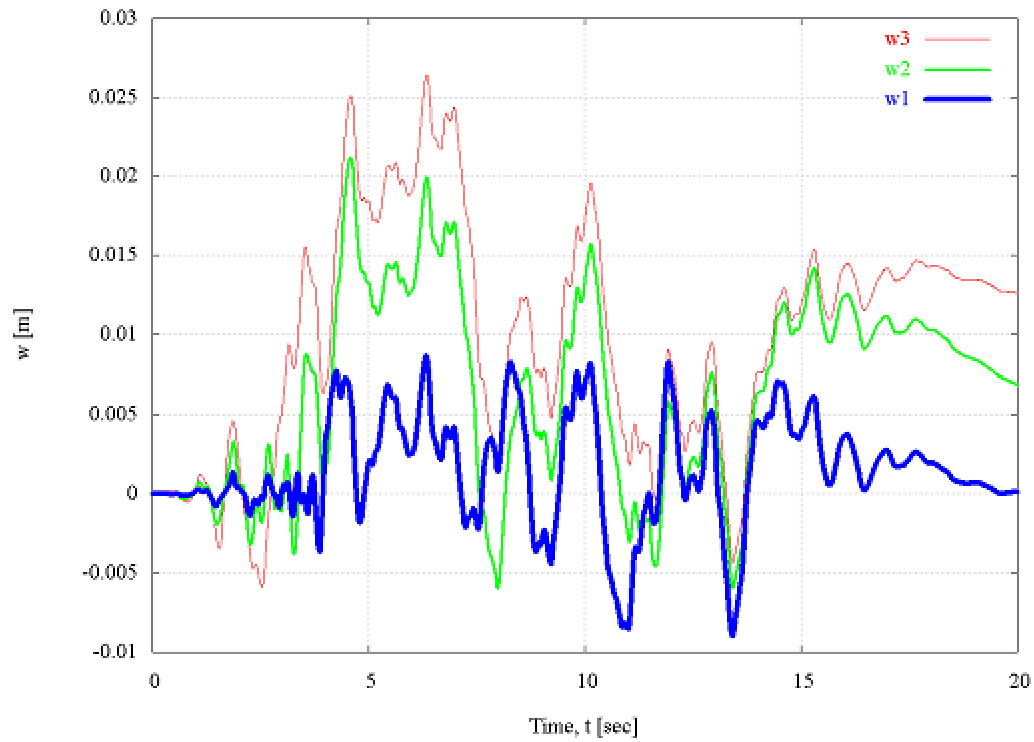


Figure 2.43: Comparison between the floor displacement responses

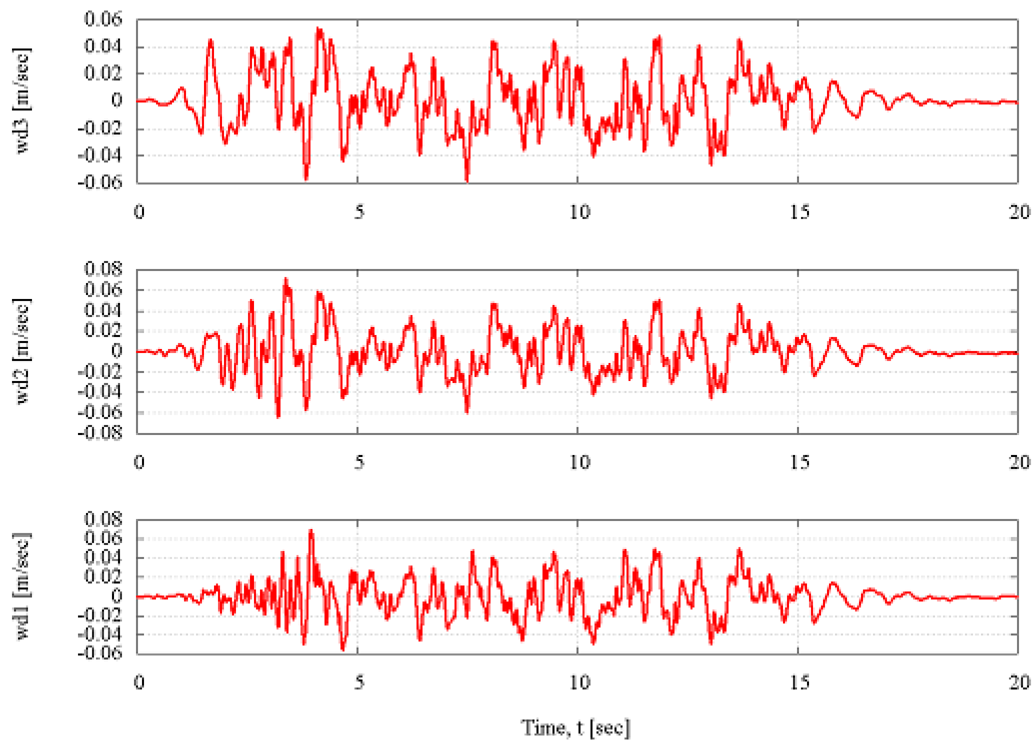


Figure 2.44: Velocity time-history

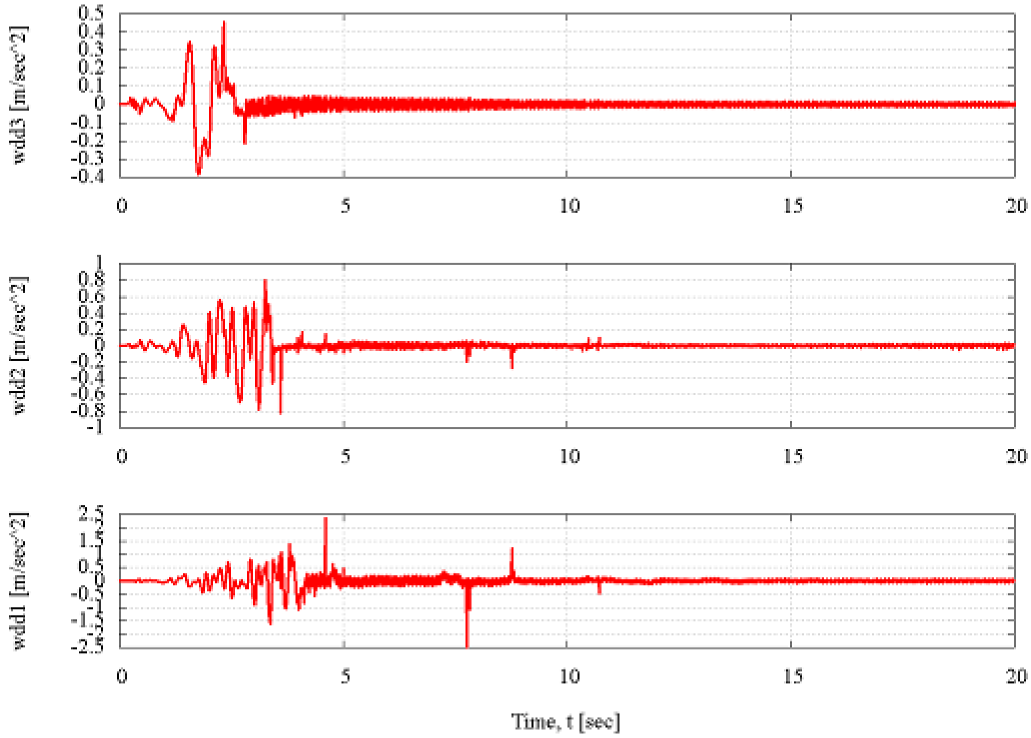


Figure 2.45: Acceleration time-history

Time [sec]	Discrete point number	Reached damage
1.88	4	Concrete cracked
1.89	3	Concrete cracked
2.32	6	Concrete cracked
2.33	5	Concrete cracked
2.88	5	Steel yielded
2.88	6	Steel yielded
3.48	4	Steel yielded
3.53	3	Steel yielded
3.80	1	Concrete cracked
3.83	2	Concrete cracked
4.01	1	Steel yielded
6.30	2	Steel yielded

Table 2.17: Damage time-history

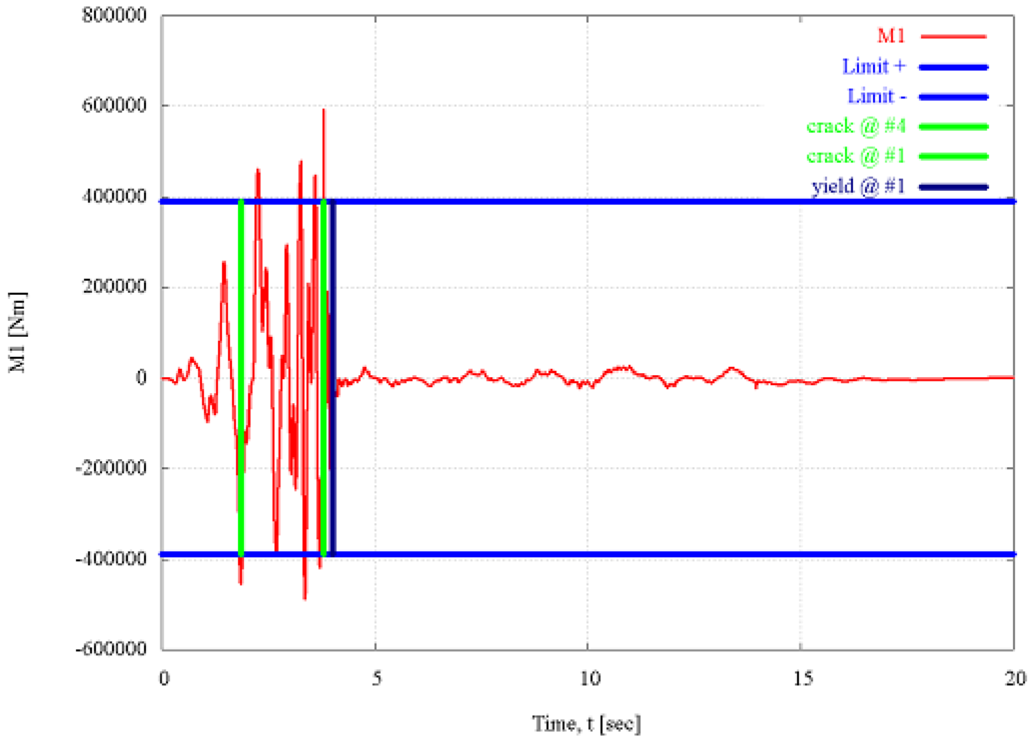


Figure 2.46: Internal moment time-history at discrete point #1

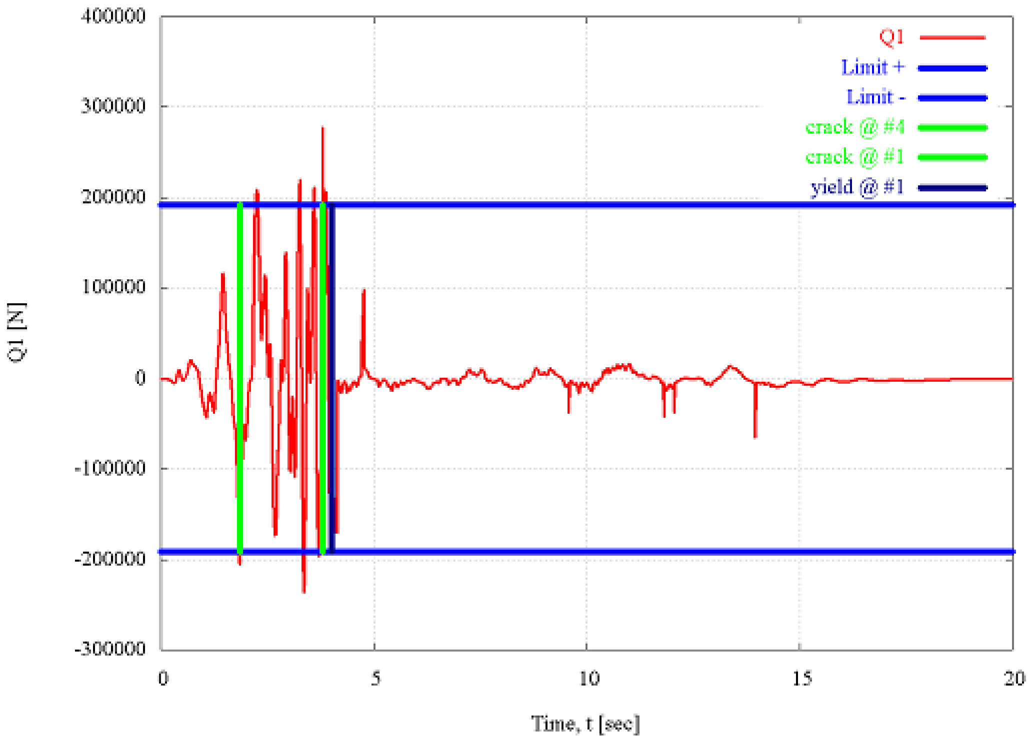


Figure 2.47 Internal shear force time-history at discrete point #1

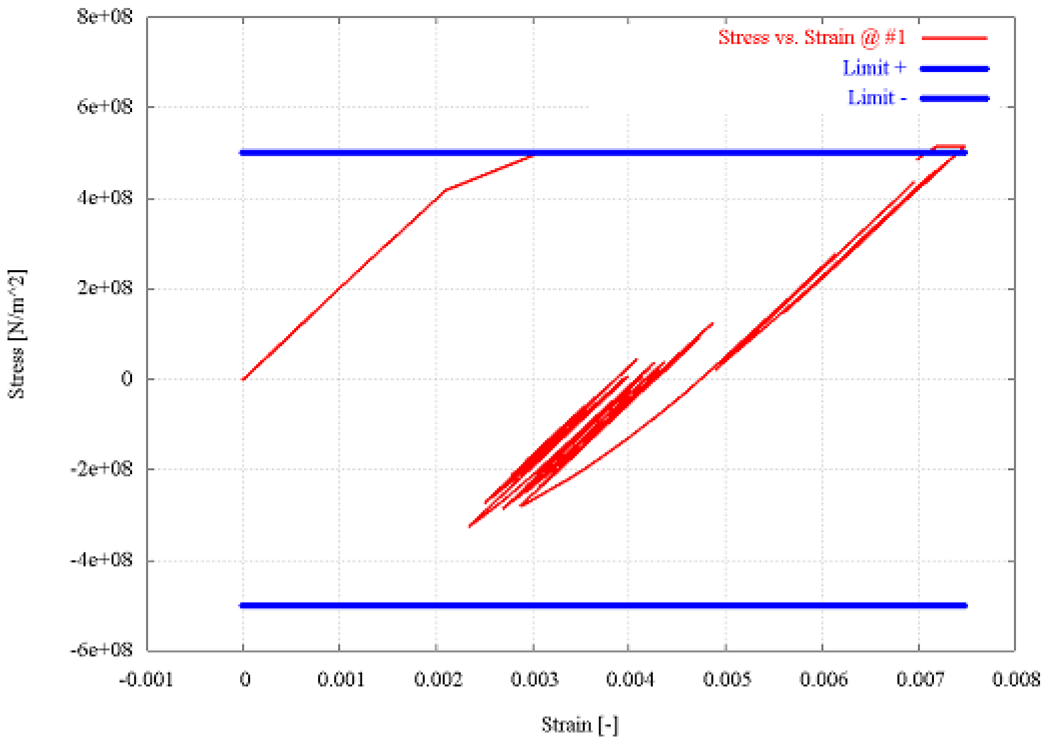


Figure 2.48 Stress-strain relation for reinforcement steel at discrete point #1

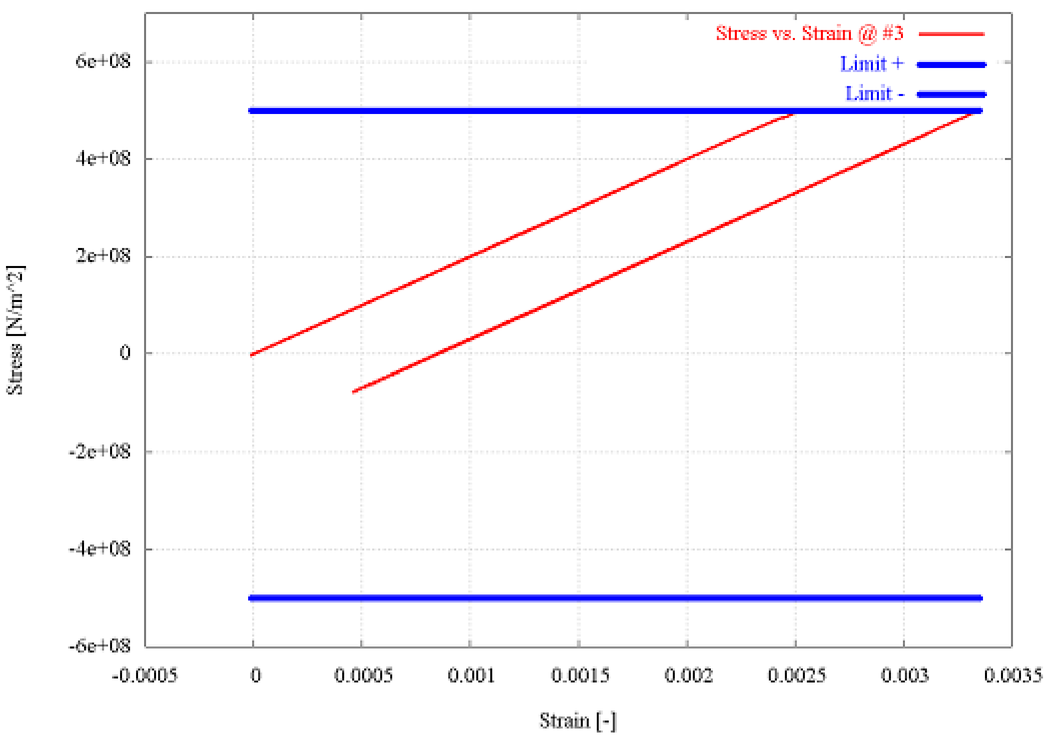


Figure 2.49: Stress-strain relation for reinforcement steel at discrete point #3

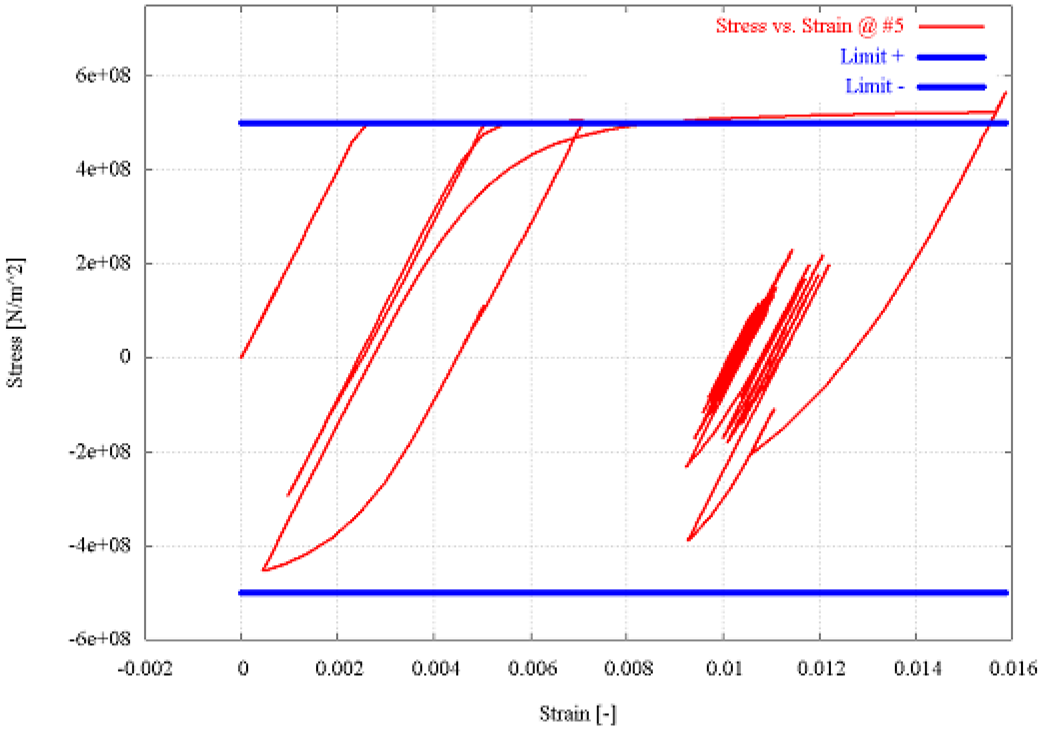


Figure 2.50: Stress-strain relation for reinforcement steel at discrete point #5

2.7.5 Conclusions

The time-history response of a 2D three storey reinforced concrete frame due to ground motion has been considered, in order to represent the seismic resistance design requirements according to “EUROCODE 8”. The support excitation is characterized by an artificial generated accelerogram compatible with the prescribed elastic design response spectrum. Both linear-elastic and inelastic material behaviour are implemented in the analysis. It is shown that the maximal values of the internal forces, e.g. moment and shear force respectively, are approximately smaller than the corresponding internal forces obtained by means of response spectrum method. That should be treated as an evidence of conformity between dynamic time-history and non-time-history techniques used in the seismic resistance design. The option for observation of nonlinear structural behaviour is the general advantage of the dynamic time-history analysis. However, the computational effort depends directly proportional on the complexity and on the number of DOF’s of the investigated structure. Additionally, convergence troubles during the nonlinear iterative procedure should be mentioned as well.

On the other hand the example demonstrates that a realistic behaviour of any structure can be determined broken down to minor structural components. This enables the clear identification of weak points of complex structures. Even so these models are not always stable and converging better approaches are difficult to achieve. This exercise helps considerably in the assessment of structures subject to seismic excitation.

3 INFORMATION RECEIVED FROM THE CZECH SIDE

The following is a compilation of the information received from the Czech side during the presentation in the bilateral meeting in December 2003 in Vienna, from the workshop within the roadmap item 6 “Site Seismicity” in Prag in March 2003, from a visit to the Technical University Prag in May 2004, a visit to Stevenson and Associates in Pilsen in July 2004 and from other sources.

The Austrian Expert Team has concluded the evaluation of the information following the VLI (verifiable line items) in July 2004. The results of the process are given as assessment to each sub-chapter and in the conclusions (chapter 4) and recommendations (chapter 5).

3.1 Legal Framework for seismic Design and Evaluation

3.1.1 Standards used at the Time of Design

Basic design and dimensioning has been carried out by Russian engineers applying their SNIP codes [61]. The basic layout of the plant is based on Russian applications, which were constructed in seismic active zones such as Georgia and Armenia. The seismic capacity has not particularly been downgraded for the location of Temelin. This creates the impression that the plant might be seismically over designed during the basic design process. A quick but critical look at the structures confirms that sufficient resistance in global terms exists. Nevertheless this does not replace a detailed consideration of seismic loads. The current trend is to go away from global approaches to site specific design methodologies.

3.1.2 Seismic Re-evaluation

A project on seismic qualification of civil structures has been carried out by Stevenson and Associates based on the following codes and standards:

- Czech legislation, act nr. 18/1997
- Degrees of the state office for nuclear safety
- IAEA recommendation 50-SG-S1, earthquakes and associated topics in relation to NPP design and evaluation
- IAEA recommendation 50-SG-D15, seismic design and qualification for NPP design
- Structural elements have been evaluated according to Czech standards to use limit state design methods (LRFD). The relevant codes have been ČSN731201 for design of concrete structures and ČSN731401 for design of steel structures
- A list of other relevant applicable codes and standards for relevant seismic acceptance criteria is provided in the POSAR (chapters 3.7 and 3.8)

Seismic qualification was carried out by Stevenson and Associates from Pilsen. They issued a document called “Detailed methodology for seismic qualification of civil structures and technological equipment” [31], which is available in Czech language only. It includes the requirements for seismic analysis and evaluation of seismic resistance of civil engineering structures and equipment installed on Temelin NPP including rules for their elaboration. A translation into English or German is not available.

3.1.3 Application of Eurocode 8

EC 8 has been in preparation at the time the seismic qualification has been performed. It has not been used for re-evaluation. After endorsement of EC 8 no new or additional re-evaluation attempt has been taken.

Assessment of the Austrian Expert Team

There is not enough information to assess the quality of the basic design carried out by Russian engineers. Anyhow the usual practice was to rather over design structures because of missing economic pressure. It can be anticipated that this has been the case also for the civil structures of NPP Temelin. Nevertheless the applied methodologies did not ask for sophisticated assessment on the performance of structures. It can be assumed that no attention has been given to displacements, performance and interface design.

The seismic re-evaluation has been performed in the light of an existing seismic over design. No need for special and deep elaboration has been identified and the applicable codes at this time, which ask for a static equivalent horizontal force of 0,1g [22] have been assessed to be sufficient. This process does not identify eventual problems with the seismic performance of the structure.

The new knowledge and approaches developed during the recent years have not been appreciated. The assessment process of civil structures and components has been completed and never taken up again after the establishment of new seismic evaluation approaches, considering longer return periods, high pga values and probabilistic methodologies.

The expert assessment of this subject is that the qualification process has been performed according to the existing rules and standards, but missed to be adapted to the state of the art and current practice subsequently. It is obvious that the old practice did not produce any results of concern, which might be expected applying the current practice (for detail refer to chapter 3.5).

3.2 Definition of the seismic Input for seismic Design

This chapter is closely related to site seismicity, which is discussed within project PN6. The assessment of the site seismicity based on seismological and geological investigations provides the input for the seismic qualification process.

3.2.1 Ground Spectra and Accelerograms

The IAEA safety guide 50-SG-S1, revision 1 (1992) [22] provides that the horizontal peak ground acceleration shall be applied with 0,10g for SL2 (SSE). This represents the extreme ground motion level with a probability of exceedance of 1000 years. The vertical ground acceleration is proposed to be 2/3 of the horizontal value, with 0,07g applied. These values correspond approximately to 7° MSK-64.

The design earthquake ground motion level with probability of exceedance within 100 years (SL1 (OBE)) has been determined by Stevenson and Associates with 50% of PGA SL2. This results in a $PGA_{SL,1hor}$ = 0,05g and $PGA_{SL,1vert}$ = 0,035g. These values correspond approximately to 6° MSK-64.

For the ground response spectra 2 approaches have been chosen:

- An envelop calculated from a set of several natural accelerograms recorded in locations with similar geological and seismological conditions, which was scaled to PGA_{hor} = 0,10g and PGA_{vert} = 0,07g
- The standard broadband NUREG/CR-0098 (median + 1 sigma) spectrum for rock sites has been applied

The SL1 ground spectra (GRS) are of the same shape as for SL2, however multiplied by a factor of 0,5. The accelerograms were generated using the program SPECTRA and the requirements regarding the minimum PSD, as presented in the U.S. NRC standard review plan (section 3.7.1, Appendix A) [32], were respected. The rise time was 5 seconds, the duration of strong motion 15 sec. and the descent time again 5 sec. These time periods were assessed to be very conservative.

3.2.2 Floor Response Spectra

For the determination of the various floor response spectra a classical time history analysis approach has been applied. The following conditions and parameter have been considered:

- A calculation was based on a 3dimensional finite element model (3D FEM)
- The 3 space components have been applied simultaneously (refer also chapter 3.2.1)
- The time history analysis has been performed with the individual sets of natural accelerograms as calculated (refer chapter 2.1)
- A horizontal component interchange with respect to the main axis of the structure has been performed (no details available)
- An appropriate element mesh has been selected for the calculation to cover all possible bending and torsional modes
- The calculation has been performed for damping of 1, 2, 3, 4, 5, 7 and 10%
- Peak broadening and smoothing according to R.G.1.22 has been applied

The documentation provided shows floor spectra for the reactor building at level 13,20 m and 28,80 m. The information contains a floor spectrum for horizontal and vertical direction. No reference is provided to the 2nd horizontal direction. Furthermore the diesel generator pumping and compressor station have been demonstrated with 3 floor spectra at the diesel generator foundation. In this case both horizontal and the vertical component have been provided.

3.2.3 Transfer Functions

According to the concerned engineers no transfer function analysis has been performed for the plant. The results of the implemented monitoring system, using the 5 registered small earthquakes, have not been used for such purpose.

Austrian Expert Assessment

The chosen approach represents the traditional methodology for seismic re-evaluation. The many options current practice offers have not been exploited. The ground response spectra as presented under PN 6 has been down scaled to a maximum of 0,10 g, which represents a questionable approach. It is to be expected that, under the very generic approach of generating time histories, calculations would not produce higher strain in the structure than a classical quasi static approach (refer to the comparison in table 2.16). Nevertheless it has to be admitted that the presented procedure conforms to the practice existent at the time of the performance of the seismic re-evaluation. At this time transfer functions have not been applied in practice and the use of real data from monitoring systems has not been considered.

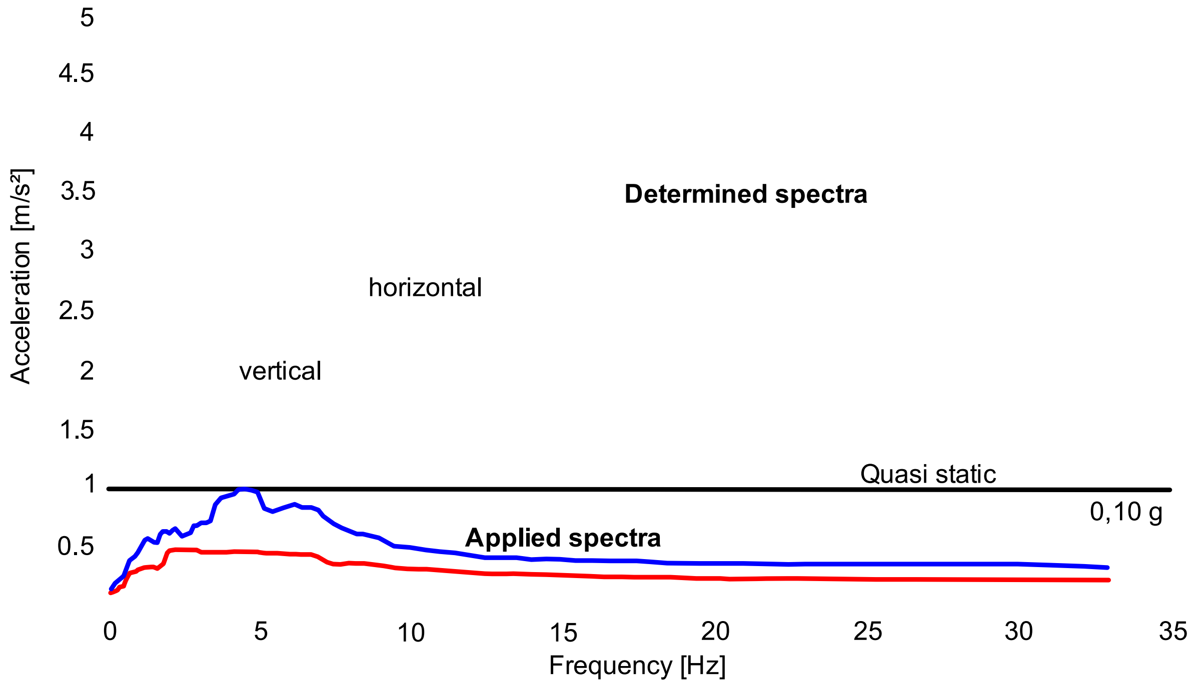


Figure 3.1 Applied spectra for seismic re-evaluation

The conclusion is that considering today's knowledge the approach would have been different. Without the performance of detailed calculations it is not possible to say whether the new approach would produce unfavourable results. Experience showed that in global respect the results are normally less severe than under traditional approach, but there are local problems detected which could not have been identified with the traditional approach.

Beside the definition of the reasonable design inputs the rules of application of these parameters have to be chosen. Seismic calculations include a chain of assumptions, which produce results depending on the tendency of the assumed values. A chain of very optimistic values will definitely produce results that will not point out any of the possible problematic areas in the structure.

3.3 Re-evaluation Methodology

3.3.1 Selection of the Evaluation Process

The detailed methodology for seismic qualification of civil structures and technological equipment has been elaborated by Stevenson and Associates from Pilsen in a 1st edition 1992 and a last valid revision of 4/1996. This methodology is the combination of the IAEA safety guides 50/SG-S1 and D15 in combination with the U.S. NUREG and local Czech standards. It comprises seismic analysis with combination of modal and space seismic responses in the following way:

- Static equivalent method was traditionally performed and
- Response spectrum modal analysis method with defined combination rules was applied

The applied rules are shown in the following table.

Methods of Analysis	Applicable Combination Rules
Response Spectrum Modal Analysis Method (RSMAM)	(a) Combination of Modal Responses • SRSS if the nearby frequencies are far to each other • ABS if the nearby frequencies are close to each other • CQC regardless to the distance between frequencies • Missing mass effect is considered when the cumulative effective mass is less than 80% in any direction and for 33Hz
	(b) Combination of Space Response Components • SRSS or, alternatively • The rule known as ± (100% ± 40% ± 40%)
Equivalent Static Method (ESM)	Combination of Space Response Components • SRSS or, alternatively • The rule known as ± (100% ± 40% ± 40%)

Table 3.1 Applied rules for seismic analysis

3.3.2 Seismic Modelling

A seismic model has been implemented with the use of finite element models (FEM) under consideration of above mentioned combination rules. The models have been used as well for the static equivalent method as the response spectrum modal analysis method. Only a single model per structure has been applied satisfying both approaches.

3.3.3 Interpretation of Results

In absence of the original design capacities for structures and components, the results of the seismic calculation have been used to perform capacity checks. According to the information of the engineers of Stevenson and Associates the capacity existing was always considerably higher than the demand calculated from the seismic analysis. This had influence on the depth of calculations and the number of items checked. The general impression appeared that the seismic capacity is by far over designed for the NPP Temelin.

Assessment of the Austrian Expert Team

The seismic re-evaluation has been carried by the stigma that the plant is considerably seismically overdesigned. Therefore no attempt has been made to enter very deep into the subject. The methods applied represent the practice at the time of performance. Nevertheless the selection of parameters, approaches and necessary assumptions do not show any attempt to detect any hidden problems.

As far as the seismic modelling is concerned the selections of the finite element mesh have been very global in size, which does not allow detecting of any local problems. A mesh with at least 10 times the number of elements would be necessary to perform this task properly.

3.4 Identification of critical Structures, Interfaces and Components

Beside the categorisation of civil structures, which is done based on the function of the structure, critical items have to be identified, which are mostly less obvious. An earthquake produces a performance of a plant, which can be entirely different from normal operation. To detect and quantify these differences requires engineering judgement.

3.4.1 Classification of Structures

A list with the seismic categorisation of civil structures has been presented by the Czech side. This list specifies the required seismic resistance for each building.

Building No.	Description	Required Seismic Resistance
800/01-06	Reactor building	SL – 2
803/01, 03	Reactor building ventilation stack	SL – 2
807/01, 02	Air pressure vessels foundations	SL – 2
586/01-03	Cooling pools with spraying systems	SL – 2
586/4	Switchboards for technical water systems	SL – 2
588	Ducts for technical service water systems piping	SL – 2
594/01	Water treatment for technical service water systems	SL – 2
442/01-03	Diesel generators, pumping and compressor stations	SL – 2
445/01-03	Diesel oil handling	SL – 2
350	Cable ducts	SL – 2
352/02	SMS sensor building	SL – 2
801/01	Auxiliary building (fresh fuel assemblies storage)	SL – 1
801/01	Auxiliary building (wardrobes and laboratories)	SL – 1
801/03	Auxiliary building (RA media treatment station)	SL – 1
803/02	Auxiliary building ventilation stack	SL – 1

Table 3.2 Seismic categorization of civil structures

This list is not the complete list of structures. Not listed are for example the cooling towers and the water intake structure.

3.4.2 Critical Interfaces

A non linear time history analysis has been performed, which has been limited to assess the structural capacity. No attempt has been made to apply these methodologies to the existing interfaces between structures and components.

3.4.3 Non structural Considerations

At the time of the performance of the seismic re-evaluation the performance of non structural components has not been assessed under the prevailing practice.

3.4.4 Components

The effect of seismic actions on components has been considered during a detailed walk down of the plant performed by Stevenson and Associates in combination with equipment qualification activities.

Assessment of the Austrian Expert Team

Studies performed after recent earthquakes showed that more than 80% of damages are caused by the performance of non structural elements. The attention during re-evaluation has to be shifted from the structures to interfaces and non structural components consequently. This step is missing here. The existing codes and standards are satisfied, but not the latest development and knowledge in this sector. Any subsequent re-evaluation has to apply performance based design principles.

3.5 Results of the Re-Evaluation Process

Finally the consequences from all the assessment and calculation work carried out are important. The results of the numerical process have to be interpreted into subsequent actions.

3.5.1 Effect on the Containment

The results of the seismic re-evaluation showed that there is no effect on the containment that would require any intervention.

3.5.2 Effect on other Structures

It has been found that all structures are seismically qualified to sustain the assumed seismic load. No consequences or upgrade measured have been identified.

3.5.3 Effect on Interfaces

This chapter has not been opened at all and therefore no consequences could arise.

3.5.4 Effect on Components

During the qualification of civil structures and components Stevenson and Associates performed a detailed walk down of the plant. At this occasion a number of short comings has been identified. The necessary retrofit measures have been proposed and implemented.

Assessment of the Austrian Expert Team

Considering the approach and the assumptions taken, it had to be expected that there would be no consequences from the re-evaluation. Considering longer return periods of 100000 years, which would result in at least an increase of the magnitude of 0,5, consequences have to be expected. Nevertheless the seriousness of these consequences depends on the actual seismic capacity of the structures, which is currently not known. Should the ground acceleration values rise from the current 0,10g to 0,20g and under the application of state of the art methods and the use of real data, it is expected that no serious consequences would arise for the civil structures. Consequences have to be expected at the interfaces, where unfavourable phase shifts might produce differential displacements which are above the capacities. Nevertheless a detailed assessment which item or location within the plant might be affected can not be done without a major effort in calculation and measurement.

3.6 Implementation of seismic Upgrade Measures

This chapter comprises the actual activities performed after the seismic re-evaluation.

3.6.1 Containment and primary Circuit

No activities were considered to be necessary.

3.6.2 Other Structures

No activities were considered to be necessary.

3.6.3 Other Measures taken

Improvements on the component level have been introduced after the walk down performed by Stevenson and Associates.

3.6.4 Use of the Monitoring System

The installed monitoring system complies with the recommendations of IAEA. It consists of a number of accelerometers that record the ground as well as selected floor accelerations. It is only used to indicate to the operator in case that a seismic event occurs. No analysis of ambient data or of micro events registered is performed. The monitoring network operated by the University of Brno, which has registered such events, is not connected to the internal system. It has not been considered to use the monitoring system for the seismic re-assessment.

3.6.5 Improvement of the Database

Raw data from the internal monitoring system are not collected. The external monitoring system has started to erect a database on events. Nevertheless the operation time is too short to draw conclusions. It is intended to use the results of the monitoring systems in future.

Assessment of the Austrian Expert Team

Considering the results of the calculations it becomes obvious that no upgrade measures have been taken. They have not been considered to be necessary. With respect to the civil structures the Austrian expert team can agree to this approach. With respect to interfaces and non structural components it has to be strongly recommended to perform analyses on this subject. It is considered to be of higher importance than the civil structures itself.

The technology for a proper use of the data of the monitoring system is not available. Nevertheless these methodologies are also not common practice in western nuclear power plants. They represent a new technology which will be integrated in future processes. Nevertheless an application for such an important issue should be considered.

It is not expected that the data collection will produce new knowledge within reasonable time.

3.7 Evaluation of the Information provided

This chapter shall summarize the impression the Austrian expert team got after the presentation from the Czech side and the collected information.

3.7.1 Evaluation of the Information in terms of consistency

All information received from the Czech side is consistent. It also conforms to information received on PN 6 site seismicity.

3.7.2 Evaluation of the Information in terms of Completeness

The information received from the Czech side appears to be complete. The minor issues identified, such as a statement on the cooling towers, are considered not to be withheld, but rather lost in the limitation of time and topics of the presentation and discussion. It can be considered that the complete information has been received.

3.7.3 Evaluation of the Information in Relation to current Practice

As described in previous chapters the re-evaluation process followed the codes and standards and current practice of the time it has been performed. This has been before the recent major earthquakes in Turkey and Taiwan, which led to a considerable change in philosophy. Therefore the provided information represents the current practice prevailing at the time of elaboration.

Most of the material worked out could be used under application of current practice methodologies. Most of the results could be simply factored or reproduced with the existing models. Elements not touched yet such as interfaces and non structural components, have to be introduced into the process on a completely new basis. In the absence of existing codes and standards this has to be done based on the new rules discussed in the respective engineering community globally.

3.7.4 Evaluation of the Information in Relation to Codes and Standards

The information provided conforms to the codes and standards prevailing at the time of performance. The new safety guidelines issues by IAEA in 2003 only provide an indication on new rules, without a firm definition. Reconsideration of seismic re-evaluation is done in many countries based on the recent earthquake history. It therefore can be assessed that the re-evaluation performed has been sufficient for the state of the art at the time, but should be reconsidered in the light of the new knowledge.

3.7.5 Typical Western Approach

The question what we would have done if we were to perform a seismic re-evaluation in 1996 has to be answered with: We would have done a very similar application with most probably reaching the same results.

Nevertheless our seismic design approach has changed considerably in the year 2000, when the conclusions have been drawn from the Kozaeli und ChiChi earthquakes in 1999. Nowadays seismic design has doubled in extents, resulting in safer structures. The consequences are not so much in terms of additional quantities necessary, but in terms of distribution of the quantities to enhance the seismic performance. There is no structure right now, which is seismically re-assessed, that does not bare any consequences such as upgrade measures. These consequences are in most of the cases not of the very expensive type and can easily be implemented. It is therefore recommended to re-open the re-evaluation chapter and take a sorrow re-consideration.

4 CONCLUSION ACCORDING TO THE VLIS

This chapter summarises the Austrian Expert Team evaluation according to the verifiable line items (see annex A) which are intended to be verified by the team in the frame of this project. Details of the items are provided under chapter 3. This chapter concludes the details and indicates remaining issues and follows up actions required for satisfactory monitoring.

For the assessment and monitoring of the seismic design for the Temelin NPP it is necessary to look into the following subjects:

VLIs – Austrian Expert Team View
1. Legal Framework for Seismic Design and Evaluation
The seismic re-evaluation has been performed in the light of an existing seismic over design, based on the legal framework in force at the time of re-evaluation. This process did not identify eventual problems with the seismic performance of the structures. The application of the current practice might produce results of concern in structural details and particular components and interfaces.
2. Definition of the Seismic Input for Seismic Design
The chosen approach represents the traditional methodology for seismic re-evaluation. The many options current practice offers have not been exploited. The presented procedure conforms to the practice existing at the time of performance. It might be expected that under the application of the current practice problems in details would be detected. **Methodology** The seismic re-evaluation has been carried out by the stigma that the plant is considerably seismically over designed. The methods applied represent practice at the time of performance, but the selection of parameters, approaches and necessary assumptions do not show any attempt to detect hidden problems. A reconsideration using the current practice is recommended.
3. Identification of critical Structures, Interfaces and Components
Critical structures have been identified, but no attempt has been made to identify interfaces and critical components. The existing codes and standards are satisfied but not the latest development and knowledge in this sector. A subsequent re-evaluation has to be proposed based on performance based design principles.
4. Results of the Re-Evaluation Process
The re-evaluation process provided that no consequences were necessary for the plant. This is the expected result under the taken assumptions. Considering longer return periods producing a different seismic input, consequences have to be expected. The seriousness can not be assessed with the existing information. It has to be recommended to perform a new re-evaluation based on the current practice.
5. Implementation of Seismic Upgrade Measures
No measures were considered to be necessary after the seismic re-evaluation of the plant. With respect to the civil structures the Austrian expert team can agree to this approach. With respect to interfaces and non structural components it has to be strongly recommended to perform analyses on this subject. A major role in the realistic assessment of the seismic hazard could play the monitoring system which has not been exploited so far. It is recommended to use the real data to determine realistic basic functions for the plant.

6. Evaluation of the Information provided
The information received from the Czech side is consistent. It also conforms to information received on PN6. The information appears to be complete. In relation to current practice it has to be expressed that the re-evaluation process followed the codes and standards at the time it has been performed. The Austrian expert team would have performed a similar seismic re-evaluation in 1996 for these structures. Nevertheless the approach has changed considerably since then and it is therefore recommended to re-open the re-evaluation chapter and take sorrow reconsideration. No attempt has been made to enter very deep into the subject. The methods applied represent a practice at the time of performance. As far as seismic modelling is concerned the selection of the finite element mesh has been very global in size which does not allow detecting any local problems. For re-evaluation it would be recommended to use a mesh with at least 10 times the number of elements.
7. Re-Evaluation Methodology
A consistent and complete information has been provided by the Czech side. The procedure has been oriented on existing legal requirements and not the evolving current practice. Considering the current practice in Western Europe the re-evaluation process should be re-opened and performed according to state of the art methodologies.

5 RECOMMENDATIONS

In order to improve the knowledge on the seismic performance and the eventual identification of necessary retrofit measures the Austrian expert team recommends to the Austrian government to propose to the Czech side the following:

- To perform a probabilistic safety analysis (PSA) on the level of the recommendation of IAEA and the current practice in Western Europe.
- To open the chapter of seismic qualification of civil structures interfaces and components again to be incorporated into the 10 year periodic safety review.
- To actively improve the monitoring system and enhance the use of actual data in the evaluation process including an improvement of the existing database.

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7 ABBREVATIONS

m_j^*	modal mass (a constant that depends on the mode shape and the mass distribution) as a consequence of decoupling the three degree of freedom system in three “quasi one degree of freedom”-systems
$\ddot{w}_g$	free-field input acceleration applied at the base of the structure
$\vec{e}$	influence coefficient vector, represents the displacements resulting from a unit support displacement
$\vec{F}_{S\,\mathrm{e},j}$	Vector of modal maximum forces (equivalent to the forces associated with the structure’s relative displacements)
q_k	generalized coordinates representing the amplitudes of the specified set of displacement patterns
$\underline{\mathbf{m}}$	mass matrix
$\vec{\Phi}_j$	Vector of modal shape j
L_j^*	earthquake excitation factor representing the extent to which the earthquake motion tends to excite a response in the assumed shape $\vec{\Phi}_j$
$S_{\mathrm{e},j}$	value of the ground acceleration corresponding with the vibration period of the modal shape j – taken from the response spectrum
$\boldsymbol{L_j^*/m_j^*}$	modal participation factor
$p(t)$	sequence of impulses of infinitesimal duration
τ	time instant, at which an impulse is starting
β and γ	define the variation of the acceleration over a time step
ξ_ι	modal damping ratio
50-SG-D1	Seismic Design and Qualification for Nuclear Power Plants
50-SG-S1	Earthquakes and Associated Topics in Relation to Nuclear Power Plant siting (IAEA Safety series)
A	acceleration
A, B, C, D, E	five main ground types
A_{ed}	is the most unfavourable combination of the components of the earthquake
A_{Ed}	design seismic action
a_g	design ground acceleration
a_{gR}	reference peak ground acceleration on type A ground
a_{vg}	vertical design ground acceleration
c	a spring constant
CQC	“Complete Quadratic Combination”
d	lateral elastic displacement of the top of the building
d	term of the column’s cross section in the global Y-direction
DCH	Design Concept (high ductility)
DCM	Design Concept (medium ductility)

d_e	displacement of the same point of the structural system, determined by a linear analysis based on the design response spectrum
DOF	Degree of freedom
d_s	displacement of a point of the structural system induced by the design seismic action
E	modulus of elasticity
EC 8	Eurocode 8
E_d	design seismic action
E_E	probable maximum value of a seismic action effect
E_{Edx}	action effects due to the application of the design seismic action along the chosen horizontal axis *x* of the structure
E_{Edy}	are the action effects due to the application of the same design seismic action along the orthogonal horizontal axis *y* of the structure
E_{Edz}	action effects due to the application of the vertical component of design seismic action
E_{Ei}	modal responses due to the vibration mode **i**
e_{li}	accidental eccentricity of storey mass **i** from its nominal location
ESM	Equivalent Static Method
E_x, E_y	maximum action effects
F	force (influence)
$\underline{f}$	Vector of natural frequency [Hz]
F(t)	a constant or a harmonic function of t corresponding to the disturbance
F_b	seismic base shear force
F_b	seismic base shear force
F_{bi}	seismic base shear forces
F_i	horizontal forces
G_k	represents the permanent loads with their characteristic values.
$G_{k,j}$	deadweight of the structure
GRS	Ground spectra
h	term of the column’s cross section in the direction of vibration
hor	horizontal
i	variable action
i	index of the actual calculated storey force
I	column’s cross-sectional moment of inertia
i and j	two vibration modes
IAEA	International Atomic Energy Agency
j	index over all existing storeys
k	modification factor to account for special regional situations
k	number of modes taken into account
k	frame’s stiffness per storey
k_{ij}	coefficients are defined as the force corresponding to coordinate **i** due to unit displacement of coordinate **j**

$\mathbf{k_w}$	factor reflecting the prevailing failure mode in structural systems with walls
l	height between the floors
L, M and H	ductility classes
LFM	Lateral Force Method
$\mathbf{L_i}$	floor-dimension perpendicular
LRFD	Limit state design method
M	wave magnitude
M	Center of mass
m	mass
$\mathbf{M_0}$	overstrength moment
MDOF	Multi degree of freedom
$\mathbf{m_i}$	every storey mass
$\mathbf{m_i}$, $\mathbf{m_j}$	masses computed
$\mathbf{m_{ij}}$	coefficients are defined as the force corresponding to coordinate **i** due to unit acceleration of coordinate **j**
$\mathbf{M_{Rb}}$	design values of the bending moments at the horizontal members
$\mathbf{M_{Rc}}$	bending moments at the observed joint occurring at the vertical members
$\mathbf{M}_{Rd}$	design flexural strength of the section
$\mathbf{M_s}$	surface-wave magnitude
n	number of storys above ground
$\mathbf{N}_{pl,Rd}$	represents the design compression resistance
NPP	Nuclear power plant
$\mathbf{N}_{Sd}$	represents the design axial force
NUREG	Report prepared for the U.S. Nuclear Regulatory Commission (NRC)
OBE	Operating Basis Earthquake
PGA	Peak ground acceleration
$\mathbf{P_k}$	represents the characteristic value of prestressing after all losses.
POSAR	Pre-operation safety analysis report
PSD	Power Spectral Density
q	Behavior factor
$\mathbf{q_0}$	basic value of the behavior factor (depends on the type of the structural system and on the regularity in elevation)
$\mathbf{Q_{1k}}$	represents the characteristic value of the traffic load.
$\mathbf{Q_2}$	quasi permanent value of actions of long duration (earth pressure, etc.)
$\mathbf{q_d}$	displacement behavior factor, assumed equal to q unless otherwise specified
$\mathbf{Q_{k,l}}$	structure’s live load
r	damping proportional to velocity
$\mathbf{R_d}$	corresponding design resistance
RSM	Response Spectra Method

RSMAM	Response Spectrum Modal Analysis Method
S	Damping correction factor
S	Center of lateral stiffness and mass
S(T)	Maximum absolute value of displacement
S_d **(T_1)**	ordinate of the design spectrum at period T_1
SDOF	Single degree of freedom
S_e**(T)**	Ground type-dependent elastic response spectra
s_i , s_j	displacements of masses m_i , m_j in the fundamental mode shape
SIR	Specific Information Request
SL1	Seismic Level 1 (according to IAEA)
SL1 (OBE)	Seismic Category 1 structures with the loads combinations as given in ACI 349 for OBE (SL-1)
SL2	Seismic Level 2 (according to IAEA)
SL2 (SSE)	Seismic Category 2 structures with the loads combinations as given in ACI 349 for SSE (SL-2)
SRSS-rule	Square root of sum of squares
SSE	Save shutdown earthquake
S_{Ve}**(T)**	elastic response spectrum (representing the vertical earthquake motion)
T	Period of vibration
t	time
T_1	fundamental period of vibration
T_1	fundamental period of vibration of the building for lateral motion
T_1	fundamental period of vibration
T_B, T_C, T_D	ground-type dependent special parameters
T_C	ground-type dependent (given in the National Annex)
T_{DLR}	Reference return period (Damage limitation requirement)
T_k	represents the period of vibration of mode *k*
T_{nrc}	Reference return period (No-collapse requirement)
U.S. NRC	US Nuclear Regulatory Commission
v	velocity
ver	vertical
VLI	Verifiable Line Items
x_h**(t)**	of Euler's homogeneous differential equation; is obtained by an exponential statement $e^{\lambda t}$
z_i, z_j	heights of the masses m_i ,m_j above the level of application of the seismic action
$\underline{\Phi}_1$	Vectors of modal shapes
T_{NXP}	Reference return period (No collapse requirement)
$[\chi]$	stiffness matrix
$[\mu]$	mass matrix

$[\rho]$	damping matrix
α_I	multiplier of the horizontal seismic design action at first attainment of member flexural resistance anywhere in the structure
α_u	multiplier of the horizontal seismic design action with all other design actions constant
γ_0	overstrength factor
γ_I	individual importance factors
γ_{ov}	material overstrength factor for steel
η	damping correction factor
λ	is a correction factor, the value is equal to: $\lambda = 0{,}85$ if $\mathbf{T_1} < 2\ \mathbf{T_C}$ and the building has more than two storys, otherwise $\lambda = 1{,}0$
ω	characteristical free-vibration frequencies
$\underline{\omega}$	Vector of circular frequency [rad / sec]
ω_D	natural circular frequency of the damped vibration
ψ_{21}	combination factor
$\psi_{E,i}\ (=\varphi \cdot \psi_{2,i})$	combination coefficients for variable action

ANNEX A

THE MONITORING SCOPE OF THE PROJECT PN8

Verifiable Line Items
Defined and accepted by the Specialist's Team
Revision 1, issued November 2003

VERIFIABLE LINE ITEMS (VLI)

VLIS – The Specialist' Team View
1. Legal Framework for Seismic Design and Evaluation
At the time of design For re-evaluation Application of EC8
2. Definition of the Seismic Input for Seismic Design
Ground Spectrum (Input from Site Seismicity) Floor Spectra Transfer Functions
3. Re-Evaluation Methodology
Selection of Evaluation Process Seismic Modelling Interpretation of Results
4. Identification of critical Structures, Interfaces and Components
Classification of Structures Critical Interfaces Non Structural Considerations
5. Results of the Re-Evaluation Process
Effect on the Containment Effect on other Structures Interfaces and Components
6. Implementation of Seismic Upgrade Measures
Containment and Primary Circuit Other Structures Other Measures taken Use of the Monitoring System Improvement of the Database
7. Evaluation of the Information provided
In terms of Consistency In terms of Completeness Relation to current Practice Relation to Codes and Standards What would we have done?

ANNEX B

SPECIFIC INFORMATION REQUEST

Special Information Request (SIR)

Introductory Remark

In order to complete the assessment and monitoring of the seismic design for the NPP Temelin it would be desirable to receive the following information:

1. Legal Framework for Seismic Design and Evaluation

Information whether an application of Eurocode 8 has been considered subsequently to the seismic re-evaluation are requested.

2. Re-Evaluation Methodology

The models shown in the presentation of Mr. Maly (FEM) are those where the seismic calculation has been performed on. Are there any other more detailed models for the structures?

3. Identification of critical Items

Has there been any re-evaluation of the consideration of interfaces and non-structural components in the re-evaluation process or after that?

4. Implementation of seismic upgrade Measures

Have there been any seismic upgrade measures implemented in the plants? If yes, which measures?

Is there any upgraded re-evaluation procedure intended under the new IAEA guidelines and the changed international practice?

ANNEX C

AUSTRIAN PROJECTS IDENTIFICATION

AUSTRIAN PROJECTS IDENTIFICATION

PN 1	Severe Accidents Related Issues – [Item No. 7a] *
PN 2	High Energy Pipe Lines at the 28.8 m Level (AQG/WPNS country specific recommendation) [Item No.1] *
PN 3	Qualification of Valves (AQG/WPNS country specific recommendation) [Item No.2] *
PN 4	Qualification of Safety Classified Components [Item No. 5] *
PN 5	Regular bilateral Meeting 2002
PN 6	Site Seismicity [Item No. 6] *
PN 7	Severe Accidents Related Issues – [Item No. 7b] *
PN 8	Seismic Design
PN 9	Reactor Pressure Vessel Integrity and Pressurised Thermal Shock [Item No. 3] *
PN 10	Integrity of Primary Loop Components – Non Destructive Testing (NDT) [Item No. 4] *
PN 11	Regular bilateral Meeting 2004

** The Items are related to Annex I of the “Conclusions of the Melk Process and Follow-up”*

ANNEX D

MONITORING MISSION STATEMENT

MONITORING MISSION STATEMENT

The Austrian expert team agreed on a “Mission Statement” to define the monitoring process coordinated by VCE.

“Monitoring” is a process performed in a predefined frame addressing selected issues defined in the “Conclusions of the Melk Process” as well as in the “Roadmap” and the solutions to these issues adopted by the Czech side.

Issues and their solutions are monitored on the basis of the reference safety criteria and requirements coherent with Safety Approaches accepted in Western Europe. The requirements are checked against the generally applied Defense in Depth Concept.

The monitoring has the objective to obtain evidence that adequate solutions have been submitted by the licensee to the licensing authority and that these solutions have appropriately evaluated and approved by the regulator. Monitoring aims at performing an evaluation of the quality and the adequacy of an overall process and the implementation results.

The Czech side has offered documentation and discussion opportunities.

The monitor, in order to form a consistent opinion should be provided with the opportunity to ask for additional information and evidence or request supporting assessments to understand the evidence presented.

Reports of the Austrian expert team therefore include monitoring results of

- What has been done
- How the applicable requirements have been addressed
- How the safety objectives and requirements compliance was analysed and justified for the proposed solutions
- How the solutions in the frame of the licensing process and considered in the related regulatory process were evaluated.

The monitors were not tasked with performing a licensing review of the Temelin NPP, and nothing in their reports may be construed to present any such review. The responsibility for the safety and licensing of Temelin remains with CEZ a.s. as the owner of the facility, and with the SÚJB, as the designated nuclear licensing and regulatory authority under Czech law.