IDM UID
VT29D6

VERSION CREATED ON / VERSION / STATUS
19 Dec 2025 / 2.1 / Approved

EXTERNAL REFERENCE / VERSION

MQP Level 3

Instructions for Seismic Analyses

This document provides instructions and guidelines for seismic analyses and seismic analysis reports. These instructions give:
- Requirements for performing seismic analyses.
- Requirements for reports that document seismic analyses.
- Guidance for how to comply with the requirements listed above.

Approval Process			
	Name	*Action*	*Job Title / Affiliation*
Author	**Zhang X.**	**19 Dec 2025:signed**	**Seismic Requirements & Analysis Coo...**
Co-Authors			
Reviewers	**Khomutnikov A.** **Schioler T.**	**19 Dec 2025:recommended** **19 Dec 2025:recommended**	**Quality Engineer** **Section Leader**
Approver	**Bartels H.- W.**	**30 Dec 2025:approved**	**Head of Division**
Information Protection Level: Non-Public - Unclassified *RO: Khomutnikov Aleksei*			
Read Access	**GG: MAC Members and Experts, AD: ITER, AD: External Collaborators, AD: External Management Advisory Board, AD: Nuclear Safety Inspectors, AD: OBS - Quality Management Division (QMD), AD: DA, AD: Auditors, AD: ITER Management Assessor, project administrator, RO, AD: IO_Director-General, AD: IDM_Contr...**		

#drn#

Change Log			
Instructions for Seismic Analyses (VT29D6)			
Version	***Latest Status***	***Issue Date***	***Description of Change***
v0.0	In Work	07 Dec 2017	
v1.0	In Work	19 Dec 2017	first version as per approved MQP doc request UXG7ZG
v1.1	Revision Required	21 Dec 2017	first version as per MQP doc Request - UXG7ZG using file Instructions for Seismic Analyses - draft08.docx.
v1.2	Approved	19 Jan 2018	Ref. [9] is added into the applicable documents; Minor modifications in Appendix A.1.2 and A.4.7.3 to address the comments from Eric; Other minor modifications to address the suggestions from Giulio.
v1.3	Approved	13 Apr 2018	The data in Table B.3-8 are corrected.
v1.4	Signed	25 Mar 2020	As per approved MQP doc request https://user.iter.org/?uid=YV3Z55 the changes are: The document was changed in the context of the IO Re-organization.
v1.5	Approved	08 Jun 2020	Under same MQP doc request minor change: Change of the versions for checklist references [6] and [7]
v2.0	Approved	16 Dec 2020	As per approved MQP doc request https://user.iter.org/?uid=4AGYV8 the document has to be reviewed by DAs in order to be included in MPA.
v2.1	Approved	19 Dec 2025	- Re-org changes (EPNS - SRO, AS - IEA, CIO removed) - Minor edits

Table of Contents

1 Purpose

This document provides instructions and guidelines for seismic analyses and seismic analysis reports. These instructions give:

- Requirements for performing seismic analyses.
- Requirements for reports that document seismic analyses.
- Guidance for how to comply with the requirements listed above.

Associated with this document are:

- A template for seismic analysis reports [8].
- Checklists for reviewers [5], independent peer reviewer [6] and technical checkers [7] of seismic analysis reports.

The QA requirements given in [1] are implemented in this document.

2 Scope

This document applies to the ITER Organization (IO) involved in the performance of seismic analyses and calculations. It also applies to Domestic Agencies (DA) or external contractors, who are asked to perform analysis or calculation tasks for the ITER project, see [1]. The rules governing the propagation of the requirements specified in these Instructions to external contractors or interveners are specified in [19], and shall be followed.

These instructions cover the activities associated with planning, preparing, technical checking and reviewing, issuing, and revising seismic analyses and calculations.

These instructions apply to the development of seismic analyses and calculations of ITER Systems, Structures and Components (SSCs) of any Quality Class (QC). The instructions are mandatory when any of the following apply to a seismic analysis or calculation:

- They are required or planned to be retained as a design verification and validation.
- They are required to document that an existing, modified, or proposed SSC will meet design or operational requirements.
- They constitute alternate calculations (see definition) for completing design verification of an SSC.
- They are required by other ITER procedures.

This process is not mandatory for preliminary or scoping calculations that are to be superseded by later analyses. For preliminary or scoping calculations the Quality Assurance (QA) requirements shall be defined on a case-by case by the Analysis Coordinator, see [1] for the definition of the role of the Analysis Coordinator.

These instructions do not cover spot-checking or surveillance activities by IO-CT on Protection Important Activities (PIAs). Requirements for these activities are specified in [1].

This document is generated from [1].

3 Definitions and acronyms

3.1 Definitions

Term	Definition
Computational model	The numerical implementation of the mathematical model, usually in the form of numerical discretization of the geometry, solution algorithm, and convergence criteria.
Conceptual model	The collection of assumptions and descriptions of physical processes representing the solid mechanics behaviour of the reality of interest from which the mathematical model and validation experiments can be constructed.
Discretization	The mapping of a continuous structure into discrete counterparts as it is done with a FE mesh.
Error	A recognisable deficiency in any phase or activity of modelling or experimentation that is not due to lack of knowledge, e.g. choosing an incorrect material property for use in the computational model, programming errors.
Margin	In these instructions, margin is defined as the difference in percent between the calculated result quantity $R_{calculated}$, e.g. membrane stress, and the allowable $R_{allowable}$: $\text{margin}\,[\%] = \frac{R_{allowable} - R_{calculated}}{R_{calculated}} \cdot 100\%$ In some calculations involving complex load combinations, different definitions of margin may be more appropriate.
Mathematical model	The mathematical equations, boundary values, initial conditions and modelling data needed to describe the conceptual model.
Seismic analysis	The computation of the responses due to earthquake for the seismic qualification of the SSC (structure, system and component), including deformations (strain, displacement, velocity and acceleration) and forces (stress, force, moment, etc.).
Uncertainty	A potential deficiency in any phase or activity of the modelling, computation or experimentation process that is due to inherent variability or lack of knowledge. Uncertainty can be as quantified as follows: $uncert.\,[\%] = \frac{R_{correct} - R_{calculation}}{R_{correct}} \cdot 100\%$ However, since a "correct" result in the ideal sense is not usually available, a result with an insignificant error can be considered correct.
Validation	The process of determining the degree to which a model is an accurate representation of the real world from the perspective of the intended uses of the mathematical model. Validation is based on comparisons between numerical simulations and relevant experimental data.
Verification	In these instructions, verification is the process of determining that computational model accurately represents the conceptual model. Note that this definition is slightly broader than the ASME V&V definition, where verification is the process of determining that a computational model accurately represents the underlying mathematical model and its solutions [12].

Table 3-1 - Definitions.

The list of definitions used in this document is given in Table 3-1. These definitions supplement those in [1] where definitions are given for alternate calculations, analyses, calculations, calculation software, deliverables, independent calculations, Independent Peer Reviewer, Reviewer, Technical Checker, and technical checking.

3.2 Abbreviations

The list of abbreviations used in this document is given in Table 3-2. For a complete list of ITER abbreviations see ITER_D_2MU6W5 - ITER Abbreviations.

BC	Boundary Condition
CAD	Computer Aided Design
DET	Data Exchange Task
DoF	Degree of Freedom
IEA	Integrated Engineering Analyses
FRS	Floor Response Spectrum (Spectra)
IDM	ITER Document Management system
INB	Installation Nucléaire de Base
IO	ITER Organization
FE	Finite Element
PD	Plant Description, ITER_D_2X6K67
PIA	Protection Important Activity
PR	Project Requirements, ITER_D_27ZRW8
QA	Quality Assurance
QC	Quality Class
RO	Responsible Officer of the system, section leader or division head
SC1	Seismic Class One
SC2	Seismic Class Two
SDC	Structural Design Criteria
SIC	Safety Importance Class
SL	Seismic Level
SMHV	Séismes Maximaux Historiquement Vraisemblables
SRD	System Requirement Document
SRO	Safety Responsible Officer
SSC	System, Structure and Component
ULS	Ultimate Limit State
V&V	Verification and Validation

Table 3-2 - Abbreviations.

4 References

[1] Procedure for Analysis and Calculations. ITER_D_22MAL7.
[2] Instructions for the Storage of Analysis Models. ITER_D_U34WF3.
[3] Software Qualification Policy. ITER_D_KTU8HH.
[4] Order dated 7 February 2012 relating to the general technical regulations applicable to INB – EN. ITER_D_7M2YKF.
[5] Reviewer Checklist for Seismic Analyses. ITER_D_Q6FH53_v2.1.
[6] Independent Peer Reviewer Checklist for Seismic Analyses. ITER_D_V5Z65L_v1.3.
[7] Technical Checker Checklist for Seismic Analyses. ITER_D_V5ZWSB_v1.2.
[8] Template for Seismic Analysis Reports. ITER_D_VAET99_v1.0.
[9] Procedure for ITER CAD Data Exchanges. ITER_D_2NCULZ.
[10] Procedure for the Preparation, Review and Approval of SRDs. ITER_D_25DSU2.
[11] IO Generic Template. ITER_D_34BAZX.
[12] ASME V&V 10-2006 – Guide for Verification and Validation in Computational Solid Mechanics.
[13] ASME V&V 10.1-2012 – An Illustration of the Concepts of Verification and Validation in Computational Solid Mechanics.
[14] ISO 16454-2007(E) Space systems - Structural design - Stress analysis requirements.
[15] Guideline for identification of the Protection important activities (PIA). ITER_D_SBYJXD.
[16] Quality Classification Determination. ITER_D_24VQES.
[17] Load Specifications, ITER_D_222QGL.
[18] Guideline-Framework Instruction for Safety Demonstration Art 3.8 INB Order. ITER_D_PQT8AC.
[19] Provisions for Implementation of the Generic Safety Requirements by the External Interveners. ITER_D_SBSTBM.
[20] Safety Important Functions and Components Classification Criteria and Methodology. ITER_D_347SF3.
[21] Methodology for the Hard Core Components. ITER_D_RMP3AC.
[22] ITER Seismic Nuclear Safety Approach. ITER_D_2DRVPE.
[23] Safety Requirements for ITER Facility Buildings. ITER_D_2E4KSJ.
[24] Load Specifications for Buildings with Safety Requirements. ITER_D_2ERTXQ.
[25] ITER Structural Design Code for Buildings (I-SDCB) - Part1: Design Criteria. ITER_D_283B24.
[26] Probabilistic seismic hazard assessment for the stress test studies for ITER and CEA Cadarache site – synthesis. ITER_D_R8CCFQ.
[27] Probabilistic seismic hazard assessment for the stress test studies for ITER and CEA Cadarache site – annexes. ITER_D_R97J7V.
[28] Report on Technical Specification for the Experimental Seismic Qualification of Active Electrical and Mechanical Components. F4E_D_24DN59.
[29] Design Seismic Floor Response Spectra in the Tokamak Complex. ITER_D_SVBRJZ.
[30] Civil Works – Floor Response Spectra. ITER_D_VCKHPA.
[31] Seismic Relative Displacements between the Building Floors. ITER_D_KWXL6B.
[32] Global Tokamak Seismic Analysis Report (relative displacement and FRSs). ITER_D_33W3P4.
[33] RN-52UL-007 Torus Displacements. ITER_D_JM4Y2A.
[34] Relative Displacement to be Considered for the Seismic Design of Multi-supported Equipment in NSC Buildings. ITER_D_SJRUAP.
[35] Methodology to be used to generate the seismic floor response spectra for ancillary buildings at ITER to enable the design of the equipment and equipment support. ITER_D_PN36V6.
[36] Excel sheets – Application of the FRS methodology, ITER_D_QN6LP3.
[37] Application of the FRS simplified methodology to Building 15. ITER_D_QPBST4.

[38] Construction Design - PBS 62.11, 62.14 and 62.74 - Tokamak complex - Design Catalogue of Cast-in-Place and Post-installed Fastening Systems - Structural Calculation Note - ENG_50_CR_110002_CW. ITER_D_AJSMNW.
[39] ITER Coordinate Systems. ITER_D_ 2A9PXZ.
[40] Template for Seismic Calculations for equipment and/or supports v2.0, Jul 2016, F4E_D_25J77B
[41] French Order of 7 February 2012 - République Française - Arrêté du 7 février 2012 fixant les règles générales relatives aux installations nucléaires de base.
[42] ASN/Guide/2/01, Guide de l'ASN Prise en compte du risque sismique a la conception des ouvrages de génie civil d'installations nucléaires de base a l' exception des stockages à long terme des déchets radioactifs, 2006.
[43] EN 1998: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings.
[44] EN 1998: Design of structures for earthquake resistance. Part 4: Silos, tanks and pipelines. January 1998.
[45] AFNOR NF EN 1998-1/NA Annexe nationale à la NF EN 1998-1:2005 Règles générales, actions sismiques et règles pour les bâtiments, Indice de classement : P 06-030-1/NA.
[46] EN 1992: Design of concrete structures, 2004
[47] DRAFT prEN 1992-4: Design of concrete structures, Part 4: Design of Fastenings for Use in Concrete, European Standard, September, 2013
[48] EN 13480 Metal Industrial Piping. Part 3: Design and Calculation, 2012
[49] Design and Construction Rules for mechanical components of nuclear installations: high-temperature, research and fusion reactors, RCC-MRx 2015, Section III – Tome 1 – Subsection Z – Appendix A1: Guide for Seismic Design of Components.
[50] RCC-CW, Rules for Design and Construction of PWR Nuclear Civil Works, AFCEN, 2015.
[51] AFPS Guides Techniques pour la prévention parasismique des installations industrielles soumises à l'arrête du 24 janvier 2011. Guide Structures Support. May 2014.
[52] AFPS Guide Séisme – Tuyauterie & Robinetterie, Version 1, 2014
[53] ASCE 4-16 Seismic Analysis of Safety - Related Nuclear Structures and Commentary, Working Group on Revision of ASCE Standard 4., ASCE Standard 4. April 2017.
[54] ASCE – American Society of Civil Engineers "Guidelines for the Seismic Design of Oil and Gas Pipeline Systems", Committee on Gas and Liquid Fuel Lifelines, 1984.
[55] ASCE – American Society of Civil Engineers "Standard 4-98. Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures", 2000.
[56] US NRC Regulatory Guide 1.92 Revision 2 Combining Modal Responses and Spatial Components in Seismic Response Analysis, July 2006.
[57] US NRC Regulatory Guide 1.61 Damping Values for Seismic Design of Nuclear Power Plants, March 2007.
[58] ACI 318-08 "Building Code Requirements for Structural Concrete and Commentary", Appendix D Anchoring to Concrete; January, 2008.
[59] IEC 60980. Recommended Practices for Seismic Qualification of Electrical Equipment of the Safety System for Nuclear Generating Stations.
[60] ISO 898-1 "Mechanical properties of fasteners made of carbon steel and alloy steel - Part 1: Bolts, screws and studs with specified property classes — Coarse thread and fine pitch thread", 2013
[61] API 620 Design and Construction of Large Welded, Low-Pressure Storage Tanks. 11th Edition Addendum 3, March 2012.
[62] API 650 Welded Steel Tanks for Oil Storage. 12th Edition, March 2013
[63] VCI Leitfaden - Der Lastfall Erdbeben im Anlagenbau - Lehrstuhl für Baustatik und Baudynamik RWTH AACHEN -, C. Butenweg, H.- J. Dargel, T.Höchst, B. Holtschoppen, R. Schwarz, M. Sippel., September 2012

[64] ATC 40 Seismic Evaluation and Retrofit of Concrete Buildings, Report SSC 96-01, 1996
[65] EPRI NP-6809 A simplified Inelastic Seismic Analysis Method for Piping Systems, 1990
[66] FEMA Seismic Design and Retrofit of Piping Systems, 2002
[67] FEMA 273 NEHRP Guidelines for the Seismic Rehabilitation of Buildings, 1997
[68] FEMA 274 NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings, 1997
[69] FEMA 440 Improvement of Nonlinear Static Seismic Analysis Procedures, 2005
[70] FEMA Guidelines for the Design of Buried Steel Pipe, 2001
[71] FEMA Guide for Seismic Evaluation of Active Mechanical Equipment, October 2004
[72] FEMA 414 Installing Seismic Restraints for Duct and Pipe, January 2004.
[73] FEMA 413 Installing Seismic Restraints for Electrical Equipment, January 2004.
[74] FIB Bulletin 58 "Design of Anchorages in Concrete", Guide to good practice, 2010
[75] Seismic Design of Storage Tanks. Recommendations of a NZSEE Study Group on Seismic Design of Storage Tanks. November 2009.
[76] TID-7024 Chapter 6 Dynamic pressure on fluid containers. US Atomic Energy Commission, Nuclear Reactors and Earthquakes. Office of Technical Service, Washington DC, 1963.
[77] R.S. Wozniak, W.W.Mitchell. "Basis of seismic design provisions for welded steel oil storage tank", API, Refining 43rd Midyear Meeting, Toronto Canada, May 1978.
[78] Young, W. C., & Budynas, R. G. "Roark's formulas for stress and strain" (Vol. 7). New York: McGraw-Hill. 2002
[79] Eligehausen, R., Mallée, R. and Silva, J.F. Anchorage in Concrete Construction. Ernst and Sohn. 2006.
[80] CSA Today, Design of Anchor Reinforcement in Concrete Pedestals, Vol. III, No. 12, Widianto, C. Patel, and J. Owen
[81] Benchmark of Different ANSYS Shell Elements for Bending Stress Results. 2N5PU7.

5 General Principles

The management requirements for seismic analyses are those defined in [1]. Technical requirements for seismic analyses are defined in 9APPENDIX A, and shall be followed.

6 Workflow

The workflow for seismic analyses is defined in [1].

7 Responsibilities

General roles and responsibilities for analysis and calculations are defined in [1]. In this document, the roles and responsibilities are defined for analyses conducted under different arrangements, for example:

- ITER Task Agreements (TAs) or Procurement Arrangements (PAs).
- Direct IO or DA contracts.
- Performed by IO or DA staff.

The requirements and responsibilities listed in [1] shall be applied to seismic analyses and the associated reports. Additional clarifications on the roles and responsibilities for seismic analyses are given in this chapter.

Document [1] defines the roles and responsibilities of the IO Responsible Officer, IO Analysis Coordinator or Requester, Contract Manager, Performer's Manager or Supervisor, Performer, Reviewer or Technical Checker, and of the Independent (Peer) Reviewer for all different types of Analyses and Calculations in general. The following chapter re-call some of them or further clarifies the specific roles within the context of the Seismic analysis process.

7.1 Performer

The Performer executes and documents the analysis or calculation in accordance with these Instructions.

7.2 Approver of Analysis Report

The approver of the seismic analysis report in IO IDM shall:

- Ensure that the Performer, reviewers and Technical Checker are Suitably Qualified and Experienced Persons (SQEP).
- Ensure that any software tools are properly qualified.
- Ensure that the purpose and scope of the document are fully met.
- Ensure that the reviewers and Technical Checker fulfil the scope of their review.
- Ensure that the review has been exhaustive and certified. The review has included all necessary actors and interfaces, and covered all relevant aspects in relation to the purpose and scope of the analysis task. As a minimum, this includes the reviews specified in Section 7.3.

7.3 Reviewers of Analysis Report

This section lists the minimum points that must be reviewed for seismic analysis reports. The points are assigned to Reviewers, an Independent Peer Reviewer and a Technical Checker, see [1] for definitions. The outcomes of the reviews shall be stored in IO IDM as specified in Chapter 8 [5] [6] [7].

To reduce the workload of the reviewers of the seismic analysis report it is required to specify the scope of review of each reviewer. This scope can be defined either in the report itself, or else be done directly in IO IDM.

For the reviews defined in Subsections 7.3.2, 7.3.3 and 7.3.4, the use of different checklists may be allowed if these checklists are agreed in writing with the entity in the IO responsible for these Instructions. Based on the current IO organisation, the responsible entity is the Integrated Engineering Analysis Section of the Central Integration Division. A prerequisite for the use of alternative checklists is a demonstration that the alternate checklists are at least as comprehensive as the ones given in these Instructions.

7.3.1 Safety Responsible Officer (SRO) Review

The SRO for the SSC shall be assigned as an Observer in IDM. If the SRO requests to be a Reviewer in IDM, the SRO shall be assigned as a Reviewer.

7.3.2 Reviewers

The checks listed in Table 7-1 shall be performed. The review may be performed by a single Reviewer or be split between two or more Reviewers.

The requirements listed Table C-1 shall be followed when performing a review. The outcome of the review shall be recorded using the template in [5].

Check ID	Check
R1	Report title, format and metadata
R2	Abstract, purpose and scope
R3	Scope of reviewers
R4	Definitions and abbreviations
R5	Units
R6	Geometry (excluding applicability)
R7	Applicability of geometry [1]
R8	Material properties (excluding applicability)
R9	Applicability of material properties [1]
R10	SDCs (excluding applicability)
R11	Applicability of SDCs [1]
R12	Loads (excluding applicability) [2]
R13	Applicability of loads [1]
R14	Conceptual model and analysis methodology
R15	Description of FE analyses (only applicable for FE analyses)
R16	Hand calculations (only applicable for hand calculations)
R17	Results
R18	Verification of FE analysis (only applicable for FE analyses)
R19	Conclusions
R20	References

Table 7-1 - Minimum checks to be performed by Reviewers.

7.3.3 Independent Peer Reviewers

The checks listed in Table 7-2 shall be performed by the Independent Peer Reviewer. Note that the list of checks is identical to that for Reviewers, except that there are no checks on the applicability of the geometry, material properties, SDCs or loads.

Also note that the requirement that check R12 be performed by the RO of the SLS of the SSC is only applicable to Reviewers, not to Independent Peer Reviewers.

The requirements listed Table C-1 shall be followed when performing an independent peer review. The outcome of the review shall be recorded using the template in [6].

[1] Checks concerning applicability shall be performed by the RO of the SSC.

[2] This check shall be performed by the RO of the SLS of the SSC. It is also recommended that the RO of the SLS checks how the loads are applied to the FE model (part of R15).

Check ID	Check
R1	Report title, format and metadata
R2	Abstract, purpose and scope
R3	Scope of reviewers
R4	Definitions and abbreviations
R5	Units
R6	Geometry (excluding applicability)
R8	Material properties (excluding applicability)
R10	SDCs (excluding applicability)
R12	Loads (excluding applicability)
R14	Conceptual model and analysis methodology
R15	Description of FE analysis (only applicable for FE analyses)
R16	Hand calculations (only applicable for hand calculations)
R17	Results
R18	Verification of FE analysis (only applicable for FE analyses)
R19	Conclusions
R20	References

Table 7-2 - Minimum checks to be performed by Independent Peer Reviewers.

7.3.4 Technical Checkers

The checks listed in Table 7-3 shall be performed by the Technical Checker:

Check ID	Check
TC1	Conceptual model and analysis methodology.
TC2	Mathematical model.
TC3	The analysis model is properly stored in the analysis database.
TC4	The model in the database matches the report.
TC5	The results of the model in the database match the description in the report.
TC6	Analysis results are reasonable, and hand calculations are correct.

Table 7-3 - Minimum checks to be performed by Technical Checkers.

The requirements listed Table C-2 shall be followed when performing a technical check. The outcome of the review shall be recorded using the template in [7].

8 Interactions with Other Processes

The interactions with other processes are defined in [1].

9 Records

Seismic analysis reports shall be titled such that the scope of the analysis (Component, PBS, loads and failure modes) is described as well as possible within the confines of a reasonable number of characters.

Seismic analysis reports that follow these instructions shall be uploaded in IO IDM as document type "Calculations" (Analysis and Calculation report following MQP procedure 22MAL7). Records of seismic analyses that are outside the scope of these instructions shall not be uploaded as document type "Calculations". This is important, as it allows the project to determine which analysis reports can be used for design verification and validation purposes, see Chapter 2.

General requirements for the storage of analysis reports are given in [1].

All analysis models that support the analysis report shall be uploaded to the IO Analysis Model Database in accordance with [2]. If calculations are performed using software such as Excel or Mathcad, the relevant spreadsheets or worksheets shall also be uploaded to the analysis model database.

The following templates shall be used for documenting reviews and technical checks, unless otherwise agreed, see Section 7.3:

- Checklist for Reviewers of seismic analysis reports [5].
- Checklist for Independent Peer Reviewers of seismic analysis reports [6].
- Checklist for Technical Checkers of seismic analysis reports [7].

Completed checklists shall be uploaded to IO IDM in a manner that makes it impossible to modify them at a later stage in an untraceable manner. Possibilities include:

- Uploading the checklists as attachments to 'comments'. This is the recommended approach.
- Attaching the checklists to the reports.
- Uploading the checklists as stand-alone documents. In this case they shall be uploaded as document type "Engineering Analysis". In addition, links shall be made from the analysis report to these checklists, thereby making it possible to identify that the review has been performed.

APPENDIX A Technical Requirements

The general requirements for analyses are given in [1]. The following chapter provides more specific requirements for the seismic analysis domain.

Appendix A.1 Requirements for all Types of Seismic Analyses

Appendix A.1.1 Conceptual Model and Analysis Method

The chosen conceptual model shall represent the physical reality sufficiently accurately to cover the intended purpose of the analysis. In order to do so it is necessary to have a clear definition of the intended use of the model.

Appropriate analysis method(s) shall be used. Note that hand calculations and finite element analyses are valid analysis methods, so long as they are used in an appropriate manner and domain. Appendix B.2 gives some guidance for the applicable methodologies. For any specific methodology chosen from those introduced in Appendix B.2, the associated requirements in Appendix B.2 shall be respected.

Appendix A.1.2 Geometry

Analyses shall be based geometry that is referenced and consistent with the current approved design of the SSC. See Appendix A.4.7.3 for how to achieve this. The uncertainty in the geometry, e.g. due to tolerances, shall be considered.

Analyses performed during or after the construction phase shall consider any relevant non-conformances.

Deviations from the referenced design geometry shall be justified. The quantitative effect of the deviations on the results shall be estimated, and considered in the conclusions of the analysis.

Appendix A.1.3 Material properties

The analysis shall be based on referenced material properties that are consistent with the procured materials. The analysis shall also consider the uncertainties in material properties. A practical way of ensuring this during the design phase is to reference and use the minimum and/or maximum values specified in the applicable Structural Design Criteria.

Appendix A.1.4 Seismic Loads

Safety Importance Class (SIC) and Seismic Class (SC) of the SSCs shall be clearly presented, as specified in Appendix B.1.1 and Appendix B.1.2. The classifications can be specified in the System Load Specification (SLS) document and explicitly referred by the analysis report.

The input seismic loads shall be clearly defined. Reference documents for the definition of the seismic loads, if any, shall be approved. Appendix B.3 gives the guidance to define the seismic loads.

Where there are uncertainties in the input data, the input data shall be considered in a conservative manner. This may require that more than one calculation is performed.

Appendix A.1.5 Units

All analyses shall be performed using S.I. base and derived units. The only exception to this rule is that degrees Celsius may be used instead of Kelvin. The table below lists the most common units used for seismic analyses.

Quantity	Unit name	Unit symbol	In SI base units
Length	Meter	m[3]	
Mass	Kilogram	kg	
Time	Second	s	
Temperature	Kelvin	K	
	Celsius	°C	
Acceleration			m/s^2
Angular acceleration			rad/s^2
Angular velocity			rad/s
Density			kg/m^3
Energy, Work	Joule	J	N·m
Force	Newton	N	$kg{\cdot}m/s^2$
Frequency	Hertz	Hz	1/s
Moment			N·m
Second moment of area			m^4
Power	Watt	W	N·m/s
Pressure	Pascal	Pa	N/m^2
Stress	Pascal	Pa	N/m^2
Young´s Modulus	Pascal	Pa	N/m^2
Velocity			m/s

Table A.1-1 – Definitions.

Appendix A.2 Additional Requirements for Finite Element Analyses

Appendix A.2.1 Software Package

Any software package used to perform FE analyses shall be validated. The software package shall be used in its validated domain.

If a validated Finite Element software package has non-negligible uncertainties when used properly, the uncertainties shall be covered either by performing sensitivity studies or by applying a suitable safety factor to the results.

Appendix A.2.2 Coordinate systems

The global coordinate system for FE models shall have its positive z-axis pointing vertically upward. For equipment located in the Tokamak Complex it is recommended to use the ITER Tokamak Global Coordinate System (TGCS) [39], since it simplifies the possible integration of the FE model in a higher level model. In case this requirement causes particular disadvantages in the performance of the analysis a different origin may be chosen. One or more local coordinate systems may also be used.

Appendix A.2.3 Element Types and Shapes

The choice of element types and shapes shall be justified.

The choice of element type is partly a question of the geometric simplification that is introduced, e.g. beam, shell or solid elements. Such choices are justified as part of the justification of the conceptual model. However, the choice of element types is also a question of the assumptions that underpin each element type and its options.

[3] In ITER the reference unit of length is [m]. All FE models shall be made with lengths being defined in meters rather than for example mm. This simplifies the integration of different models, as well as the exchange of models between different IO divisions and IODAs. A standard unit of length also reduces the risk of mistakes which may occur when material properties are transferred between FE models with different units.

Many FE elements have limits to the shape they can have whilst giving reliable results. The documentation for FE codes usually comes with definitions of acceptable shape criteria, such as Jacobian ratios. The software itself also usually comes with shape checking tools that check if the FE mesh respects these criteria. Shape checking shall be performed. If poor quality elements have been identified by the check, their use shall be justified.

Appendix B.10 gives some guidance on the choice of element types and shapes.

Appendix A.2.4 Solution Settings

The solution settings chosen for the analysis shall be documented and justified. Appendix B.12 gives some advice on this topic.

Appendix A.2.5 Verification

The accuracy of the results obtained with the FE model shall be verified by the performer of the analysis. The mandatory verifications listed here are partly based on [14].

The results of these verifications of the FE model shall be reported in the analysis report. Appendix B.9 gives some background information on the necessity of the requirements.

Appendix A.2.5.1 Numerical Code Verification

The purpose of code verification is to check whether in the FE software the solution algorithms are correctly implemented. This is typically done by the software vendor for commercial software. Analysis software and its installation of the computer used for the analysis shall be qualified according to [3].

Appendix A.2.5.2 Mass Check

A mass check shall be performed if inertial effects are relevant to the analysis. This is almost always the case for seismic analyses.

The total mass, centre of gravity, and second moments of inertia are usually calculated by default by FE codes. They are usually given in the general analysis output associated with the solution phase. The total mass, centre of gravity and if possible the inertia of the FE model shall be documented and compared to the values listed in the system load specification. Significant differences have to be explained; as such differences usually indicate modelling errors.

For complex structures it is often necessary to perform this check on a component level.

Appendix A.2.5.3 Unit Acceleration Load Check

Unit acceleration load checks shall be performed if inertial effects are relevant to the analysis. This is almost always the case for seismic analyses.

In a verifying analysis only the unit acceleration shall be applied to the final FE model. It shall be checked whether the reaction forces on the constraints of the FE model correspond to the mass of the FE model.

Appendix A.2.5.4 Seismic Load Check

The purpose of this check is to verify that the seismic loads defined in Appendix A.1.4 are applied correctly (magnitude, direction, etc.).

For equivalent static analysis, it shall be checked whether the total reaction forces and moments on the supporting constraints correspond to the total applied loads, i.e. the product of acceleration and masses. This verifying analysis shall be static to prevent that dynamic effects influence the results. In case the FE model has more than one constraint it shall be checked whether the distribution of the reaction forces is reasonable.

For response spectrum analysis, under each direction of excitation, it shall be checked that the reaction forces on the constraints have the most dominant component in the same direction.

Appendix A.2.5.5 Contact Check

This check is only required if contact elements are used.

It shall be demonstrated that the behaviour of the contact elements is as intended. In order to reduce the probability of such errors, it is recommended that the guidance in Appendix B.13 is followed.

Appendix A.2.5.6 Damping Check

This check is only required if structural damping is used, typically for dynamic analysts.

It shall be demonstrated that damping has been applied as intended. Appendix B.14 gives some guidance for the use of damping, and how its application can be verified.

Appendix A.2.5.7 FE Mesh Discretization

A common cause of poor accuracy in FE models is insufficient mesh density. The FE code does not warn the user of inaccuracies due to too coarse a mesh. The mesh density shall therefore be justified, and the sensitivity of the results to the mesh density shall be considered in the interpretation of the results. Several methods exist for this, but Appendix B.8 gives some guidance in this regard.

Appendix A.2.5.8 Comparison with Alternative Results

A verification of calculated results with alternative calculations shall be performed for every FE analysis. Alternative results can be obtained with any of the following methods:

- A hand calculation (usually the analytical result of a simplified conceptual model).
- The same analysis performed with a different, already verified FE model that is validated to be suitable for the analysis task.
- By performing a comparable FE analysis independently (or referring to a previous independently performed analysis). Being performed independently includes that it is performed by a different analyst based on comparable input data such as geometry, material properties, and loads. The FE mesh, the boundary conditions, and load application must be created anew, and the element types, real constants, and solution settings must be chosen anew, i.e. not copied from the original analysis. Also the conclusions must be drawn independently from the independent analysis results.

All methods have in common that the results are obtained in an alternative way that bypasses possible sources of error and uncertainty in the FE analysis.

In case more than one type of result is intended to be obtained by the seismic analysis, alternative results may be required for each type of result.

Appendix A.3 Additional Requirements for Hand Calculations

An essential part of a hand calculation is the justification of the conceptual model that the hand calculation is based on, see subsection Appendix A.1.1.

The estimated accuracy of the hand calculation must be stated, along with the consequently chosen uncertainty factor.

All equations used in the calculation shall be shown. A reference shall be given for any non-trivial analytical formulas used. To improve the clarity it is recommended to use an equation editor. Equations shall be referenced, for example using the format below:

Second moment of area *I* can be calculated as in (Eq. 1) and (Eq. 2):

$$I = \frac{bh^3}{12} \qquad \textbf{(Eq. 1)}$$

$$= 2.554 \times 10^{-6}\ \text{m}^4 \qquad \textbf{(Eq. 2)}$$

where b = 1.135 m, the width of the cross-section
h = 0.03 m, the height of the cross-section

- In the first part the equation is given using symbols.
- The result shall be given including the unit.
- All symbols used in the equation shall be defined somewhere in the document.

Appendix A.4 Requirements for Seismic analysis Reports

This section covers the requirements for seismic analysis reports. Analyses performed inside IO shall follow the template for seismic analysis reports, [8]. This is based on the IO Generic Template for documents [11]. DAs and subcontractors may use their own templates, e.g. [40], but the reports shall contain all of the contents described in this section.

Seismic analysis reports should be provided in Microsoft Word format (.doc or .docx) in order to facilitate revisions and updates. If final reports are provided in "pdf" format, the Word version shall be stored in IDM as an attached file.

It is recognised that seismic analysis reports can serve a wide range of purposes, and that a fixed list of section headings is not always appropriate. The list of headings covered here is appropriate in most cases. If any of the sections listed here are not applicable for a particular report, the sections shall be included, along with the text "Not applicable" underneath it.

The author should add additional sections if these are required. This is especially the case when a single document reports about both seismic and structural analyses. In this case, the report template prescribed by the seismic MQP instructions can be merged to the one for structural analyses.

The headings in this section are capitalised to indicate that they correspond directly to headings in the template for seismic analysis reports.

Appendix A.4.1 ABSTRACT

The abstract of the seismic analysis report shall contain the following information:

- The ITER SSC to which the seismic analysis is related.
- The assessed components or parts of the system. PBS codes should be used where practical.
- The seismic classification of the SSC and the input seismic load of the seismic analysis.
- Any recent significant changes of design, structural design criteria or load specification.
- A statement that the report was written following these instructions, and that the loads applied in the assessments are consistent with the system load specification.

Appendix A.4.2 CHANGE LOG

This section is mandatory, unless changes are logged using the in-built feature in IDM.

This section contains a log of the changes made to the document between different versions. If changes are not logged directly in IDM, they can be logged using the format below.

Version	Location	Change
1.1	2.2	The following sentence was inserted: Masses are occasionally given in metric tonnes instead of kilos.
	Appendix A	The presentation of the stiffness and flexibility matrices was improved.
1.0		First version.

Table A.4-1 – Definitions.

Appendix A.4.3 SCOPE OF REVIEWERS

The scope of the review shall be specified for each reviewer, allowing each reviewer to focus on his/her part. Chapter 7 defines the required reviewers and their scope. Additional reviewers can be added.

The scope of reviewers can be specified either in the analysis report itself or directly in IDM. The former is often necessary due to the character limits of the relevant fields in IO IDM. This section is mandatory, unless the scope of reviewers is specified directly in IDM. If the scope of reviewers is not specified directly in IO IDM, it can be specified using the format below.

Reviewer	Scope of review
J. Bloggs	Reviewer o Design philosophy and analysis methodology are well documented and sound. o ….
J. Smith	Reviewer (Load Specification) o All loads come from the relevant reviewed and approved System Load Specification. o The relevant System Load Specification has been interpreted correctly.

Table A.4-2 – Definitions.

Appendix A.4.4 PURPOSE

This section of the report outlines the aim of the report.

Appendix A.4.5 SCOPE

This section should contain a description of the parts of the ITER Project to which the document applies. It may also be necessary to define areas where this document is NOT applicable. Applicability can typically be defined in terms of:

- Geometry – e.g. pressure vessel with its nozzle but not the connected pipe and its weld. PBS codes should be used where practical.
- Loads – e.g. only the category III SMHV loads specified in the SLS [x].
- Results – e.g. seismic interface loads, maximum acceleration, etc.
- Context – e.g. Preliminary Design Review.

Appendix A.4.6 DEFINITIONS AND ABBREVIATIONS

This section could contain lists of all definitions and abbreviations used in the document.

Appendix A.4.6.1 DEFINITIONS

This section is only required if certain terms have specific meaning in the context of this document, that is outside the normally accepted dictionary definitions. As an example, see Table A.4-3.

Term	Definition
Computational model	The numerical implementation of the mathematical model, usually in the form of numerical discretization of the geometry, solution algorithm, and convergence criteria.
Validation	The process of determining the degree to which a model is an accurate representation of the real world from the perspective of the intended uses of the mathematical model. Validation is based on comparisons between numerical simulations and relevant experimental data.

Table A.4-3 – Definitions.

Appendix A.4.6.2 ABBREVIATIONS

All abbreviations and acronyms used in the report shall be listed in alphabetical order, e.g. as below:

BC	Boundary Condition
EM	ElectroMagnetic
FE	Finite Element

Table A.4-4 – Abbreviations.

For a complete list of standard ITER abbreviations see: ITER_D_2MU6W5 - ITER Abbreviations.

Appendix A.4.7 UNITS AND INPUT DATA

Appendix A.4.7.1 UNITS

All units used in the analyses and the analysis report shall be listed. These units shall be S.I. base and derived units. Whilst FE models shall be always created using S.I. units, analysis reports may use standard S.I. prefixes to aid presentation. For example, whilst Young's Modulus in an FE model shall always be defined in Pa, the analysis report could list this parameter in GPa.

Appendix A.4.7.2 SAFETY IMPORTANCE CLASS AND SESIMIC CLASS

Safety importance class (SIC) and Seismic Class (SC) of the SSCs, as described in Appendix B.1.1 and Appendix B.1.2, shall be clearly stated in the analysis report, referring to the SLS.

Appendix A.4.7.3 GEOMETRY

The geometry on which the analysis is based shall be unambiguously traceable. Ways of achieving this include:

- Providing references to Data Exchange Tasks (DET) [9]. This is currently the recommended approach in ITER.
- Providing ENOVIA references to frozen CAD files or drawings.
- Including a full set of dimensions and/or drawings in the analysis report.
- Attaching CAD files or drawings to the FE model in the ITER Analysis Model Database.

A figure of the geometry used in the seismic analysis shall be shown, and the main dimensions relevant to the analysis indicated. Special attributes of the geometry that cannot be easily recognized in a figure and that are relevant to the analysis shall be described, e.g. clearance of pins or geometrical imperfections.

In case geometrical imperfections are considered in the analysis, which are not contained in the initial geometry but are imposed through a modification of the initial FE mesh (e.g. after a linear buckling analysis), these imperfections shall be described in the results section. The maximum magnitude(s) of the imperfections shall however be stated in this section.

Appendix A.4.7.4 MATERIAL PROPERTIES

The physical material properties of the analysed SSC shall be listed here. These material properties shall be traceable, and consistent with the procured materials.

Material properties can be reported in tables like the ones below. It shall be clearly documented which parts are made from which materials. This can for example be done by adding a column to the table below. The link between materials and geometry can also be made in the preceding "Geometry" section.

Material designation	Property	Notation	Value	Unit	Source
X2CrNiMo 17-12-2 austenitic stainless steel plate (Grade 1.4404)	Young's modulus	E	200	GPa	[X] §A3.3S.22
	Poisson's ratio	ν	0.3	-	[X] §A3.3S.23
	Density	ρ	7930	kg/m^3	[X] §A3.3S.24
	Allowable stress for Class-1 components	S_m	127	MPa	[X] §A3.3S.43
16Cr–12Ni–2Mo Stainless steel type 316L - plate (UNS designation S31603)	Modulus of Elasticity	E	195	GPa	[Y] §II.D Table TM-1 Group G
	Poisson's ratio	ν	0.31	-	[Y] §II.D Table PRD High alloy steels (300 series)
	Density	ρ	8030	kg/m^3	
	Allowable stress for Class-1 components	S_m	115	MPa	[Y] §II.D Table 2A Line 36

Table A.4-5 – Example table of material properties at 20°C. In this example, [X] is RCC-MRX 2015, [Y] is ASME BPVC 2015.

Appendix A.4.7.5 DAMPING

If damping is considered in a model, the level of damping shall be listed here. The level of damping shall be justified, for example by referring to the applicable SLS.

Appendix A.4.7.6 STRUCTURAL DESIGN CRITERIA

If the design is verified in the analysis report, the design code applicable to the analysis of the system shall be stated here. This shall be consistent with the definition in the relevant SRD, in the section "Applicable Codes and Standards", see [10].

The rules and limits from the design code applicable to this analysis shall be extracted from the design code and summarized here. Since design codes are written in a very general manner, it is often difficult to comprehend how they apply to specific cases. It is therefore recommended to explain how the design code applies to the analysis.

The service limits of the system are usually not given in the design code but are listed in the SRD or another related document. The corresponding reference shall be given and the service limits applicable to this analysis shall be listed, e.g. maximum service temperature, allowable displacements etc.

Appendix A.4.7.7 SEISMIC LOADS

The input seismic loads used for analyses shall be listed here, with reference to the applicable System Load Specification. All loads shall be described clearly and unambiguously. Where there is uncertainty in the loads, these uncertainties shall be reported.

The transient or spectral loads shall be given either in form of a table or on a diagram that allows the identification of characteristic magnitudes of the load time or frequency functions. Loads that are shown graphically should also be defined numerically. Lengthy input may be put in appendices or in attached text files or spreadsheets.

Appendix A.4.8 METHODOLOGY

The principle of the analysis approach shall be described.

The conceptual model shall be justified, in particular the inherent simplifications compared to the physical reality. Appendix B.7 gives some advice on this topic. Where simplifications, e.g. linear material properties, are allowed/required by the structural design criteria, it can simply be stated that the simplification is in accordance with the chosen design criteria.

It shall be justified that the analysis methods are used in their validated domains. If the analysis method is allowed/required by the structural design criteria, it can simply be stated that the method is in accordance with the chosen design criteria.
Sensitivity studies may be required if the conceptual model or analysis method cannot be justified by referring to the structural design criteria.

Appendix A.4.9 DESCRIPTION OF FE ANALYSIS

This section is mandatory if the report concerns an FE analysis. All subsections are mandatory, unless stated otherwise.

Appendix A.4.9.1 TYPE OF ANALYSIS

The type of analysis shall be stated, e.g. single point response spectrum analysis.

Appendix A.4.9.2 SOFTWARE PACKAGE

The name and version number of the software package used for the analysis shall be stated, e.g. "The FE analysis was performed using ANSYS 17.1 Mechanical APDL".

In case analyses are classified as PIA, the performer has to check that the software package is accepted as validated by the responsible entity in the IO safety department. It shall be justified that the software package is used in its validated domain.

It shall be stated what uncertainties, if any, are associated with the use of the validated Finite Element software package for the reported analysis.

Appendix A.4.9.3 COORDINATE SYSTEM(S)

All coordinate systems used in the FE analysis shall be defined. This includes but is not limited to the following:

- Local coordinate systems that are used for boundary conditions and load application.
- Coordinate systems used for results.
- The global coordinate system in which the FE model geometry is created.

Appendix A.4.9.4 FE MATERIAL PROPERTIES

All material properties used in the FE model shall be listed. These may be different from the physical material properties, for example, when ‘smeared’ material properties are used, or when material densities are modified to match masses listed in the System Load Specification. In case the physical material properties are used in the FE model, it is acceptable to combine the requirements of this section with those of Appendix A.4.7.4 and report all of the required information in a single paragraph.

In case tuned densities are used, they should be compared to the theoretical values. Significant discrepancies should be explained, as they often indicate problems with the FE model.

Since material properties are usually temperature dependent, the assumed temperature(s) of the structure could be stated in this section.

Material properties can be reported in tables like the ones below.

Part	**Material number**	**Young's modulus (GPa)**	**Poisson's ratio**	**Density (kg/m^3)**
Top lid	1	195	0.31	8600
Upper cylinder	2	195	0.31	8800
Lower cylinder	3	195	0.31	8430

Table A.4-6 – Example table of material properties 1. Young's modulus and Poisson's ratio from [X], densities have been modified to ensure that the mass of each component matches the values given in the Load Specification [Y].

Material		**Part**	**Material properties**				
#	**Name**		**Name**	**Notation**	**Value**	**Unit**	**Source**
1	Stainless steel 304(L)	Top lid	Young's modulus	EX	195	GPa	[X] p.165
			Poisson's ratio (minor)	NUXY	0.31		[X] p.180
			Density	DENS	8600	kg/m^3	[X] p.201
2	Stainless steel 304(L)	Upper cylinder	Young's modulus	EX	195	GPa	…
			Poisson's ratio (minor)	NUXY	0.31		…
			Density	DENS	8800	kg/m^3	…
3	Stainless steel 304(L)	Lower cylinder	Young's modulus	EX	195	GPa	…
			Poisson's ratio (minor)	NUXY	0.31		…
			Density	DENS	8430	kg/m^3	…

Table A.4-7 – Example table of material properties 2.

Appendix A.4.9.5 FE MESH

This section is mandatory, but the subheadings may change depending on the analysis software. It has to include a comprehensive description of the mesh used i.e. element shapes, order, formulation and settings.

Appendix B.18 gives some guidance on how to report the FE mesh.

Appendix A.4.9.6 BOUNDARY CONDITIONS

Each set of boundary conditions (BCs) shall be described, including degrees of freedom and the coordinate system. The BCs shall also be shown on a figure.

In case the BCs are not constant throughout the analysis, the changes shall be described.

Internal constraints (e.g. coupled equations) shall be described in this section Boundary Conditions or in a separate section. They shall also be shown in one or more figures.

The basic principles of the constraints are part of the justification of the conceptual model, and should therefore be justified in section Appendix A.1.1. Such justification does not need to be duplicated here. This paragraph should focus on their implementation in the FE model, for example: The rotation and translation of surface X is restrained by setting all Degrees of Freedom (DoFs) of all nodes of the surface to zero.

Appendix A.4.9.7 LOAD APPLICATION

This section covers how the defined loads are applied to the FE model.

Appendix A.4.9.8 SOLUTION SETTINGS

The solution settings shall be listed and justified.

Appendix A.4.10 RESULTS

- All results values shall be given with their units. Clear titles shall be given when presenting graphs.
- All relevant results to meet the scope of the seismic analysis shall be given. In case a large number of similar results are calculated, it may be more practical to present some results either in appendices or in attached spreadsheets.
- In case an uncertainty factor is applied to the results to account for the uncertainties of the seismic analysis, it is recommended to give in the Results chapter the result value found in the analysis excluding the uncertainty factor. The uncertainty factor should, however, be mentioned. It is recommended to apply the uncertainty factor to the results in a separate section or in the conclusions.
- Results shall be given corresponding to the design criteria. For example if the failure is membrane plus bending stress, peak stresses are not required.
- When giving reaction forces or moments the direction of a positive reaction force shall be specified or shown in a figure, unless it is obvious. As the direction is dependent on the component the load is acting on, the latter shall be specified. For example, “forces applied by component X to the supporting structure”.
- The point of summation of moments shall also be explicitly stated.
- The coordinate system used for the results shall be specified.
- If modal damping is used for response spectrum or mode-superposition transient analyses, the damping ratio for each mode shall be reported.

For each type of result (e.g. reaction force, maximum displacement, etc.) the result values that are most critical - due to the related load case or the associated location in the structure - shall be compared to the relevant structural design criteria for the verification of the design.

Appendix A.4.11 VERIFICATION OF THE FE ANALYSIS

This section is mandatory if the report concerns an FE analysis. The results of all of the checks listed in Sub-section Appendix A.2.5 and applicable to the analysis shall be reported here.

A comparison shall be made between the results of the FE analysis and those of the alternative calculation(s) – see paragraph Appendix A.2.5.8. An example of how this could be presented is shown below:

Type of Result	**Result of this analysis**	**Result of independent assessment**	**Difference**	**Reference**
Circumferential membrane stress in pipe due to internal pressure	55 MPa	60 MPa	9 %	
End deflection of structure due to gravity	9.8 mm	11.0 mm	11 %	

Table A.4-8 – Example of how the results of an FE analysis could be compared to results from alternate calculations.

The reference can be another document, or a different section in the analysis report, e.g. in case of a hand calculation.

Appendix A.4.12 CONCLUSIONS

- The conclusions shall summarize the most significant findings, and be comprehensible for persons familiar with the design and loads of the system, with an engineering background but not necessarily with expertise in seismic analyses.
- The scope of the seismic analysis should be recalled before writing the conclusions.
- The result values given in the conclusions shall consider the uncertainty of the seismic analysis.
- Results for which the FE model cannot meet accuracy requirements shall either not be reported or be marked as "preliminary" or "best estimates".
- Result values shall not be given without a judgement. Rather than an absolute judgement (e.g. «the design criteria are met»), it is recommended to make quantitative judgements, e.g. the margin to the structural design criteria.
- If appropriate, recommendations for improvements of the structure can be given in the conclusion chapter or in a separate chapter.

Appendix A.4.13 REFERENCES

- All documents that are referenced by the analysis report shall be listed here.
- References shall be stored in IDM or be publically available (e.g. design codes or engineering handbooks).
- References shall be approved.
- References to IDM documents shall include the version numbers.
- An approver and at least one reviewer must be assigned to a reference.
- It is recommended to use the cross-reference or bibliography capability in MS Word.

Appendix A.4.14 APPENDIX A

Appendices can be included if appropriate.

APPENDIX B Technical Guidance and Specific Requirements

This document is not intended to be a manual for how to perform seismic analyses. Nonetheless, this chapter contains some advice and guidance on the topic. The chapter does not introduce any compulsory requirements by itself.

Appendix B.1 Seismic Requirements

Appendix B.1.1 Safety Importance Class

Safety Importance Class (SIC) describes a classification scheme for SSC of ITER that performs a safety function and contributes towards meeting the General Safety Objectives at ITER during incident/accident situations.

Those SSCs assigned a SIC will receive adequate attention at all stages: from design, until operation. The objective is to ensure and demonstrate that they will meet the minimum performance and reliability requirements throughout their lifecycle so that the safety function is provided when required.

SSCs are assigned a particular SIC that is based on the consequences of their failure. Knowing the specific SIC (or at least knowing where to find it) is relevant because it is sometimes referenced in the seismic classification that will be explained here below.

SSCs classified SIC are divided into:

- SIC-1 SSCs are those required to bring to and to maintain ITER in a safe state;
- SIC-2 SSCs are those used to prevent, detect or mitigate incidents or accidents, but not SIC-1 (not required for ITER to reach a safe state).

All other SSCs can be:

- either “non-SIC”,
- or Safety Relevant (SR), which while not being SIC, may have some relevance to safety.

This classification is stated by ITER [20]: in its Annex A, all systems and subsystems with an assigned SIC are included, with the specification on whether they are SIC-1 or SIC-2 and the safety function they provide; additionally Annex B in [20] includes a table with identified SR systems and subsystems.

SR SSCs are not credited in the safety analysis and their failure would not impact any safety function, hence no safety requirements are defined for these SR SSCs in the design phase, however in operation, some requirements, such as periodical maintenance, could be defined.

It is important to note that SSCs classified SIC are a subset of Protection Important Components (PIC), as defined in the French INB Order [41].

In addition to this classification, a methodology has been developed to perform the stress test [26] [27] assessment of the ITER Hard Core Components [21] under scenarios that can lead to cliff edge effect. The ITER Hard Core Components are a sub-set of the SSC classified SIC. The seismic load the Hard Core Components have to resist is defined by the SL-3 response spectrum (see Appendix B.3.2.5).

Appendix B.1.2 Seismic Class

Those SIC SSCs that are required to perform functions that are important for safety in the event of an earthquake shall be designed to withstand the event and maintain the required capability. The collapse, falling, dislodgement or other spatial response of a component as a result of an earthquake shall not jeopardize the functioning of other components providing a safety function.

The seismic design of equipment is based on a group of seismic requirements, which come from three different origins:

- Functional safety requirements for operational or maintenance states, conditions following an accident or other abnormal facility states, in the event of an earthquake. These requirements are described in [49] [50]. The reference earthquake is the SL-2 one (Appendix B.3.2.2).
- French regulation requirements, which are mainly those in Eurocode 8 Part 1 (EC8-1) [43] complemented by the French national annexes [45]; the main goal of these requirements are the protection of people.
- Investment protection requirements. The reference earthquake in this case is the SL-1 one (Appendix B.3.2.4), which is less intense than the SL-2 cited above.

Components and structures are classified to facilitate the design process. The seismic classification principle is based on the safety objective and functional requirements in the event of an earthquake. The seismic classes defined are as follows:

- SC1 (SF): Seismic class one-SF: Structural stability and required functional seismic safety performance maintained in the event of an earthquake. The respect of this level of requirement guarantees the level of safety as throughout the normal operation of the equipment. Nevertheless, taking into account seismic load characteristics, fatigue is not taken into account.
- SC1 (S): Seismic class one-S: Structural stability maintained in the event of an earthquake, i.e. no rupture of piping, no collapse of structures or equipment, limited plastic strain, limited concrete cracking, structural support functions maintained. With this level of requirement, it is possible that a small level of deformation could occur. Consequently, it could be necessary to inspect equipment before re-using it.
- SC2: Seismic class two: Non-damage to SC1 equipment; absence of damage to SC1 equipment for buildings and structures housing and protecting SSCs classified SIC, or to buildings that can potentially damage such structures in the event of collapse, no other requirements regarding structural or functional performance in the event of an earthquake,
- NSC: Non-seismic category. No seismic requirements for nuclear safety. However, all systems, structures and components must respect the requirement that there must be no failure that would prevent a SIC-1 or SIC-2 component from performing its safety function. The French regulation requirements for normal buildings apply.

SSCs belonging to one of the first three classes above will be referred to as “seismic classified”.

SIC systems and components that are required to perform safety functions during or after an SL-2 (SSE) earthquake are identified in Annex 1 of the Seismic Nuclear Safety Approach [22], establishing the seismic requirements in each case.

For the particular case of civil work structures, there is a specific document [23], which refers to [24]. For building structures, the applicable seismic classes are SC1(S), SC2 and NSC. Appendix 1 in [23] indicates the seismic class of ITER facility civil work structures. In this same document [23], in its chapter 5, detailed safety requirements for each civil work structure are detailed.

When a class of equipment such as SC1, (SF) or (S), is encompassed in a set of different equipment class it is important to ensure the propagation of this class, so as not to forget parts or subsets affecting more severe class equipment.

Therefore, it is necessary to represent the most sensitive equipment in such a way that their positioning is most representative in their context. An example is presented in

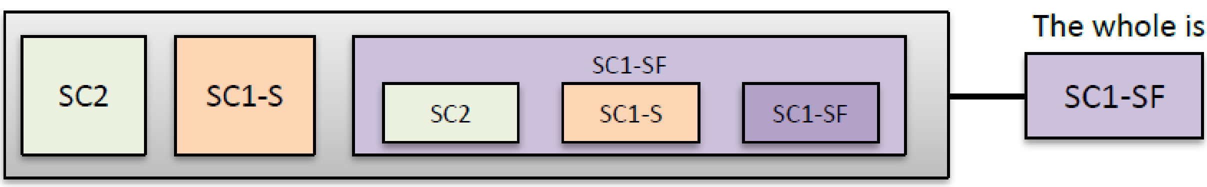


Figure B.1-1 – Example of seismic classification logic.

Appendix B.1.3 Seismic Load Levels

ITER load categories are generally defined in Load Specification [17]: section 4, with the damage limits to be considered in Tables 4-1 and 4-2.

For seismic classified SSCs (SC1 and SC2), the following three levels of seismic load are considered [17].

- **SL-1**: Earthquake that could affect the location during the operating life of the equipment to be qualified. For this type of earthquake, components shall be designed to continue to operate without modification. SL-1 event shall be assumed to occur 5 times at maximum. It is equivalent to the operating basis earthquakes or "OBE". SL-1 is a category II load.
- **SMHV**: Séismes Maximaux Historiquement Vraisemblables (Maximum Historically Probable Earthquakes) that are the most severe earthquakes liable to occur over a period of about 1000 years. SMHV is a category III load.
- **SL-2**: Design Response Spectra (DRS) [22], also called Safe Shutdown Earthquake (SSE), corresponds to the seismic level required by French nuclear practise, which is defined by two spectra: SMS and PALEO spectra. SL-2 is a category IV load. In this event, it shall be demonstrated that all safety functions are maintained.

For NSC SSCs, the seismic requirement is defined by Eurocode 8 (**EC8)** [43], which is not included in the ITER load category. For investment protection purpose, the category II seismic load **SL-1** shall be considered.

Additionally, for Hard Core Components (HCC) [21], a beyond design base seismic level is defined as **SL-3**, which is a category V load.

The specific seismic input for each seismic event (EC8, SL-1, SMHV, SL-2 or SL-3) will be described in Appendix B.3.2.

Appendix B.1.4 Combination of Seismic loads with Other Loads

Load combinations are defined in the Load Specification [17]: section 5, with the possible load combinations listed in Appendix B.

Seismic loads are combined with other loads and some of the combinations could be negligible or enveloped by others. This should be assessed on a case-by-case basis.

Appendix B.1.5 Specific Design Provisions for SSCs

The strength of a component during an earthquake is determined by the two following general principles:

- A component design that helps control its deformation modes in earthquakes, with strong anchoring and fasteners, such that, beyond the conditions required for the design, the component remains stable during an earthquake.
- The choice of appropriate constructive arrangements enabling pre-fracture deformation. These constructive arrangements shall be defined as part of the design to ensure satisfactory post-elastic behaviour from the components. This is generally known as ductile behaviour.

The strength objective for the components shall be determined by complying with the provisions listed below. A good understanding of the design provisions helps to plan the seismic analysis properly.

Appendix B.1.5.1 Separation between Components and/or from the Building

The risk of interaction between two components, one of which is seismic classified and/or the risk of interaction between a component and the building are not allowed in the event of an earthquake. If the separation distance between the components or between the component and the building does not rule out this risk alone, the connections, or fasteners between components or between the component and the building shall be designed to prevent interactions.

Appendix B.1.5.2 Pipes

Pipes that have minimum support for the actual weight have shown excellent strength when facing inertial earthquake loads (i.e. no differential displacements); on the other hand, fracture has been observed when there was a forced movement incompatible with the rigidity of the pipes. Some examples of situations that can cause this differential movement follow:

- Sliding of a heavy component whose piping is connected in a short run to a neighbouring component.
- Differential movement between the points of two components which deform, even elastically, under the force of the earthquake and are connected by pipes.
- Significant movement due to the flexibility of the main pipes, which would not damage these pipes, but may prove to be excessive for branch connections that are weaker and more rigid (i.e. too short or too straight). The same flexible branch connection performs well.

These examples explain why it is necessary for pipes (or pipe networks) to have as much flexibility as possible to withstand earthquakes. It can be achieved by the presence of expansion joints and elbows throughout their runs.

The FEMA 414 guide [72] provides guidelines on how to attach pipes to a building in order to minimize earthquake damage.

Appendix B.1.5.3 Pumps, Compressors, Rotating Machinery and Similar

These types of components, which already undergo heavy acceleration during normal operation, are intrinsically not impacted by earthquakes, but they could be affected by components that ensure indispensable auxiliary functions (emergency batteries, diesel generators, etc.). One cause of vulnerability resides in the required connections to the environment which shall demonstrate adequate flexibility (electric cables, fuel lines, cooling water, drains, etc.). The need is of special importance for equipment set on isolation systems.

The FEMA 414 guide [72] includes a detail on how to attach pipes with in-line equipment, like pumps.

Appendix B.1.5.4 Valves

This type of component is generally resistant to earthquakes. Nevertheless, cases of loss of service associated with degradation of the (manual or motorized) operating mechanisms have been observed. In addition, valves directly supported on civil works are less sensitive to earthquakes than those supported by pipes. On small pipes, the center of gravity of the motor may present quite a strong eccentricity. This could lead to excessive deformation making it inoperable. It has been noted that the configuration where the motor and pipes are horizontal is less effective than where the motor is vertical to the pipes. This is due to the effects of actual weight. The risk of shock to maintenance devices and valve motors on neighbouring structures and components should also be monitored. These impact risks are often offset by the flexibility of pipes that cause major movement during earthquakes.

The FEMA 414 guide [72] includes a short section devoted to valves and valve actuators.

Appendix B.1.5.5 Cable Runs

These components perform very well during earthquakes. The only incidents encountered are associated with anchoring failure. Cables may break. They may also damage neighbouring components because of their weight in the event of loss of balance, falling or excessive support flexibility.

The FEMA 413 guide [73], provides guidance on how to attach electrical equipment to a building to minimize earthquake damage, including a section for cables that describes three ways of assembling a cable connection by using: bolts, ferrule assemblies or wire rope grips

Appendix B.1.5.6 Batteries

Examples of batteries simply moved about or fallen over during an earthquake are quite common. This phenomenon, when it happens, has a tendency to occur throughout the location. Another risk involves batteries set laterally. Even if they do not fall over they may be damaged and put out of service by hitting one another. For constructive arrangements, proper behaviour is observed when the battery stacks are held in place by well-designed devices that prevent falling, and when the batteries in the same row are held in place by fasteners at the top and bottom. Some guidance on the attachment of batteries can be found in the FEMA 413 guide [73], depending on how the equipment is installed (i.e. directly on the floor, directly to the wall, roof-mounted, isolated… etc.)

Appendix B.1.5.7 Electrical Cabinets

Electrical cabinets often fall over if they have not been anchored to the ground. Also, frequently the doors open and components set on wheels or in unprotected racks could fall out. Neighbouring cabinets banging into each other, because they are not held in place, seems to be the main cause of damage to relays.

It should however be noted that this type of damage is relatively easy to avoid with simple constructive arrangements like proper anchoring to the ground. Most recent cabinet models have proven to be much better when they have stiffeners on the back and in the corners. Some guidance on the attachment of electrical cabinets can be found in the FEMA 413 guide [73].

Appendix B.1.5.8 Anchoring and Attachments

The effects of actual earthquakes on facilities show that nearly all earthquakes, even relatively small ones, reveal problems with anchoring, stowing, and tie-down in general.

Undersized anchor bolts could result in either assimilating a seismic load with a static force or in underestimating the dynamic force by not taking into account the actual non-linear behaviour of the component and anchorage. Even by adding margin, the normal practice is to neglect the effect of cumulative damage from cyclic loads on concrete, a significant force particularly in shearing.

When sizing seismic components, the resistance capacity of their connections to structural elements must be evaluated considering the condition of cracked concrete, unless it can be demonstrated that the concrete does not crack under seismic load. As a consequence, when sizing components we recommend choosing bolts approved for cracked concrete.

Both FEMA 413 [73] and FEMA 414 [72] include a section (in both documents the same section) on how to proceed with the anchor installation for attaching electrical equipment and pipes.

Appendix B.1.5.9 Provisions for Operability Requirements

Complying with a component operability requirement during and/or after an earthquake can never be guaranteed solely from calculations. As a result, seismic qualification of a component shall be supplied by the manufacturer. In all cases, it is important to ensure that the seismic qualification properly considers the seismic loads under the various operating conditions of the component defined in the Equipment Specification, and the component's installation conditions inside the facility (support, anchoring, imposed loads, etc.). Before this seismic qualification can take place, a qualification specification describing the test program and the criteria to be met shall be drafted, following the guidance of experimental seismic qualification [28].

Appendix B.2 Applicable Methodologies for Seismic Analysis

Appendix B.2.1 General Overview

Appendix B.2.1.1 Static and Dynamic Methods

In the seismic codes, it is common to find both methods, simplified equivalent static analysis, allowed in some circumstances, and dynamic methods that can entail deferent degrees of sophistication.

In general for buildings, a Response Spectrum analysis will be the reference method, which is a linear dynamic analysis in the frequency domain; in some particular cases a time domain analysis will be required for the derivation of FRS.

Appendix B.2.1.2 Time Domain and Frequency Domain

Generally speaking, dynamic calculations can be performed in the time domain or the frequency domain. Time domain calculations have the time as a physical value and both, input and output quantities, are expressed as a function of time. Time domain analyses can consist of a full direct integration over the time of the complete model that idealizes the situation to be studied, which is needed if nonlinearities are to be accounted for; alternatively the analysis can be a mode-based transient response one in which the solution is offered as a linear combination of a selected number of eigenmodes, which only permits accounting for linear situations.

Frequency domain calculations rely on the eigenmodes and natural frequencies of the structure. Both input and output are functions of frequency. There are several procedures:

- Steady-state harmonic response analysis: A steady-state dynamic analysis based on the eigenmodes of the system; the output is a linear response to harmonic excitation.
- Response spectrum analysis: the output is the estimated peak response of a system to the input response spectrum, which is a function of frequency.
- Random response analysis: this type of procedure is used when the structure is excited continuously by a random loading that can be expressed statistically, often in terms of a Power Spectral Density (PSD) function. The output is expressed based on the eigenmodes in terms of statistical quantities such as the mean value and the standard deviation of the output quantities.

Most of the dynamic calculations will follow the response spectrum methodology, explained in detail in section Appendix B.2.5.1.

Appendix B.2.1.3 Linear and non-linear methods

A linear analysis is that in which the model has constant properties (geometry, mechanical properties and boundary conditions (BCs)) during the load application. Strictly speaking everything in life is non-linear, so a linear analysis is justified when changes with respect to geometry, mechanical properties or boundary conditions are considered to be of negligible impact for the results of interest.

Frequency domain analyses, which are based on eigenmodes, are linear by definition (i.e. the eigenmodes contain the geometry, mechanical properties and boundary conditions and remain constant throughout the analysis). So the question on whether the consideration of nonlinearities is pertinent is in the context of static analysis, or dynamic analysis in the time domain.

The presence of contacts in the model is a source of nonlinearity (it can be understood as a change in the BCs) even if the rest of the model aspects can be represented with linear behaviours.

Appendix B.2.2 Modelling Principles

Appendix B.2.2.1 General Ideas

SSCs need to be modelled to perform a seismic analysis. This modelling has to be a reliable representation of stiffness (Appendix B.2.2.2), mass (Appendix B.2.2.3) and energy dissipation capacity (Appendix B.2.2.4) of all the SSCs in order to reproduce its dynamic response. In addition, if needed the model needs to take into consideration the effects of supported equipment (Appendix B.2.2.5) and hydrodynamic effects of fluids interacting significantly with the SSCs (section Appendix B.2.9).

The modelling has to have an adequately detailed representation of the geometry, the mass and the mechanical properties that permits to properly reproduce the dynamic behaviour. A finite element model could generally meet this requirement.

Models may be two or three dimensional and may include global or local behaviour based on the component's characteristics and requirements.

Appendix B.2.2.2 Stiffness Modelling

The adequate representation of global stiffness of the model will depend on the following aspects:

- Geometry, which has to be a true representation of the actual geometry. Simplifications need to be duly justified (i.e. point masses).
- Mechanical properties.
- In case of a FEM, the type of finite element has to be adequate for the type of calculation as well as the type of demands that the structure will undergo.
- Boundary conditions represent the actual situation of the SSCs and their interactions.

Appendix B.2.2.3 Mass Modelling

The modelled masses are determined by the geometry represented and the values of material density. The mass distribution of the FEM must be consistent with the reference mass distribution of the system which is modelled: the values of mass and the position of the center of gravity (CoG) of each sub-system must be similar.

If certain component parts are represented by concentrated mass (for example valves mounted on pipes), each of these masses shall be positioned on its own CoG.

Appendix B.2.2.4 Energy Dissipation

Energy dissipation comes from the distributed behavior of the materials making up the components and/or the special local devices designed for this purpose (e.g., dampers).

In terms of component linear behavior, the overall dissipation capacity of the component is modelled conventionally by viscous damping.

Appendix B.2.2.5 Supported Equipment

In addition to considering the masses of supported equipment, the possible dynamic interactions of this equipment with the components that support may need to be considered for seismic analysis of the supporting components.

If the mass of equipment is significant compared to the mass of the supporting component analysed, the coupling between the supporting structure and the equipment shall be considered. See section Appendix B.2.6 for more details on coupling criteria.

Appendix B.2.2.6 Modal Analysis

The first step for conducting a modal analysis is the extraction of natural frequencies and their corresponding mode-shape. For each natural eigenmode, at least the following information should be gathered:

- Natural frequencies. All the natural frequencies below the cut-off frequency should be calculated.
- Mode-shape.
- Modal mass in each of the three spatial directions and for each eigenmode. In particular, it is useful to know the percentage of mass that is mobilized by each mode in each spatial direction.

There is an approximate procedure for estimating the first natural frequency: the Rayleigh coefficient method.

- For a SDOF system: $f = \frac{1}{2\pi}\sqrt{\frac{k}{m}} = \frac{1}{2\pi}\sqrt{\frac{g}{\delta}}$
- For a MDOF system: $f = \frac{1}{2\pi}\sqrt{\frac{a_c}{d_{rep}}}$; where d_{rep} is the representative displacement under the imposed static acceleration a_c, which can be estimated by the maximum displacement u_{max}, as $\frac{u_{max}}{1.5} < d_{rep} < u_{max}$.

Appendix B.2.3 Linear Equivalent Static Methods

Appendix B.2.3.1 General Considerations

Linear static methods using simplified models and seismic loads may be used to consider the effects of an earthquake on design of components and/or their parts. These methods are mainly used for design of supports and their SSCs. Some general conditions for the applicability of the linear static method in its traditional form (i.e. single force or a uniform acceleration) follow:

- The SSC has a regular framing;
- The distribution of stiffness and mass is homogeneous;
- The contribution from modes higher than the first dominant one is small.

In case of simple but irregular SSCs, or the rest of cases not included in the previous conditions, the method may be applied if its adequacy is demonstrated with parametric analyses. Specific considerations are given in Appendix B.2.3.2 for applying this method to irregular structures.

Static methods consist of uniformly imposing on the SSC an equivalent static acceleration.

$$a_{eq} = \alpha_{st} a_c$$

Where:

a_c is the maximum spectral value of the input ARS (acceleration response spectrum) within the frequency range above the natural frequency of the relevant main mode of the SSC.

- The relevant main mode shall be considered the first mode that gives significant contribution to the response of interest.
- If the relevant main mode is unknown, a_c is considered the peak spectral value of the input ARS.
- If the SSC is so rigid that the natural frequency of any main mode is always above the spectral cut-off frequency, a_c is considered ZPA (Zero Period Acceleration).
- If it is justified that the SSC is so rigid that the natural frequency of any main mode is always above certain frequency, a_c can be considered the maximum spectral value of the input ARS within the frequency range above that frequency.

α_{st} is the amplification factor to take into account that the SSC does not behave as a SDOF (single degree of freedom) system.

- α_{st} will be 1.5 in a general case.
- If the SSC is supported on a flexible single support so that the first main mode is dominant, α_{st} can be set to 1.0.
- If the SSC is so rigid that the natural frequency of any main mode is above the spectral cut-off frequency, α_{st} can be set to 1.0.

To calculate the interface load, the following guidance and criteria are offered to apply the equivalent static load to the SSC:

- If the SSC is so rigid that the natural frequency of any main mode is always above the spectral cut-off frequency, the total equivalent static load $F_c = a_{eq}M_{ssc}$ can be applied on the center of gravity, with α_{st} equal to 1.0.
- If the SSC is a cantilever with periodic response, the total equivalent static load can be applied at a distance from the base of 1.2 times that of the center of gravity.
- If there is a clearly predominant mode with an effective mass of at least 90% of the total mass, and only the interface loads of the SSC are interested, the spectral acceleration can be proportionally distributed according to the modal shape, with α_{st} equal to 1.0.
- In the rest of general cases, the equivalent acceleration a_{eq} can be applied uniformly all over the mass; hence, the equivalent force will be distributed along the SSC proportionally to the mass.

In general, a mathematical model of the SSC may be used to determine the distribution of the lateral loads to elements of the lateral-force-resisting system.

In case there is a previous dynamic analysis (for instance the response spectrum analysis with a finite element model), and equivalent static analyses of particular parts need to be performed, the equivalent static force F_c can be derived from this existing dynamic analysis. For the construction of the global dynamic analysis, the potential coupling or dynamic interaction of the equipment with the supporting structure need to be identified following the modelling principles in Appendix B.2.3.2.

Appendix B.2.3.2 Considerations for Irregular Structures

Some considerations, guidance and criteria are offered in the present section in order to make the equivalent static method valid for irregular structures, or those having significant influence of more than one mode.

Structures with irregular framing, or non-homogeneous distribution of stiffness and mass, may have the first eigenmode with a shape that does not have the same qualitative deformation as the general deformed shape that results in the global response spectrum analysis; hence it is not possible to find a single load that is representatively enough for the equivalent static analysis. This is the case, for instance, for structures whose first eigenmode is mainly torsional.

For such cases, the application of the load for the equivalent static analysis should follow the pattern of the first modal shape or of the most relevant eigenmodes: the load direction at each point shall be that of the displacement in the first mode and the amount proportional to the distribution of mass and to the amount of displacement. The applicability of this option remains subject to the fact that the contribution from modes higher than the first dominant one is small.

In more general cases, where there are probably several modes influencing the dynamic response, the procedure should be oriented to perform a study on a part-by-part basis. From a global dynamic analysis (typically a response spectrum method), contours of ZPA can be obtained as well as FRS at points where equipment is attached. The objective is to divide the global model into sub-systems.

For small components fixed to the main structure, the information provided by the FRS at the interfaces is probably enough and it can be directly the input for the seismic study of the component.

For large components, the division in subsystems can be conducted on the basis of the maps of ZPA, identifying parts for which the found ZPA is relatively uniform or has a distribution that can be easily reproduced (i.e. linear). The interface loads at the boundaries of each of the parts should be extracted from the reference calculations.

Each of these (large) subsystems can be studied individually with its own equivalent static analysis: an inertial force for the subsystem can be concluded; interface forces have to be controlled along the interfaces where the cut with the main system has been done. Complementary interface loads should be eventually added when the subsystem is support of another system which is not represented in the model.

Appendix B.2.3.3 Main Steps for Determining the Equivalent Static Loads

A general outline is presented below for deriving from a global model the sets of static loads that can be employed in the seismic design or verification of equipment.

- Gain insight into the behaviour of individual systems to be studied for simple load cases, identifying:
    - o Spatial distribution of loads between interfaces for hyperstatic system;
    - o Key parameters such as mass, centre of gravity, geometry, load paths.
- Identify coupled and uncoupled equipment, and include the needed one in the global model, according to the principles in Appendix B.2.2.5.
- Determination of the eigenmodes (natural frequencies and mode-shape),
- Conduct a dynamic analysis (a response spectrum analysis and/or time-history analysis), see section Appendix B.2.5.
- Extract the main results: relative displacement, absolute acceleration, local and global interface loads, information on the concomitance (or non-concomitance) of loads, etc.
- Split the equipment into sub-systems for which the acceleration is almost uniform (or known with analytical formula – linear distribution).
- Determine values of acceleration (inertial loads) coherent with the maps of ZPA and conservative with respect to the local and global interface loads.
- For large components, the loads can be defined with the inertial loads and the distribution of local interface loads.
- For small components fixed to the main structure, FRS at the interfaces can be directly the input data for the analysis.

Appendix B.2.4 Non-linear Equivalent Static Methods

These methods are acceptable as long as they consider the non-linear behaviour of components keeping consistency with the functional requirements that are attributed to them (section Appendix B.1.2).

The non-linear behaviour of the component can be accounted for (with static analysis) the two following methods [54]:

- The introduction of a behaviour factor, known also as equivalent non-linear method.
- A push-over analysis.

Appendix B.2.4.1 Approximate Non-linear Method

The equivalent non-linear method considers the material non-linearity by means of a behaviour factor that reduces the seismic forces that have been obtained with a static linear analysis. This method is acceptable for configuring components whose sole behaviour requirements are the absence of interaction with neighbouring components and/or civil works, or the unit's stability.

Appendix B.2.4.2 Push-over Analysis

In a push-over analysis, the seismic response is estimated by applying a monotonically increasing force, consistent with the main mode shape. The non-linear behaviour of the materials or other types of relevant non-linearity are included in the calculation. In this type of study special attention needs to be paid to the following aspects [54], which will be explained in detail later on in this same section:

- The applied loads have to be a good representation of the seismic action, especially when the behaviour of the structure is influenced by more than one single mode. A method for assessing whether this is the case is explained below.
- The consequences of the potential development of initial plastic deformations.
- The response in the three different directions is studied independently, but the seismic action itself acts in the three directions.
- Different load cases will most likely have to be considered.
- Uncertainties in the evaluation of the material properties (i.e. rigidities, ultimate strength, ultimate deformation, etc.) need to be considered as well as their impact on the results.

To determine if higher modes of response are significant, a procedure similar to the one indicated in ASCE 4 [53] can be followed:

- A first response spectrum analysis shall be performed following the guidance in section Appendix B.2.5.1.
- A second response-spectrum analysis shall be performed considering only the first mode.
- Both previous results are compared. If either result of interest (typically the shear force or global overturning moment at the base or any inter-story location of interest) considering multiple modes is more than 30% greater than the values calculated using only the first mode, higher modes shall be considered significant.

The French nuclear regulator [54] and ASCE 4 [53], both related to nuclear installations, include the pushover analysis as a valid option. Besides, ASCE 4 [53] refers to ASCE/SEI 41-13 [55] and also to FEMA 273 [67] and 274 [68].

The three elements needed for conducting a pushover analysis are:

- model representing the structure, system or component;
- load pattern;
- target displacement.

The model (typically an FEM) should account for various types of non-linearity: those related to geometry and those in relation with the material constitutive model. If some non-linearity is discarded, it should be verified in the results the validity of the hypothesis.

Lateral loads shall be applied to the mathematical model in proportion to the distribution of mass at each level. At the same time the vertical distribution of these forces shall be proportional to the shape of the fundamental mode in the direction under consideration (i.e. multiplying the mode shape by the mass matrix). More than one seismic force pattern has been used in the past as a way to bound the range of actions that may occur during actual dynamic response. Research in FEMA 440 [69] has shown that multiple force patterns do little to improve the accuracy of nonlinear static procedures and that a single pattern based on the first mode shape is recommended [55].

With the model and the load defined, the calculation can be conducted; the key result to be derived is the relation between base shear force and lateral displacement of the control node, which, in a general case, shall be located at the centre of the top roof or cover. An example of pushover curve is presented in Figure B.2-1.

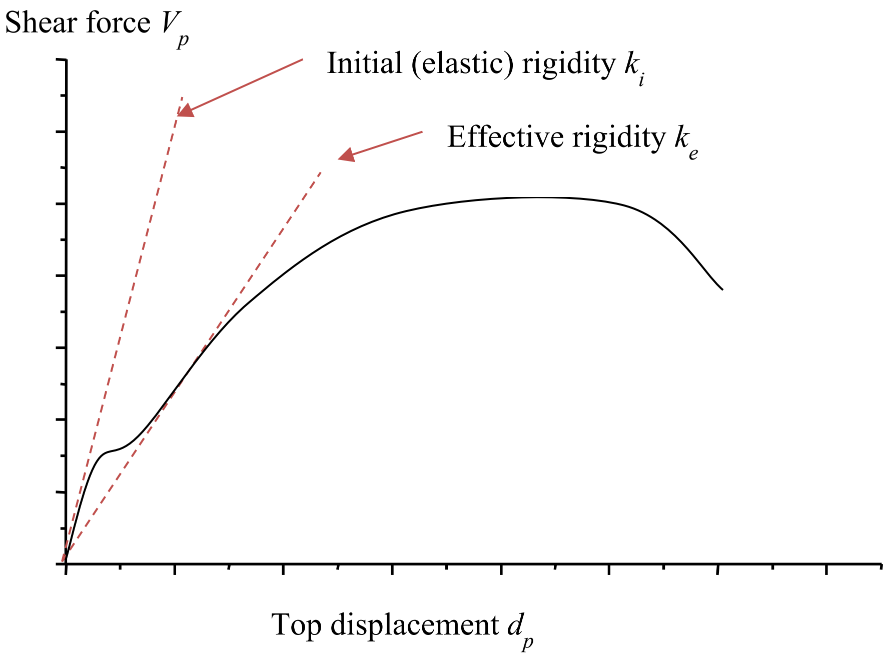


Figure B.2-1 – Example of pushover curve.

The information of the pushover curve, which is essentially a way of defining the structure capacity, has to be compared with the seismic demand. A way of doing this is the capacity spectrum method. This method provides a graphical representation of the expected seismic performance of the structure and finds the intersection of the structure's capacity pushover curve with the response spectrum representation of the earthquake's displacement demand on the structure. The intersection is the performance point, and the corresponding displacement is the estimated displacement demand on the structure for the specified level of hazard [64].

Demand and capacity need to be represented in the same plot; hence, abscissae and ordinates need to be homogeneous.

The demand (response spectrum) needs to be represented in the form of Acceleration Displacement Response Spectra (ADRS). This is achieved by considering the relationship between pseudo-acceleration S_a and pseudo-displacement S_d:

$$S_a(T) = \omega^2 S_d(T) = \left(\frac{2\pi}{T}\right)^2 S_d(T)$$

The ADRS can be plotted as a parametric curve: there is a pair (S_d (T), S_a (T)), for each value of the period T, that can be represented in a plot such as the one in Figure B.2-2. The period or frequency that corresponds to each point can be concluded from the slope of the line from that point the origin:

$$\omega^2 = \left(\frac{2\pi}{T}\right)^2 = \frac{S_a(T)}{S_d(T)}$$

In order to transform the pushover curve, which will be typically from the calculation in terms of base shear V_p and top displacement d_p into equivalent spectral acceleration S_a and spectral displacement S_d, the following equations can be employed:

$$S_a = \frac{V_p}{M_{SSC}\alpha_1}$$

$$S_d = \frac{d_p}{PF_1\phi_{p1}}$$

where

α_1 is the modal mass coefficient for the first natural mode

M_{SSC} is the mass of the structure being studied

PF_1 is the modal participation factor for the first natural mode

ϕ_{p1} is the amplitude of the top point (where displacements are retrieved in the pushover) for the first natural mode

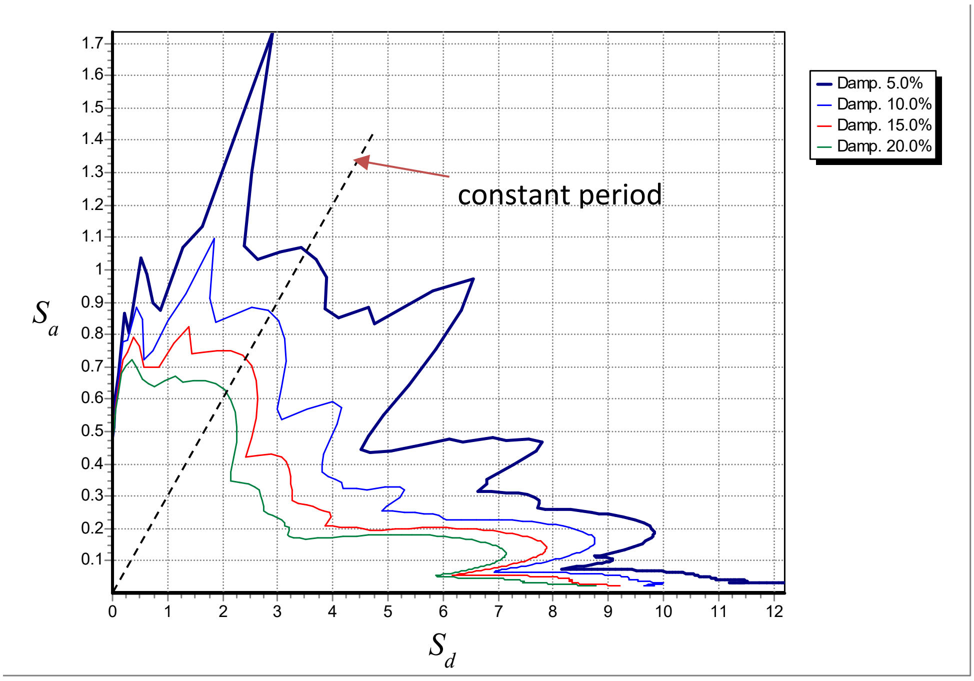


Figure B.2-2 – Example of ADRS.

With the demand (response spectrum) and the capacity (pushover curve) plotted in the same graph, something similar to the top plot of Figure B.2-3 will be obtained in which the points where the capacity crosses the demand need to be identified (blue circles).

In order to find the performance point, a representation like the one on the second plot of this same figure needs to be conducted. This new plot has the same abscise, the spectral displacement, and the fraction of critical damping in the ordinates. The identification of the fraction of critical damping according to the demand and the capacity for identified points (blue circles) is to be calculated next. The critical damping according to the demand response spectrum can be directly concluded from the response spectrum plotted in the top graph; for determining the fraction of critical damping according to the pushover curve some hypothesis and mathematical model needs to be considered. The equivalent viscous damping associated with a maximum displacement can be derived with an idealisation as the one presented in Figure B.2-4 [64].

Once the fraction of critical damping for the spectral displacements of the points of intersection is identified for both, demand (brown curve) and capacity (blue curve) (bottom part of Figure B.2-3), the displacement for which they intersect is the performance point.

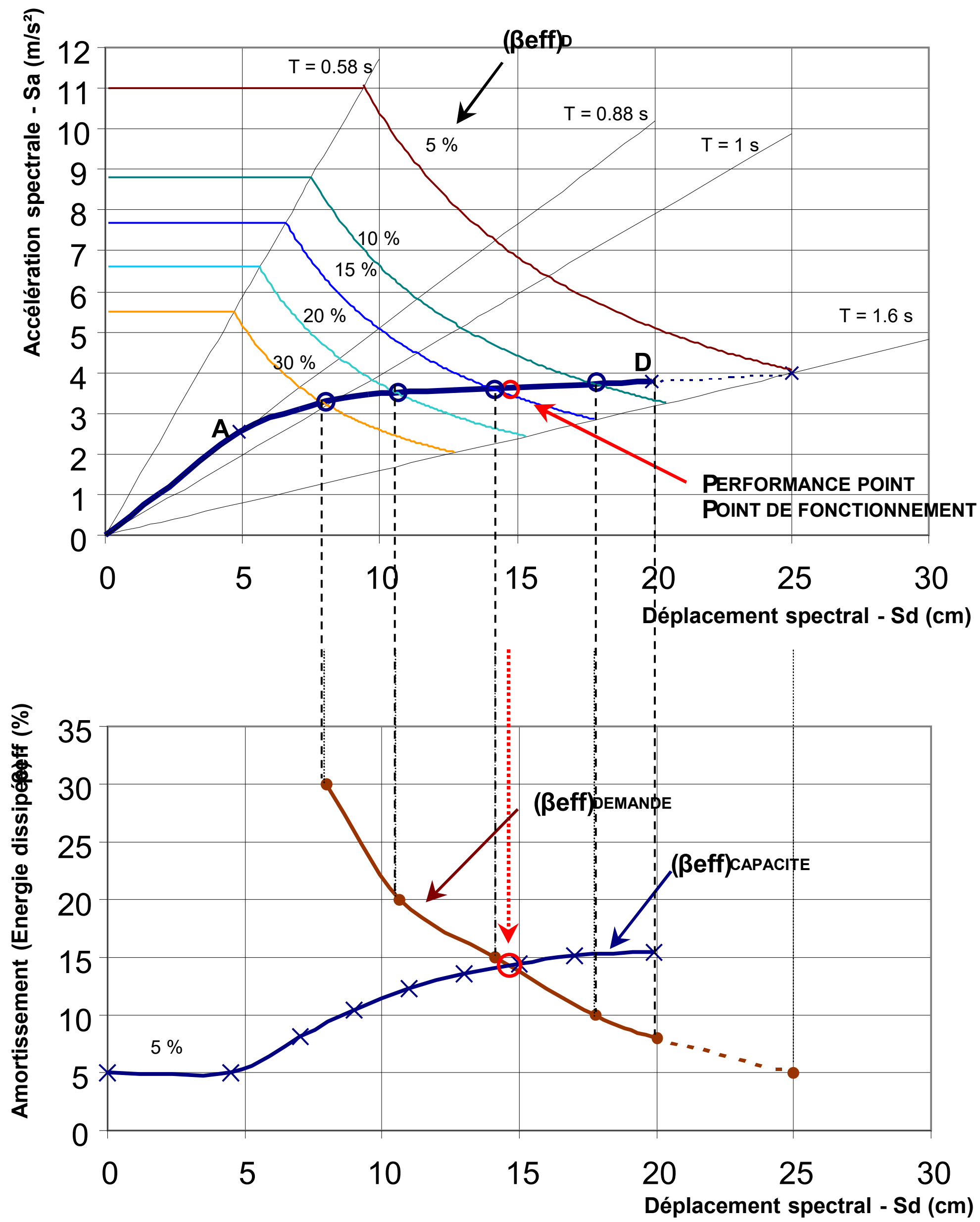


Figure B.2-3 – Pushover analysis and determination of performance point.

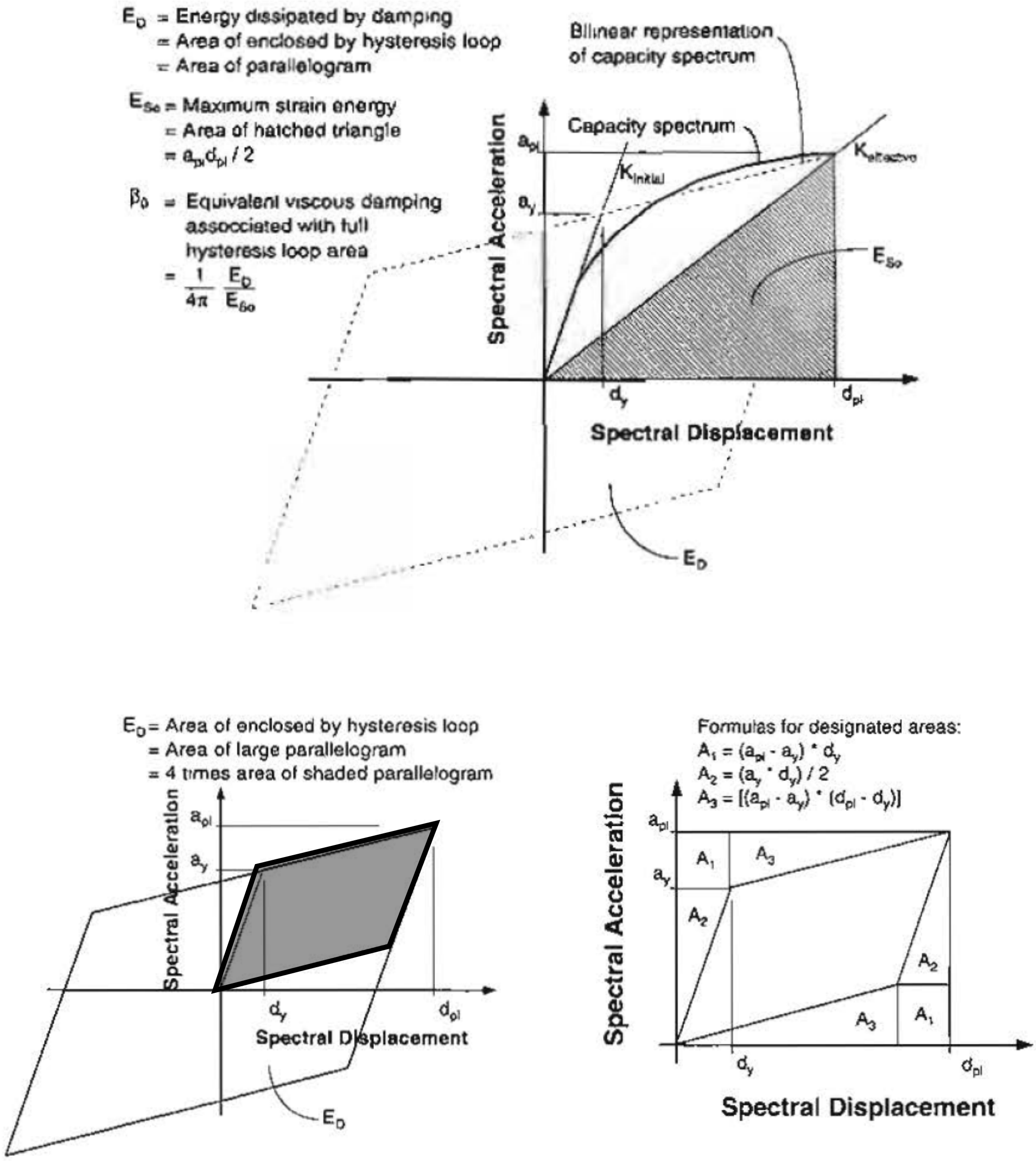


Figure B.2-4 – Derivation of damping for capacity curve.

Appendix B.2.4.3 Sequentially Non-linear Analysis

This type of analysis is to be applied when facing a non-linear calculation in which the governing sources of non-linearity are concentrated at localised points, but the global response keeps being linear. The most typical example is that of hinges that develop in a pipe system [65] (explained in detail in section Appendix B.6.2.6), but the concept may be applied to other SSCs.

The general outline of the process is as follows:

1) A preliminary linear analysis is needed in order to identify the points where the linear response is exceeded and hence they are candidates where this idealisation will be introduced.
2) A new model is constructed in which points identified in the previous step are substituted by a release of the corresponding degrees of freedom (DOFs) and the maximum forces, concomitant with the released DOFs, that the structure can develop at those points are applied.

3) With such a new model a new calculation is performed. Two aspects are to be verified:
   - The level of displacement and/or rotation at points with non-linear behaviour is in agreement with the ductility that can be assured.
   - There are no new points that exceed the linear behaviour. In case there are, another iteration, going back to point 2) above, needs to be performed.

Appendix B.2.5 Dynamic Methods

Appendix B.2.5.1 Response Spectrum Analysis

Appendix B.2.5.1.1 General Description of the Procedure

Response-spectrum analysis (RSA) is a linear-dynamic analysis method which evaluates the maximum absolute value of a scalar physical variable that is linearly dependent on the amplitude of the modes.

The response spectrum represents the maximum amplification experienced by a linear oscillator as a function of its natural frequency and damping ratio for a particular input motion. If it is not explicitly stated, the response spectrum is always the acceleration response spectrum in this document.

The different steps of the calculation go through the obtaining of:

- Eigenmodes, natural frequencies, modal damping and modal participation factors in each direction.
- Maximum amplitude of the response for each mode according to the response spectrum and the participation factor of the mode in the direction under consideration.
- The response to one direction of the earthquake, by combining the previously calculated modal responses.
- Absolute value of the overall response including all earthquake directions by adequate combination of the responses in the three earthquake directions.

Generally speaking, the method described in U.S NRC RG 1.92 [56] is recommended. However, practically, the procedure describe in the following sections is acceptable, which is in agreement with the requirements in ASN/Guide/2/01 [42].

Appendix B.2.5.1.2 Modal Analysis

The obtaining of eigen-modes is oriented to the modal combination procedure that will be followed (see section Appendix B.2.5.1.4). All modes with a frequency below the cut-off frequency f_c will be obtained.

The cut-off frequency f_c is defined as the frequency above which an oscillator follows the motion of its support [42]. Practically, the f_2 in Figure B.2-5 and Figure B.2-6 (from [56]) can be considered as f_c, above which only rigid response is expected. The value is chosen as the frequency where spectral curves with different damping ratios converge.

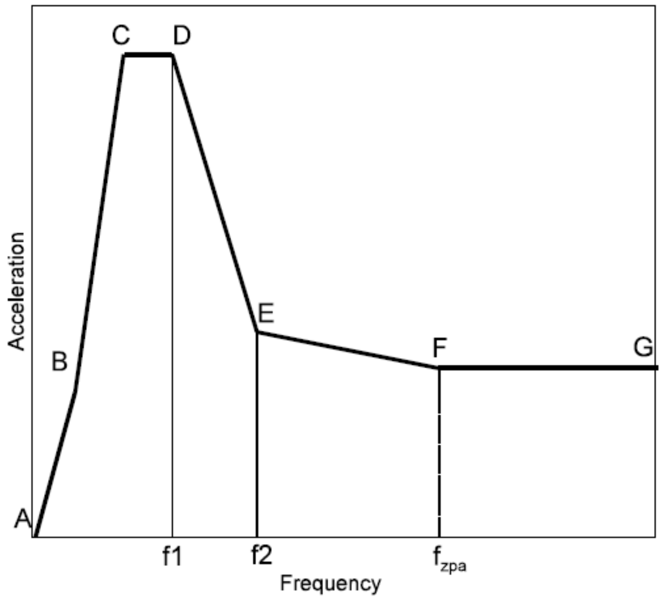


Figure B.2-5 – A narrow-banded response spectrum.

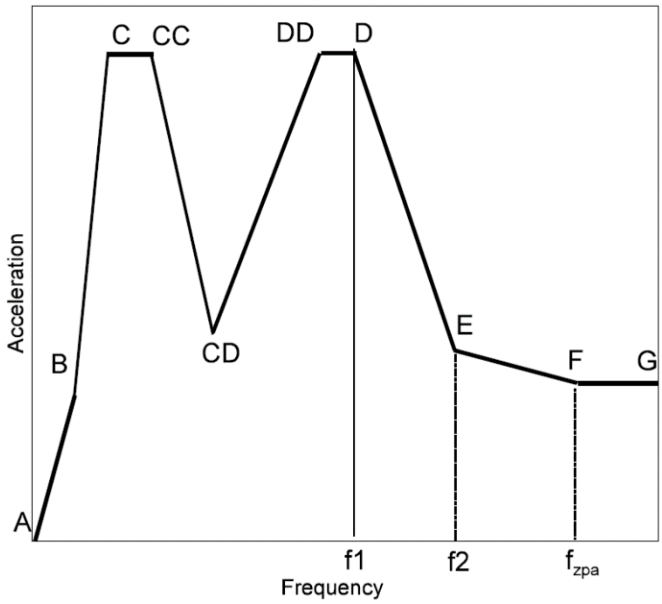


Figure B.2-6 – A multiple narrow-banded response spectrum.

Apart from the mode shape and the natural frequency, the modal participation factor in each direction will be needed for conducting the response spectrum analysis.

Additionally, given the modal combination procedure that will be followed (section Appendix B.2.5.1.4), for each degree of freedom (DOF) included in the dynamic analysis, the fraction of DOF mass included in the summation of all modes needs to be determined.

There is an additional "mode" to be calculated: the pseudo mode that represents all modes with frequencies that are equal to or higher than the cut-off frequency f_c. The rigid response to the pseudo mode is calculated by performing a static analysis for an applied load that equals the missing mass multiplied by a constant acceleration value, which is the maximum between the spectrum ZPA and the spectral value at f_c, which is often a little higher than ZPA.

The eigenmodes can be obtained in a prestressed system with the "stress stiffening" effect included (the initial stress matrix is included if the base state step definition included nonlinear geometric effects), which may be necessary in the dynamic study of preloaded systems.

Follower forces (such as concentrated loads that rotate with the nodal rotation or pressure loads) lead to a non-symmetric load stiffness; an assessment on the potential impact of this fact on the results should be discussed. If the effect of the follower forces is expected to be relevant, eigenvalue extraction should be performed considering the non-symmetric stiffness matrix.

Appendix B.2.5.1.3 Damping

The response spectra to be used in the response spectrum analysis correspond to a certain value of damping ratio to the critical damping (see section Appendix B.3.2 and Appendix B.3.4). This damping is representative of the one that corresponds to the structure or equipment being studied.

Values of damping admissible for different materials at ULS are indicated in section A.3.1 of the ITER Structural Design Code [25], namely:

- Reinforced concrete: 7%
- Pre-stressed concrete: 5%
- Bolted Steel: 7%
- Welded steel: 4%

The above values are consistent with those indicated in the document of Load Specification [17] for SL-2 and SMHV earthquakes. In this same document, damping values for equipment inside buildings are indicated. The values are the ones in the RG 1.61 [57].

Appendix B.2.5.1.4 Modal Combination

The modal combination is to combine the responses to all the modes, including modes with natural frequencies below f_c and the pseudo mode, to calculate the maximum response to a component of input seismic motion (x, y or z).

Complete Quadratic Combination (CQC) method is recommended to combine the maximum modal responses to all the modes with natural frequencies below f_c, to consider the correlation between the maximum modal responses. Square Root of Sum of Squares (SRSS) method is optional if it is justified that the modes are sufficiently separated so that the correlation is negligible.

SRSS method is recommended to combine the response to the pseudo mode with the combined maximum response to all other modes. The result is the maximum absolute response to a component of input seismic motion.

Appendix B.2.5.1.5 Spatial Combination

The spatial combination is to combine the responses to all components of seismic motion in three directions, x, y and z, assuming they are uncorrelated to each other. It is not dependant on the method used to obtain these responses, such as RSA, transient analysis, equivalent static analysis, etc.

SRSS method is recommended to perform the spatial combination:

$$R = \sqrt{R_x^2 + R_y^2 + R_z^2}$$

New-mark method is optional:

$$\begin{aligned} R &= \max\left(|R_x| + 0.4|R_y| + 0.4|R_z|; |R_y| + 0.4|R_z| + 0.4|R_x|; |R_z| + 0.4|R_x| + 0.4|R_y|\right) \\ &= 0.6 * \max\left(|R_x|; |R_y|; |R_z|\right) + 0.4 * \left(|R_x| + |R_y| + |R_z|\right) \end{aligned}$$

Appendix B.2.5.2 Transient Methods

Appendix B.2.5.2.1 General Description of the Procedure

The various steps in a transient method are listed as follows:

- the selection of accelerograms corresponding to the seismic movement used for the design;

- the resolution of movement equations over time, which may be different depending on the methodology chosen (i.e. mode based or full integration).

Special attention is given to the variability of results based on accelerograms. A minimum of tree transient calculations shall be performed with independent accelerograms.

Appendix B.2.5.2.2 Transient Methods for Linear Models

The transient response of a linear model to seismic excitation represented by one or more accelerograms is determined by the integration of either of the following:

- Decoupled differential equations obtained by projecting the structure's dynamic behaviour over the modal base (i.e. modal based). In this case, the representation of internal damping is the same as for spectral methods and it is assigned on a mode-by-mode basis (section 7.5.1.3).
- The equation for movement expressed in the natural physical base of the model. In this case, internal damping is defined by a damping matrix C representing the physical phenomena (viscous damping proportional to velocity). The internal damping matrix can be determined by following the Rayleigh formulation.

The modal based transient solution is a cost-effective option since the effective number of DOF in the model is reduced to the number of modes being considered. The obtaining of the modes follows the same indications that have been indicated in section 7.5.1.2. The pseudo mode considering the missing mass is also calculated in this case. Only modes with a frequency below the cut-off frequency f_c participate in the modal solution; the missing mass contribution, scaled to the instantaneous input acceleration, is treated as an additional mode in the algebraic summation of modal responses at each time step and the missing mass contribution is considered for all degrees of freedom. This is in agreement with RG 1.92 [56]. The time step for the transient analysis should be sufficiently short to well represent the dynamic response of the structure (i.e. for a seismic load with a cut-off frequency of 33 Hz, it is recommended to consider a time step shorter or equal to 5 ms).

Appendix B.2.5.2.3 Transient Methods for Non-linear Models

Using non-linear calculations remains a sensitive issue and requires relatively complex testing. These calculations are used to represent the behaviour for which the transient linear methods are not pertinent or for additional assessment when required.

If the non-linearity involved does not have a dissipative character (such as geometric non-linearity in a component whose parts remain elastic), the damping matrix of the model is calculated like linear models (see sections Appendix B.2.5.2.2 and Appendix B.2.5.1.3). On the other hand, if the dissipative character is taken into consideration in the non-linear behaviour of components (such as plastic deformations or contact friction with relative displacement, for example), the contribution of this phenomenon shall not be considered in the damping matrix which shall be calculated to represent only "structural" damping. It should be clear in the model how damping is modelled, according to its origin, in order to avoid a double counting of it.

If the damping matrix is completely eliminated (because all the damping is introduced otherwise), there may be numerical instabilities; it is allowed in such situation to introduce a "numerical" damping matrix proportional to the stiffness matrix, resulting the additional damping of 2% at the most for the fundamental mode and always below 5% till the cut-off frequency [42].

Appendix B.2.6 Multi-supported SSC

In the case of the SSC supported at points with uncorrelated seismic responses, the seismic load to be applied in the equivalent static analysis should be composed of an inertial component and a kinematic component (differential displacements between anchor points of the equipment).

The effect of relative support displacements is obtained by using the most conservative combinations of peak support displacements or by proper representation of the relative phasing characteristics associated with the different support displacements.

The combination of the response to the inertial load R_{iI} with that to the differential displacements R_{kI} should be performed, in a general case, employing the absolute sum method, as indicated in RG 1.92 [56]:

$$R_I = |R_{iI}| + |R_{kI}|$$

being R_I the total response in the I direction ($I = x,y,z$).

If duly justified, the SRSS can be employed, as indicated in ASCE 4 [53]:

$$R_I = \sqrt{R_{iI}^2 + R_{kI}^2}$$

In section Appendix B.3.3 there are some indications on how to determine de differential displacements.

Appendix B.2.7 Uncertainties

All computational models reproduce reality in an approximate manner due to existing uncertainties in the input parameters.

The parameters that define the model shall be evaluated in relation with their uncertainties and their impact on the results has to be quantified; if necessary, sensitivity analysis shall be conducted.

In general, in determining frequency an uncertainty of ±10% is to be assumed, unless another value can be justified in view of the uncertainties in the input data [42].

The influence of the uncertainty should be taken into account in such a way that the most unfavourable results fall within the indicated margin. This could be achieved by varying the computational model, although it is frequently conducted by modifying the excitation. Some cases follow [42]:

- In time history analyses (modal or direct integration, see section Appendix B.2.5.2) the time history steps can be elongated or compressed.
- In the response spectrum method (section Appendix B.2.5.1) the influence of natural-frequency uncertainties is usually less when dealing with smoothed design spectra (for instance the EC8 one, section Appendix B.3.2.1). If it is not possible to consider the influence of uncertainties as negligible in the results, the maximum spectral values in the range up to ±10% around the natural frequencies should be used in the analysis. A narrower range may be justified as per the existing uncertainties in the input data.

Appendix B.2.8 Supported equipment and coupling

The seismic action travels through the earth crust up to the soil where the buildings and sometimes the equipment relies. The characterization of the seismic action, by means for example of a response spectrum, is usually provided at the ground surface under the hypothesis of free field conditions. In reality, the response of the soil influences the motion of the structure and the motion of the structure influences the response of the soil, fact that is usually known as soil structure interaction (SSI).

Similarly, when the equipment is located on top of a building or structure, the structure influences the motion of the equipment; or in other words, the response spectrum to be considered as input for the study of the equipment, known as FRS, is derived from both, that on top of the soil and the structure characteristics. The needed FRS can be derived analytically, as will be explained in section Appendix B.3.4, or if the complexity of the supporting structure advices it, an ad-hoc model may be created for concluding the corresponding FRS.

Similarly, to the SSI effect, the presence of the equipment also influences to some extent, the response of the structure. In general, it will be enough considering the additional mass introduced by the equipment; however, in some particular cases the rigidity of the structure might be also affected by the presence of some equipment, in which case it can only be accounted for through a fully coupled numerical model.

The EQUIPMENT and supporting component may be considered uncoupled for dynamic behaviour if one of the following criteria, described in Guide ASN 2/01 [42], is checked:

- $R_m < 0.01$,
- $R_m < 0.1$ and $R_f > 1.25$,
- $R_m < 0.1$ and $R_f < 0.80$.

where:

R_m is the ratio between the mass of the supported equipment and that of the supporting structure;

R_f is the ratio between the fundamental frequency of the supported equipment and the fundamental frequency of the supporting structure.

The French guide AFPS [51] in its section 3.3.3, distinguishes four possible models for representing the supported equipment the supporting structure:

- Model A: the supporting structure response is governed by its own main mode and mass. The supporting displacement is then directly applied to the supported equipment.
- Model B: it is similar situation to the one described in model A, except for the mass of the supporting structure that needs to be increased with that of the supported equipment.
- Model C: the supporting structure and the supported equipment responses are governed by their respective modes and they are coupled.
- Model D: the only influence of the supporting structure is due to its stiffness (hence its mass can be neglected) and the supported equipment is governed by its own main frequency and mass.

The first three models are depicted at the top left side of Figure B.2-7, which is taken from ASCE 4 [53] and is also included in the AFPS guide [51]; model D corresponds to two springs in series (with the stiffness of the supporting structure and the supported equipment) underlying the mass of the equipment (Figure B**.2-8**).

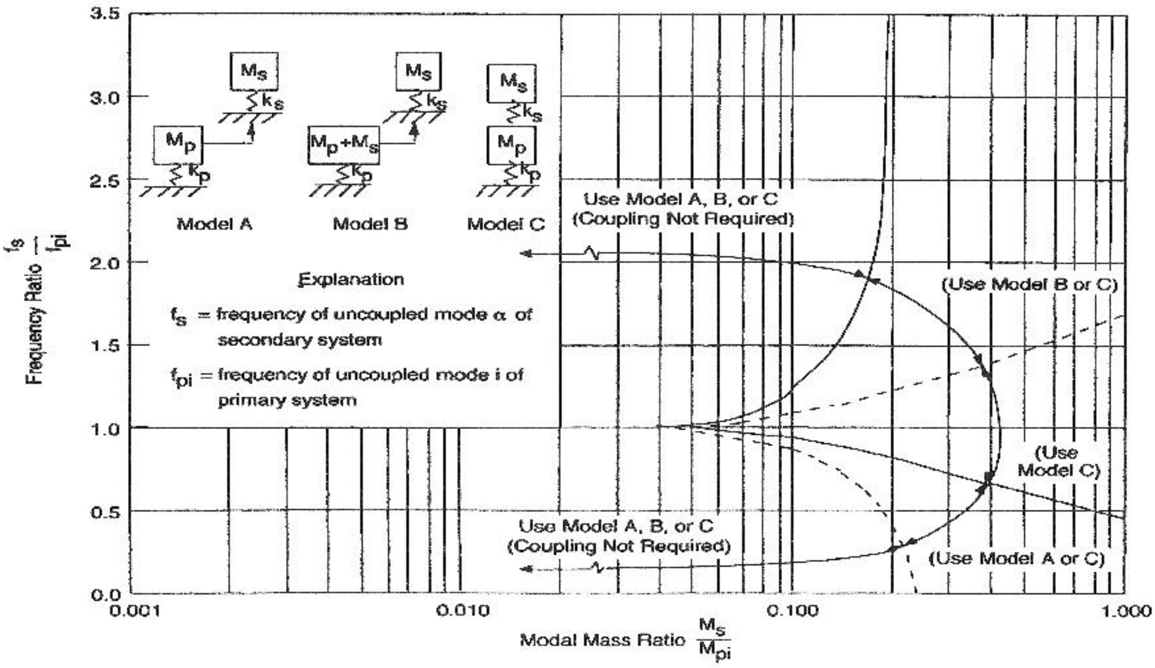


Figure B.2-7 – Modelling of supporting structure and supported equipment in different situations in relation to respective masses and frequencies.

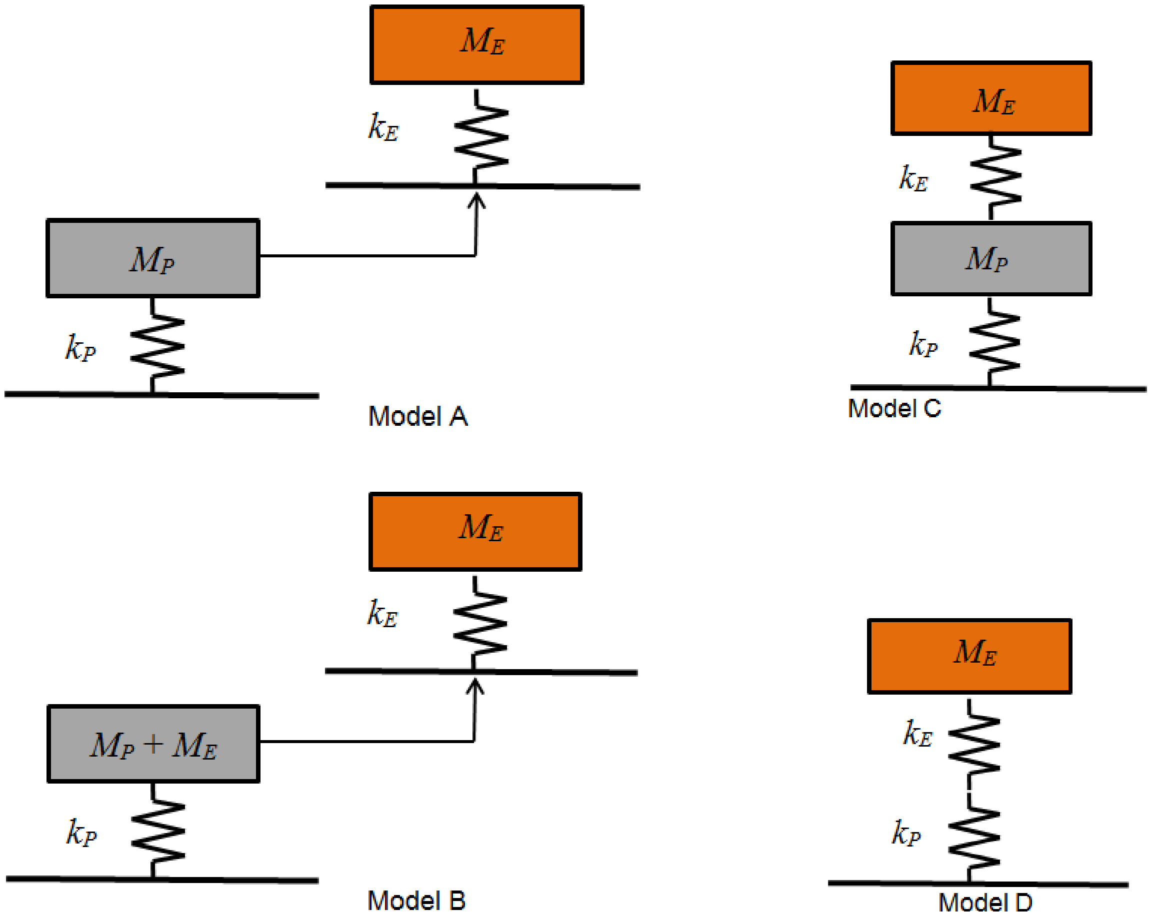


Figure B.2-8 – Different modelling of support structures and equipment.

The ordinate in Figure B.2-7 corresponds to the variable R_f explained above. The abscise corresponds to the modal mass ratio $\frac{M_E}{M_{pi}}$, where:

M_E is the mass of the supported equipment (named Ms in Figure B.2-7);

M_{pi} is obtained from the normalized modal vectors of the supporting structure as:

$$M_{pi} = \left(\frac{1}{\phi_{ci}}\right)^2$$

Where:

ϕ_{ci} is the i-mode vector value from the supporting structure's modal displacement at the location where the supported equipment is connected, taken from the i-modal vector , normalised so that the generalized mass is unity: $\{\phi_{pi}\}^T[M_p]\{\phi_{pi}\} = 1$.

When the equipment is said to be uncoupled as per the above criteria, only its weight is considered in the bearing component model. The equipment may then be studied independently by considering it is subject to accelerations from the seismic analysis of the bearing component.

When equipment is coupled the equipment model shall check the following properties:

- The modeled mass shall be equal to the equipment mass.

- The natural frequencies shall be retrieved in all directions that impact the response of the bearing component,
- If the equipment is set on several points of bearing components, the influence of equipment rigidity shall be carefully analyzed.

In relation with the latest point, it is important to distinguish the type of attachment between equipment and supporting structure:

- For equipment with a single-point attachment, the modelling principles indicated in section Appendix B.2.2.5 shall be considered. In particular, for situations that comply with Model A the supported equipment may not be included in the global model and can have its own uncoupled calculation with the motion of the attachment point; for the other situations, the inclusion of the equipment in the global model is needed according to the details given in section Appendix B.2.2.5 for Models B, C and D.
- In the case of equipment with multipoint attachment, its stiffness may restrict the movement of the supporting system and it may be necessary to include a complete coupling that accounts for the relative stiffness of the supported equipment to the supporting structure. In particular, coupling is required when values of the key design parameters from the coupled model are more than 10% higher than those from an uncoupled model. Since this condition may be difficult to demonstrate a-priori, for multi-supported equipment, as a general rule, coupling will be considered for multi-supported equipment.

Note that the conditions described above may be “modified” if justification can be provided on the behavior of the structures involved.

Appendix B.2.9 Liquids

When seismic behavior of components is influenced by the presence of fluid, the interaction with the fluid shall be taken into account. The fluid-structure interaction has a mainly inertial effect (added mass) and an effect due to free surface movement (sloshing) if it exists. These issues shall be dealt:

- either by simplified methods,
- or by more elaborated methods (discretization of the fluid in finite elements, integral equations, etc.).

In some cases where the fluid is very confined or the movement of the structure is significant compared to the size of the fluid space, damping effects may be added. In addition, in these cases, the interaction with the fluid may be non-linear. The consequences of these effects shall be evaluated. Otherwise, arguments justifying conservatism for not considering these effects shall be submitted.

The phenomena associated with fluid-structure interaction for vessels is dealt in the specific section devoted to vessels (section 10).

Appendix B.2.10 Influence of Seismic Isolation of Buildings on Component Design

Seismic isolation of buildings leads to floor spectra different from floor spectra of conventional (non-isolated) buildings. Floor spectra in isolated building floors have a narrow seismic amplification area (peak of the spectra) located at very low frequency (typically in the order of 1 Hz) and a very wide range of frequencies in which the behavior is virtually static at a new excitation level equal to the floor’s maximum acceleration (portion corresponding to ZPA in conventional building spectra).

For floor spectra from conventional buildings, the quasi-static behavior area is located at high frequency (in the order of 25 to 30 Hz) and involves “rigid” components (compared to seismic excitation) in limited numbers and often quite "strong" because they are well-anchored.

On the other hand, for floor spectra in isolated buildings, the area with near static components started at low frequency (in the order of 1 to 2 Hz), is very wide and includes several types of components.

Demand on most of the components in isolated buildings is generally less than the effort for these same components in conventional buildings.

Seismic isolation can also include dissipation energy devices which modify also the floor spectra characteristics. The various aspects of seismic isolation must be correctly represented during the analysis of a building and valid approaches for calculating floor movement shall be used.

Vertical spectra generally are not reduced.

Appendix B.2.11 Experimental seismic qualification

The experimental seismic qualification will follow the guidance established by F4E [28].

The first step in the process is to identify the seismic class of the equipment amongst the ones indicated in section 6 of the Seismic Nuclear Safety Approach document [22] in order to decide the functional requirement of the component or equipment to be tested. Seismic classes are indicated in Table 1 in Appendix 1 of the Safety Requirements document [23].

The successive steps of the seismic qualification analysis (SQA) are indicated in chapter 5 of the IEC 60980 [59] and a specific description of the qualification test sequence is provided in section 6 by F4E [28].

Regarding the seismic loading, the information contained in Appendix A [28] can be replaces by the explanations indicated in this document regarding elastic response spectrum in section Appendix B.3.2.

It should be noted that in case of having knowledge of similar equipment in service that has been exposed to natural seismic action, or that has been qualified by testing, the information can be employed as a means of justification. This option is indicated by F4E [28] in section 10.

Appendix B.3 Description of Seismic Motion

Appendix B.3.1 General Considerations

SSCs can be theoretically classified as deformation or acceleration sensitive. If the performance of an SSC is controlled by the supporting structure deformation (typically equipment anchored to different floor levels of a structure), it is deformation sensitive. When it is not vulnerable to damage from differential displacements, it is acceleration sensitive. Acceleration sensitive SSCs are vulnerable to shifting or overturning, if their anchorage or bracing is inadequate. Of course, the distinction between both types of equipment is somewhat theoretical since many of them are both deformation and acceleration sensitive, although a primary mode of behaviour can generally be identified.

The description of the seismic motion is generally defined by the acceleration response spectrum, which is particularly suited for acceleration sensitive components. However, within an elastic approach it is straightforward to obtain the corresponding displacement response spectrum.

Appendix B.3.2 Elastic Response Spectra of Ground Motion

Appendix B.3.2.1 Eurocode 8 Response Spectrum

Eurocode 8 (EC8) in its Part 1 [43] (EC8-1) provides a general definition of design spectra. France has adopted EC8-1 as part of its general rules for seismic design, supplemented with French national annexes [45].

In particular, the EC8 response spectrum that will be considered in ITER is the one that corresponds to seismic zone 4 and importance class III.

The analytical expressions that define the EC8 horizontal elastic response is:

$$0 \leq T \leq T_B: \quad S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2.5 - 1)\right]$$

$$T_B \leq T \leq T_C: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5$$

$$T_C \leq T \leq T_D: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_C}{T}$$

$$T_D \leq T \leq 4s: \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_C T_D}{T^2}$$

where:

$S_e(T)$ is the horizontal elastic response spectrum;

T is the vibration period of a linear SDOF system;

a_g is the design ground acceleration on type A ground ($a_g = \gamma_I \cdot a_{gR}$),
for ITER project, $a_{gR} = 1.6\ m/s^2$ for seismic zone 4,
$\gamma_I = 1.2$ for importance class III;

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor: $\eta = \max\left(\sqrt{10/(5+\xi)}, 0.55\right)$ where ξ is the viscous damping ratio of the structure.

According to the French administration, T_B, T_C, T_D and S are listed in the following Table B.3-1, for seismic zone 4 only where ITER locates in.

Ground Type	S	T_B	T_C	T_D
A	**1**	**0.03**	**0.2**	**2.5**
B	1.35	0.05	0.25	2.5
C	**1.5**	**0.06**	**0.4**	**2**
D	1.6	0.1	0.6	1.5
E	1.8	0.08	0.45	1.25

Table B.3-1 – Parameters for EC8 horizontal elastic response spectrum.

The analytical expressions that define the EC8 vertical elastic response is:

$$0 \leq T \leq T_B: \qquad S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3.0 - 1)\right]$$

$$T_B \leq T \leq T_C: \qquad S_{ve}(T) = a_{vg} \cdot \eta \cdot 3.0$$

$$T_C \leq T \leq T_D: \qquad S_{ve}(T) = a_{vg} \cdot \eta \cdot 3.0 \cdot \frac{T_C}{T}$$

$$T_D \leq T \leq 4s: \qquad S_{ve}(T) = a_{vg} \cdot \eta \cdot 3.0 \cdot \frac{T_C T_D}{T^2}$$

where (in addition to or different from those for the horizontal spectrum):

$S_{ve}(T)$ is the vertical elastic response spectrum;

a_{vg} is the vertical design ground acceleration, $a_{vg}/a_g = 0.9$ for ITER (seismic zone 4);

$T_B = 0.05\ s$, $T_C = 0.15\ s$ and $T_D = 1.0\ s$ for ITER (seismic zone 4).

Appendix B.3.2.2 Category IV seismic load: SL-2 Ground Response Spectrum

The SL-2 ground response spectra, defined as the envelope of the PALEO and SMS spectra, represent the action of an extremely unlikely event, load category IV, according to the terminology of the Load Specifications [17].

Table B.3-2 and Table B.3-3 give the SL-2 horizontal ground response spectra for two soil types separately: rock (soil type A) and alluvium (soil type C for ITER). For a damping ratio not listed in the tables, the following formula is recommended to perform the interpolation to calculate the corresponding spectrum. This interpolation method is applicable to any type of acceleration response spectrum (not limited to SL-2 spectrum).

$$S_{\xi_i} = S_{\xi_1} + \left(S_{\xi_2} - S_{\xi_1}\right) \cdot \frac{ln\left(\xi_i/\xi_1\right)}{ln\left(\xi_2/\xi_1\right)}$$

where:

S is the spectrum (acceleration values) that depends on the damping ratio;

ξ_1, ξ_2 are the two adjacent damping ratios listed in the tables;

ξ_i is the damping ratio for which the spectrum is wanted, $\xi_1 < \xi_i < \xi_2$.

Frequency (Hz)	Acceleration (g) for different viscous damping ratios						
	0.5%	2%	5%	7%	10%	20%	30%
0.10	0.0023	0.0023	0.0023	0.0023	0.0023	0.0023	0.0023
0.25	0.0413	0.0321	0.0260	0.0236	0.0211	0.0163	0.0141
0.40	0.0931	0.0743	0.0619	0.0570	0.0515	0.0398	0.0330
1.00	0.3343	0.2517	0.1971	0.1768	0.1558	0.1180	0.0976
1.42	0.4499	0.3455	0.2765	0.2502	0.2209	0.1632	0.1341
2.00	0.6495	0.4831	0.3731	0.3335	0.2912	0.2164	0.1791
2.82	0.8423	0.6259	0.4829	0.4346	0.3808	0.2814	0.2314
3.98	1.0380	0.7669	0.5877	0.5254	0.4590	0.3422	0.2795
5.62	1.3613	0.9856	0.7373	0.6450	0.5604	0.4083	0.3360
7.94	1.3334	0.9754	0.7388	0.6576	0.5726	0.4348	0.3658
11.22	1.0648	0.8032	0.6303	0.5720	0.5161	0.4157	0.3653
15.84	0.7917	0.6148	0.4979	0.4641	0.4309	0.3756	0.3451
22.38	0.5076	0.4380	0.3920	0.3785	0.3644	0.3386	0.3240
31.62	0.3224	0.3198	0.3181	0.3172	0.3161	0.3150	0.3150
34.00	0.3150	0.3150	0.3150	0.3150	0.3150	0.3150	0.3150
100.00	0.3150	0.3150	0.3150	0.3150	0.3150	0.3150	0.3150

Table B.3-2 – SL-2 response spectrum for soil type - rock.

Frequency (Hz)	Acceleration (g) for different viscous damping ratios						
	0.5%	2%	5%	7%	10%	20%	30%
0.10	0.0035	0.0035	0.0035	0.0035	0.0035	0.0035	0.0035
0.25	0.0494	0.0397	0.0333	0.0306	0.0278	0.0223	0.0194
0.40	0.1203	0.0961	0.0801	0.0739	0.0672	0.0534	0.0446
1.00	0.5273	0.3975	0.3117	0.2803	0.2464	0.1823	0.1499
1.42	0.7110	0.5332	0.4157	0.3722	0.3273	0.2442	0.2005
2.00	0.9691	0.7146	0.5464	0.4860	0.4246	0.3128	0.2557
2.82	1.1541	0.8594	0.6646	0.5944	0.5192	0.3846	0.3137
3.98	1.1989	0.8992	0.7011	0.6302	0.5565	0.4199	0.3500
5.62	1.3765	1.0205	0.7852	0.6983	0.6109	0.4601	0.3861
7.94	1.1986	0.9002	0.7030	0.6357	0.5702	0.4536	0.3953
11.22	0.8672	0.6846	0.5639	0.5247	0.4857	0.4173	0.3815
15.84	0.6672	0.5434	0.4616	0.4399	0.4190	0.3820	0.3617
22.38	0.4645	0.4214	0.3929	0.3839	0.3746	0.3566	0.3466
31.62	0.3482	0.3450	0.3429	0.3423	0.3423	0.3423	0.3423
34.00	0.3423	0.3423	0.3423	0.3423	0.3423	0.3423	0.3423
100.00	0.3423	0.3423	0.3423	0.3423	0.3423	0.3423	0.3423

Table B.3-3 – SL-2 response spectrum for soil type - alluvium.

The spectrum given for 30% damping ratio is the minimum one, hence it is the one to be used for the SL-2 spectra when the damping is equal to or greater than 30%.

The vertical response spectrum is obtained by multiplying by 2/3 the horizontal one according to the Load Specifications [17].

Appendix B.3.2.3 Category III seismic load: SMHV Ground Response Spectrum

The SMHV ground response spectra represent the action of a very unlikely event, load category III, according to the terminology in the Load Specifications [17], in which the spectrum is defined in a tabular way.

Table B.3-4 and Table B.3-5 give the SMHV horizontal ground response spectra for two soil types separately: rock (soil type A) and alluvium (soil type C for ITER). The spectrum given for 20% damping ratio is the minimum one, hence it is the one to be used for the SMHV spectra when the damping is equal to or greater than 20%.

Similarly, the SMHV vertical ground response spectrum is obtained by multiplying by 2/3 the horizontal one according to the Load Specifications [17].

Frequency (Hz)	**Acceleration (g) for different viscous damping ratios**					
	0.5%	**2%**	**5%**	**7%**	**10%**	**20%**
0.10	0.0006	0.0006	0.0006	0.0006	0.0006	0.0006
0.25	0.0117	0.0094	0.0079	0.0072	0.0064	0.0050
0.40	0.0271	0.0217	0.0181	0.0168	0.0152	0.0120
1.00	0.1053	0.0812	0.0653	0.0590	0.0530	0.0420
1.42	0.1772	0.1388	0.1134	0.1035	0.0923	0.0702
2.00	0.2975	0.2278	0.1817	0.1647	0.1460	0.1115
2.82	0.4903	0.3703	0.2910	0.2603	0.2287	0.1695
3.98	0.6927	0.5158	0.3989	0.3573	0.3117	0.2302
5.62	0.9479	0.6871	0.5147	0.4525	0.3926	0.2852
7.94	0.9455	0.6986	0.5354	0.4753	0.4130	0.3103
11.22	0.7898	0.5899	0.4578	0.4144	0.3722	0.2969
15.84	0.5503	0.4318	0.3535	0.3294	0.3059	0.2658
22.38	0.3657	0.3123	0.2770	0.2670	0.2568	0.2383
31.62	0.2241	0.2232	0.2226	0.2221	0.2215	0.2194
34.00	0.2194	0.2194	0.2194	0.2194	0.2194	0.2194
100.00	0.2194	0.2194	0.2194	0.2194	0.2194	0.2194

Table B.3-4 – SMHV response spectrum for soil type - rock.

Frequency (Hz)	**Acceleration (g) for different viscous damping ratios**					
	0.5%	**2%**	**5%**	**7%**	**10%**	**20%**
0.10	0.0008	0.0008	0.0008	0.0008	0.0008	0.0008
0.25	0.0100	0.0087	0.0082	0.0075	0.0062	0.0054
0.40	0.0244	0.0209	0.0195	0.0179	0.0146	0.0128
1.00	0.1244	0.1011	0.0924	0.0830	0.0650	0.0546
1.42	0.2144	0.1724	0.1556	0.1377	0.1051	0.0877
2.00	0.3369	0.2716	0.2457	0.2172	0.1616	0.1321
2.82	0.5227	0.3956	0.3526	0.3083	0.2310	0.1871
3.98	0.6059	0.4737	0.4286	0.3775	0.2823	0.2337
5.62	0.7114	0.5524	0.4900	0.4253	0.3220	0.2673
7.94	0.6447	0.5059	0.4594	0.4148	0.3231	0.2789
11.22	0.4960	0.4102	0.3801	0.3496	0.2983	0.2705
15.84	0.3850	0.3283	0.3127	0.2975	0.2703	0.2551
22.38	0.3004	0.2775	0.2708	0.2640	0.2510	0.2435
31.62	0.2408	0.2399	0.2395	0.2389	0.2384	0.2384
34.00	0.2384	0.2384	0.2384	0.2384	0.2384	0.2384
100.00	0.2384	0.2384	0.2384	0.2384	0.2384	0.2384

Table B.3-5 – SMHV response spectrum for soil type - alluvium.

Appendix B.3.2.4 Category II seismic load: SL-1 Ground Response Spectrum

The SL-1 ground response spectra represent the horizontal ground motion of the action of a likely event, load category II, according to the terminology in the Load Specifications [17].

The SL-1 response spectra are obtained by dividing by 4 the SL-2 ones. Table B.3-6 and Table B.3-7 give the SL-1 spectra for two soil types separately: rock (soil type A) and alluvium (soil type C for ITER). The spectrum given for 30% damping ratio is the minimum one, hence it is the one to be used for the SL-2 spectra when the damping is equal to or greater than 30%.

Similarly, the vertical response spectrum is obtained by multiplying by 2/3 the horizontal one according to the Load Specifications [17].

Frequency (Hz)	Acceleration (g) for different viscous damping ratios						
	0.5%	2%	5%	7%	10%	20%	30%
0.10	0.0006	0.0006	0.0006	0.0006	0.0006	0.0006	0.0006
0.25	0.0103	0.0080	0.0065	0.0059	0.0053	0.0041	0.0035
0.40	0.0233	0.0186	0.0155	0.0143	0.0129	0.0100	0.0083
1.00	0.0836	0.0629	0.0493	0.0442	0.0390	0.0295	0.0244
1.42	0.1125	0.0864	0.0691	0.0626	0.0552	0.0408	0.0335
2.00	0.1624	0.1208	0.0933	0.0834	0.0728	0.0541	0.0448
2.82	0.2106	0.1565	0.1207	0.1087	0.0952	0.0704	0.0579
3.98	0.2595	0.1917	0.1469	0.1314	0.1148	0.0856	0.0699
5.62	0.3403	0.2464	0.1843	0.1613	0.1401	0.1021	0.0840
7.94	0.3333	0.2439	0.1847	0.1644	0.1432	0.1087	0.0915
11.22	0.2662	0.2008	0.1576	0.1430	0.1290	0.1039	0.0913
15.84	0.1979	0.1537	0.1245	0.1160	0.1077	0.0939	0.0863
22.38	0.1269	0.1095	0.0980	0.0946	0.0911	0.0847	0.0810
31.62	0.0806	0.0800	0.0795	0.0793	0.0790	0.0788	0.0788
34.00	0.0788	0.0788	0.0788	0.0788	0.0788	0.0788	0.0788
100.00	0.0788	0.0788	0.0788	0.0788	0.0788	0.0788	0.0788

Table B.3-6 – SL-1 response spectrum for soil type - rock.

Frequency (Hz)	Acceleration (g) for different viscous damping ratios						
	0.5%	2%	5%	7%	10%	20%	30%
0.10	0.0009	0.0009	0.0009	0.0009	0.0009	0.0009	0.0009
0.25	0.0123	0.0099	0.0083	0.0077	0.0070	0.0056	0.0049
0.40	0.0301	0.0240	0.0200	0.0185	0.0168	0.0134	0.0112
1.00	0.1318	0.0994	0.0779	0.0701	0.0616	0.0456	0.0375
1.42	0.1777	0.1333	0.1039	0.0931	0.0818	0.0611	0.0501
2.00	0.2423	0.1787	0.1366	0.1215	0.1062	0.0782	0.0639
2.82	0.2885	0.2149	0.1662	0.1486	0.1298	0.0962	0.0784
3.98	0.2997	0.2248	0.1753	0.1576	0.1391	0.1050	0.0875
5.62	0.3441	0.2551	0.1963	0.1746	0.1527	0.1150	0.0965
7.94	0.2996	0.2251	0.1758	0.1589	0.1426	0.1134	0.0988
11.22	0.2168	0.1712	0.1410	0.1312	0.1214	0.1043	0.0954
15.84	0.1668	0.1359	0.1154	0.1100	0.1048	0.0955	0.0904
22.38	0.1161	0.1054	0.0982	0.0960	0.0937	0.0892	0.0867
31.62	0.0870	0.0863	0.0857	0.0856	0.0856	0.0856	0.0856
34.00	0.0856	0.0856	0.0856	0.0856	0.0856	0.0856	0.0856
100.00	0.0856	0.0856	0.0856	0.0856	0.0856	0.0856	0.0856

Table B.3-7 – SL-1 response spectrum for soil type - alluvium.

Appendix B.3.2.5 Beyond design seismic load: SL-3 Ground Response Spectrum

As a beyond design seismic load for Hard Core Components (HCCs) only, the SL-3 ground response spectrum represents the horizontal ground motion of an extremely unlikely event with a return period higher than 20000 years. It is defined as the envelope of the following two spectra:

- The SMS spectrum increased by a factor of 1.5;
- The PALEO spectrum.

Table B.3-8 gives the spectrum for rock (soil type A, the only one of interest). The spectrum given for 30% damping ratio is the minimum one, hence it is the one to be used for the SL-2 spectra when the damping is equal to or greater than 30%.

Similarly, the vertical response spectrum is obtained by multiplying by 2/3 the horizontal one according to the Load Specifications [17].

Frequency (Hz)	Acceleration (g) for different viscous damping ratios						
	0.5%	2%	5%	7%	10%	20%	30%
0.10	0.0023	0.0023	0.0023	0.0023	0.0023	0.0023	0.0023
0.25	0.0413	0.0321	0.0260	0.0236	0.0211	0.0163	0.0141
0.40	0.0931	0.0743	0.0619	0.0570	0.0515	0.0398	0.0330
1.00	0.3343	0.2517	0.1971	0.1768	0.1558	0.1180	0.0976
1.42	0.4623	0.3609	0.2939	0.2679	0.2382	0.1794	0.1493
2.00	0.7475	0.5673	0.4482	0.4047	0.3572	0.2706	0.2235
2.82	1.1419	0.8615	0.6761	0.6059	0.5316	0.3944	0.3236
3.98	1.5570	1.1504	0.8816	0.7881	0.6885	0.5133	0.4193
5.62	2.0419	1.4784	1.1060	0.9675	0.8406	0.6125	0.5040
7.94	2.0000	1.4631	1.1082	0.9864	0.8589	0.6522	0.5487
11.22	1.5972	1.2048	0.9455	0.8580	0.7742	0.6236	0.5480
15.84	1.1875	0.9222	0.7469	0.6962	0.6464	0.5634	0.5177
22.38	0.7614	0.6570	0.5880	0.5678	0.5466	0.5079	0.4860
31.62	0.4836	0.4797	0.4772	0.4758	0.4742	0.4725	0.4725
34.00	0.4725	0.4725	0.4725	0.4725	0.4725	0.4725	0.4725
100.00	0.4725	0.4725	0.4725	0.4725	0.4725	0.4725	0.4725

Table B.3-8 – SL-3 response spectrum for soil type - rock.

Appendix B.3.3 Relative displacement

In the case of multi-supported SSC supported by structural elements whose seismic responses are not correlated, it is advisable to make sure that the seismic load is composed of an inertial component and a kinematical component [42].

The inertial component of the seismic load can be defined with Floor Response Spectra (FRS). The enveloped FRS over all supports are usually used as the uniform input at all supports.

The kinematical component of the seismic load is the field of differential displacements between the supports at the anchorage points of the SSC, which can be represented by the relative displacement between the supports, applied in the most unfavourable way(s).

The kinematical component can be estimated by conservatively combining the absolute values of the structural displacements, with the structures being considered separately [42]. To avoid excessive conservatism, some efforts were made in the following documents:

- The seismic relative displacements between the floors of TKC are provided in [31].

- The seismic relative displacements between the Tokamak and the building can be found in [32], additionally in [33] an example on how to extract values of displacement is presented.
- For ancillary buildings, there is a report that contains the relative displacements to be considered in the seismic design of SSCs supported by multiple floors [35].

Appendix B.3.4 Floor Response Spectra (FRS)

Appendix B.3.4.1 Overview

The seismic action defined in Appendix B.3.2 corresponds to the ground surface, for the types of soil specified, namely Type A (rock) or Type C (alluvium), and in free field conditions. These spectra constitute the action that is directly applied for the seismic design of buildings directly on the ground surface.

However, when the SSC inside a building has to be designed, the seismic action defined on top of the soil “travels” through the building before affecting the equipment; in this process its frequency content varies depending on the dynamic characteristics of the building. The seismic action to be considered in the design of the SSC has to consider both, the ground spectrum and the building properties.

Appendix B.3.4.2 FRS for Nuclear Buildings

FRS for the Tokamak Complex (TKC) are provided by ITER [29]. The FRSs are provided for a very high number of points (more than 1000 points) and it is important to check that the point(s) identified as input data for the analysis of the SSC are consistent with the location of the SSC. It is advised to provide maps or drawings as evidence of the correct interpretation of the FRS document.

FRS for Hot Cell Complex (HCC) are provide in [30]. It is currently at preliminary design stage and will be updated in the future. Since there is no additional broadening applied on the spectra in this document, one has to consider the uncertainty in the natural frequency of the SSC if the spectra are directly used as the input for the seismic analysis of the SSC.

Appendix B.3.4.3 FRS for Ancillary Buildings

A methodology for generating floor response spectrum (FRS) for ancillary buildings has been proposed in [35]. Application of this methodology is already presented in documents attached to [35] for some of the ancillary buildings, which can also be found in the IDM folder [36]. An example is given in [37].

Appendix B.3.4.4 FRS in Tokamak

As [29] provides FRS for the Tokamak Complex excluding the Tokamak machine, [33] provides FRS in the Tokamak. An uncertainty factor of 1.25 is required to be applied to all FRS provided in [33].

Appendix B.3.4.5 Propagation of FRS through Major SSC to Sub SSC

In the case of a sufficiently homogeneous major SSC without any significantly oscillating partial system, the response spectra at the place of installation of a sub SSC may be determined by the substitution method presented below.

The substitution method is a good approximation if the response of the major SSC is dominated by a single natural mode. If more natural modes are significant contributors then this procedure is increasingly conservative.

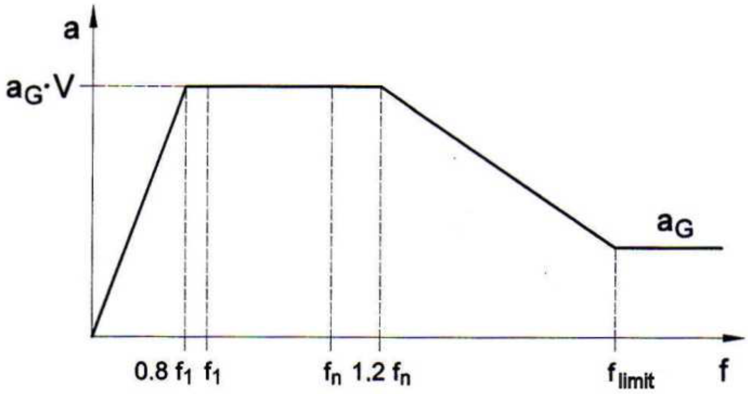


Figure B.3-1 – Determination of the shape of the response spectrum.

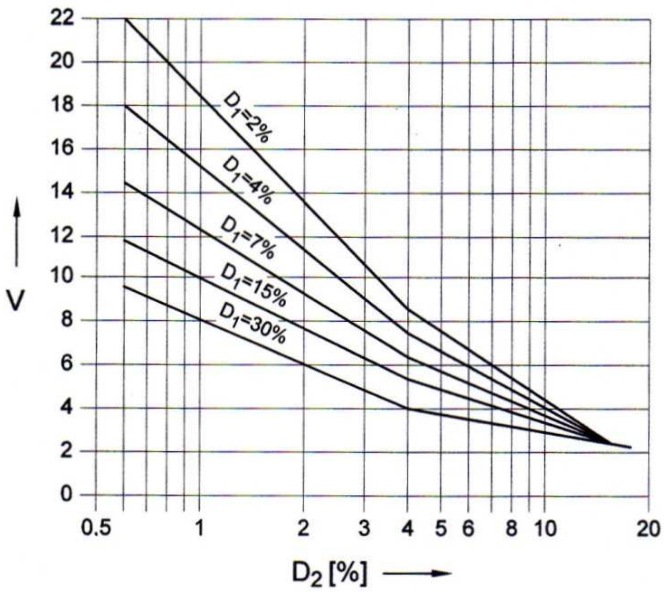


Figure B.3-2 – Determination of the spectrum amplification factor.

The shape of the response spectrum shall be determined as shown in Figure B.3-1. The spectrum amplification factor with respect to the acceleration of the major SSC at the place of installation of the sub SSC (here: the acceleration of the component) shall be determined as shown in Figure B.3-2.

where:

f is the frequency, in logarithmic axis;

f_1 is the lowest relevant frequency of the major SSC at the lower limit value in the variation range of the SSC parameters, however, not higher than the rightmost corner frequency of the highest plateau of the associated input response spectrum;

f_n is the highest relevant frequency of the major SSC for the upper limit value in the variation range of the component parameters, however, not higher than the rightmost corner frequency of the highest plateau of the associated response spectrum;

f_{limit} is the upper limit frequency of the response spectrum of the major SSC;

a is the FRS at the sub SSC support;

a_G is the maximum acceleration of the major SSC at the sub SSC support, which is the ZPA of a;

V is the spectra amplification factor as shown in Figure B.3-2;

D_1 is the damping ratio of the major SSC;

D_2 is the damping ratio of the sub SSC.

The acceleration of the major SSC shall be determined as specified in Appendix B.2.5. The obtaining of the ZPA can be achieved by conducting a response spectrum analysis: the acceleration resulting at each point in the response spectrum analysis (Appendix B.2.5.1) will be the ZPA of the FRS at that same point.

Appendix B.4 Design and Verification of Anchors

The applicable codes and standards in the ITER context are the following ones:

- ITER Structural Design Code for Buildings [25]
- Eurocode 2 part 4 [47], devoted to anchors, which will become applicable in France in the near future.

For embedded plates, the applicable internal ITER documents will be considered [38]

Many of the descriptions and procedures described below come from the AFPS guidelines [51].

Appendix B.4.1 Scope

This chapter provides guidelines on the design and verification of post-installed and cast-in place anchors. Being a seismic design, the system will be required to have adequate ductility, which in general terms means that the anchorage strength should be governed by ductile yielding of a steel element.

Appendix B.4.2 Definitions

Appendix B.4.2.1 General Definitions

The term anchorage refers to the mechanical elements that guarantee a role in the interface between the equipment and its support.

The anchorage generally consists of a fixture and anchors embedded in the base material, usually the supporting concrete. It is considered that the anchorage also includes that part of the concrete that guarantees the fixing as well as the correct transmission of the anchor forces. The fixture distributes loads to the anchors.

The anchorage role is to guarantee the transmission of forces between the equipment and the supporting structure and to assure the compatibility of deformations across the interface. At the same time, acceptable limits must be maintained to ensure the correct operation of the equipment.

Appendix B.4.2.2 Types of anchors

There are several types of anchors according to the way they are set up or the way they work.

- Passive anchorages: post-installed or cast-in place
- Active anchorages: mainly pre-stressed
- Through-bolts, with or without sleeve

The two types indicated for the passive anchorages are treated in this guide. These two types are presented in Figure B.4-1. The length referred to as h_{ef} in Figure B.4-1 is called l_b in the remaining of the chapter.

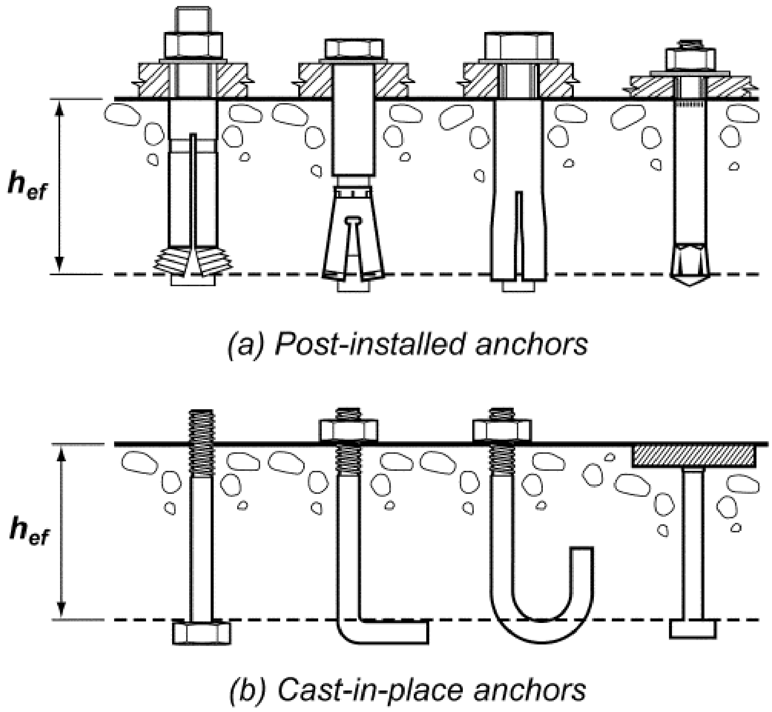


Figure B.4-1 – Types of anchors and effective length.

Appendix B.4.2.2.1 Post- installed anchors

Post- installed anchors are achieved by placing steel bolts in predrilled holes in the hard concrete, anchored by expansion, shape locking, or grouting. For structural applications, these bolts can be classified into four categories:

- Torque-controlled expansion anchor (type A)
- Deformation-controlled expansion anchor (type B)
- Undercut anchors (type C)
- Bonded anchors (type D)

- For type A bolts, the expansion of the cone takes place by applying a torque to a nut or bolt, the degree of anchorage being controlled by the value of the applied torque, in particular it derives its tensile resistance from the expansion of one or more sleeves or other components against the sides of the drilled hole through the application of torque, which pulls the cone(s) into the expansion sleeve(s) during installation. It is useful to distinguish:
    - o Single cone sleeve type (type A1, Figure B.4-2) for which the diameter of the hole exceeds that of the bolt, existing a spacer sleeve.
    - o Bolt expansion anchor or wedge type (type A2, Figure 9 2) for which the diameter of the orifice is identical to that of the threaded part.

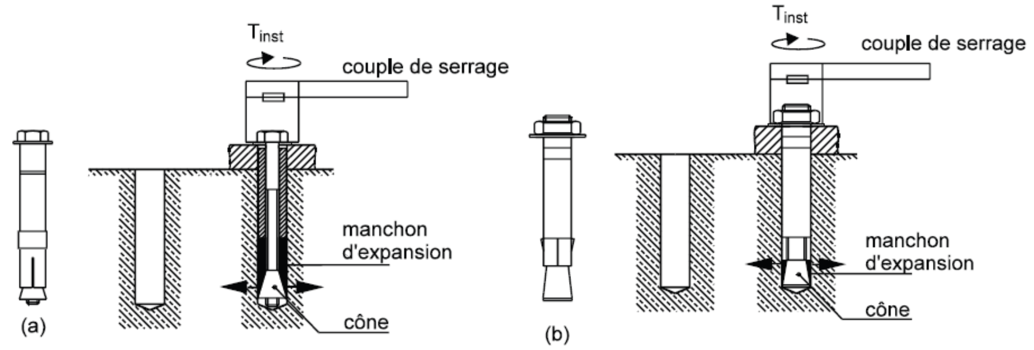


Figure B.4-2 – Bolts of type A1 (left) and A2 (right).

- For type B bolts, deformation-controlled expansion anchor, the expansion is generally obtained by friction of a sleeve or cone Figure B.4-3). It derives its tensile resistance by expansion against the side of the drilled hole through movement of an internal plug in the sleeve or through movement of the sleeve over an expansion element (plug). Once set, no further expansion can occur.

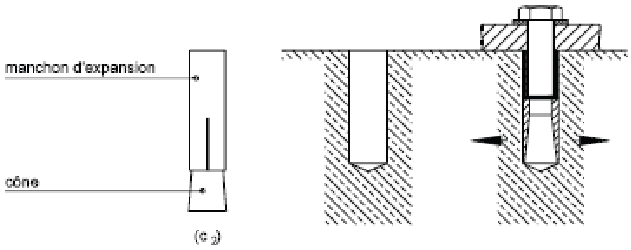


Figure B.4-3 – Bolt of type B (expansion by striking).

- For type C bolts, the strength is assured by the mechanical interlock provided by undercutting the concrete at the end of the anchor (Figure B.4-4). The undercutting is achieved with a special drill before installing the fastener or alternatively by the fastener itself during its installation, as depicted in Figure B.4-4.

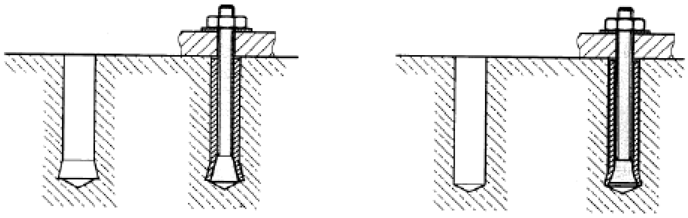


Figure B.4-4 – Bolt of type C: undercut anchors.

- For the type D bolts, bonded anchors, the anchorage is provided because the metallic elements are bonded to the surface of the orifice by means of an especial mortar and the forces are transmitted to the concrete by adhesion across the mortar-concrete and the mortar-metal interfaces. In the particular case of bonded expansion anchors, it is designed such that the anchor bolt can move relative to the hardened bonding compound resulting in follow-up expansion.

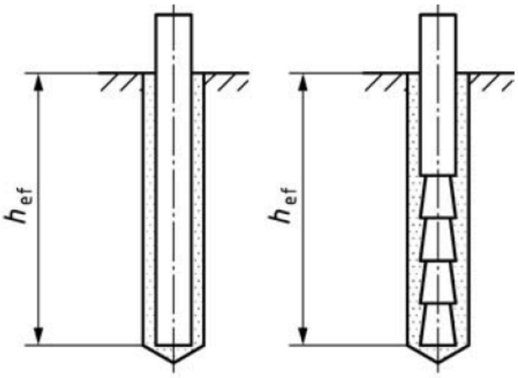


Figure B.4-5 – Bolts of type D: bonded anchors (left), bonded expansion anchors (right).

For new installations torque-controlled expansion anchor (type A) bolts should be preferred.

Generally speaking, the verifications to be applied to all categories consist in defining first the nominal strength capacity of the anchor and then introducing the effects of location (group effects, edge effects, etc.), concrete quality, and possible cracking of the concrete in order to establish the verification criteria for tension, shear, and combined tension and shear. All of those verifications are described in sections Appendix B.4.5 and Appendix B.4.6.

Appendix B.4.2.2.2 Cast-in place anchors

Cast-in place anchors are those installed before placing the concrete.

For cast-in-place anchors, various configurations can be distinguished depending on the characteristics of the bolt:

- Anchors with a straight bolt,
- Anchors with a headed bolt,
- Anchors with a hooked bolt.

- The straight bolt anchors are only adequate if anchor depth (l_b) is sufficient to mobilise an adhesive force capable of withstanding the tensile demands.

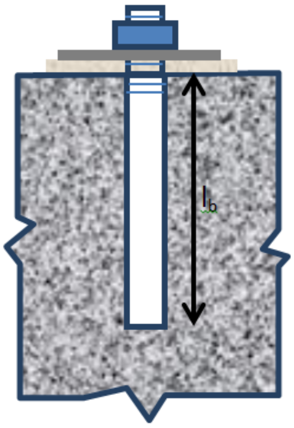


Figure B.4-6 – Cast-in-place anchorage with straight stem.

- Anchors with headed bolts. The bolt is provided at the head with a washer or plate, either welded or fixed with a nut, that provides the anchorage function. The plate must be sufficiently stiff to distribute uniformly the force exerted on the concrete and its connection with the steel axis must have adequate resistance to ensure the transmission of the tensile forces between the stem and the plate. The stiffness requirements on the plate generally ensure that the strength is sufficient. This configuration is particularly adequate when the anchorage depth is limited. Construction criteria must also be considered for the supporting concrete, taking into account the nature of the demands transmitted to the concrete.

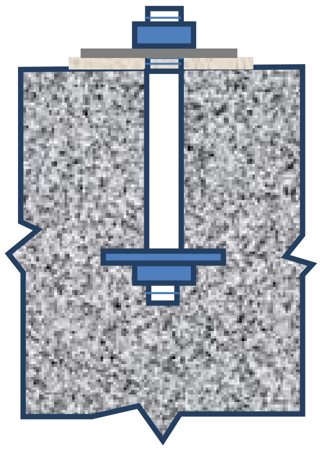


Figure B.4-7 – Cast-in-place anchorage with straight stem and embedded plate.

- Anchorages with hooked bolts. They can have either a 90 degree bend (L-bolt) or a 180-degree-bend (J-bolt). They clearly improve the performance of straight bolts by the contribution provided by the curved part of the stem. The hook is generally arranged to gather the reinforcing bars of a layer placed at the anchorage depth.

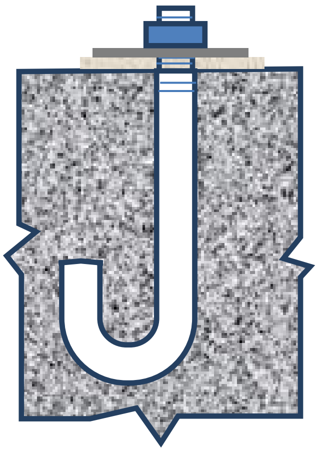


Figure B.4-8 – Cast-in-place anchorage with a curved stem.

Appendix B.4.3 Determination of demand

Appendix B.4.3.1 General considerations

The demands transmitted across the interfaces are determined either with models of the equipment to which the accelerations are applied by their support structures (main and intermediate) or through floor response spectra. It is understood that the FRSs are calculated following the considerations in section Appendix B.2.5.2 related to supported equipment and coupling.

Depending on the case, the demands taken into account for anchorage verification may be:

- "global" demands (normal forces, shear forces, and bending moments) produced by the calculations at the interface (e.g.: a steel structural member), as schematically shown in Figure B.4-9,
- "local" demands, consisting of tension and shear directly applied to the anchor bolt, and the local pressures applied at the interface.

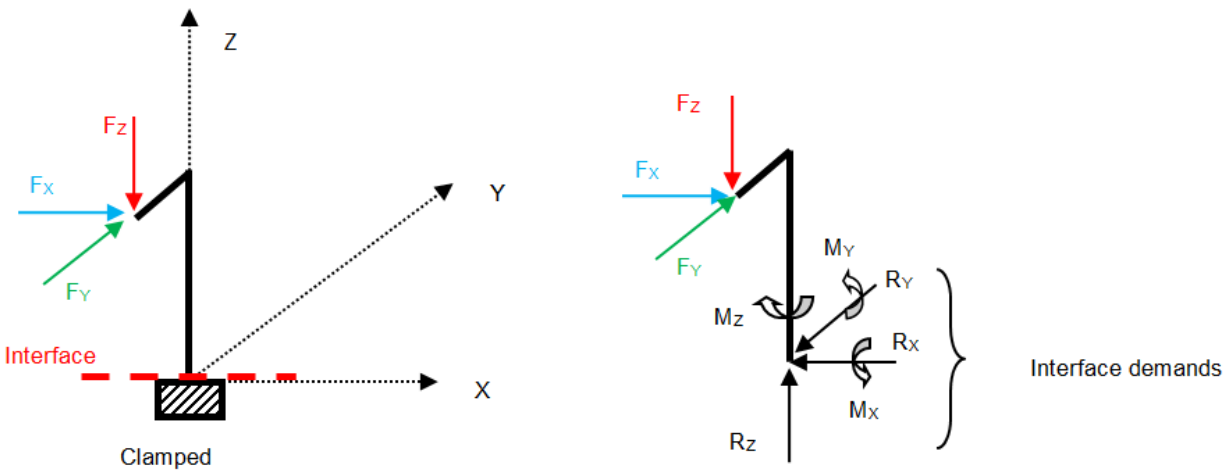


Figure B.4-9 – Example of interface demands.

The tension and shear demands on each anchor and the compression transmitted locally from the overall actions must be consistent with the mechanical behaviour of the interface with the support structure.

Particular attention must be paid to the modelling of the interfaces, taking into account construction details:

- The potential pre-stressing of bolts may affect the modelling conditions;
- Assemblies with anisotropic clearance (oblong orifices), often required to avoid parasitic stresses under thermal deformations, may constitute fixed supports in one direction and sliding supports in another. The clearances, compared with the seismic displacements, may lead to different modelling assumptions depending on the size of the earthquake (Figure B.4-10);
- The distribution of forces among the anchors may depend on the stiffness of the plate and the location of the bolts (as can be seen in Figure B.4-11, Figure B.4-12 and Figure B.4-13 in the following sections).
- The assumption of a fixed connection implies the absence of rotations at the anchorage. This entails some minimum rotational stiffness requirements, which may be evaluated for certain anchorage configurations following Eurocode 3 Part 1-8 [45].

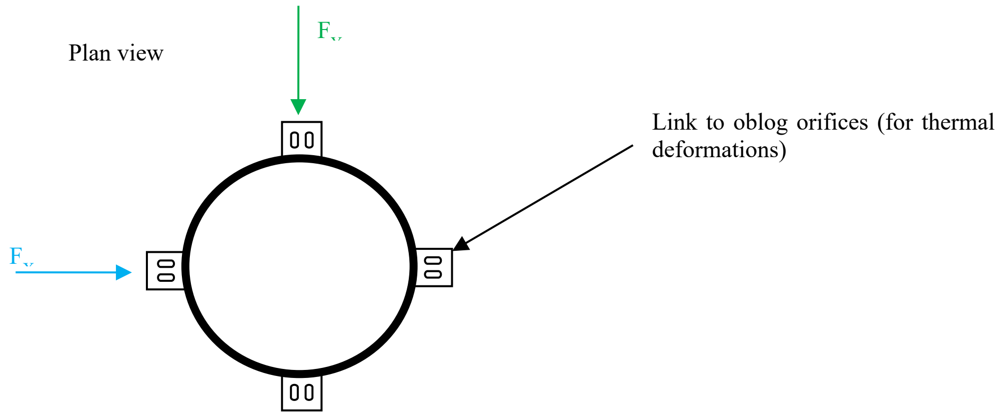


Figure B.4-10 – Example of interface demands.

The load transfer mechanisms across the interface are very diverse and require a specific consideration, which normally require specific analyses by someone with a good knowledge of the equipment, the anchorages, and the support structure. In the following sections, specific guidance is provided for the obtaining of the tension and shear forces on the individual bolts. The cases included are those considered as more representative of the situations to be found in the ITER context.

In general, the action effects on the individual anchors may be defined using an elastic analysis, which is conservative for ductile failure modes, and the anchors are expected to have a ductile behaviour (see section Appendix B.4.3.5).

Appendix B.4.3.2 Obtaining of tension loads on anchors

For the calculation of the tension loads on the anchors, the following assumptions will be made:

- The fixture is considered rigid.
- All anchors have the same characteristics.
- Compressions are transmitted to the concrete by the fixture (and not by the anchors).
- A linear distribution, similar to the one assumed for reinforced concrete sections, is assumed between the tension of anchors and the compression of the concrete.

For threaded anchors the stressed crossed section should be provided by the manufacturer, in its absence, the indications in ISO 898-1 [60] can be considered.

An example of how the elastic distribution of normal forces on anchors can be calculated is presented in Figure B.4-11. As indicated above, it is assumed that the compression is transmitted to the concrete by the fixture. A linear distribution considering the concrete Young modulus can be calculated, or alternatively a simplified approach in which the compression is concentrated at the attachment toe can be considered.

In case the fixture exhibits large deformation (Figure B.4-12), a particular analysis, first of the fixture itself, and then of the distribution among the anchors should be conducted.

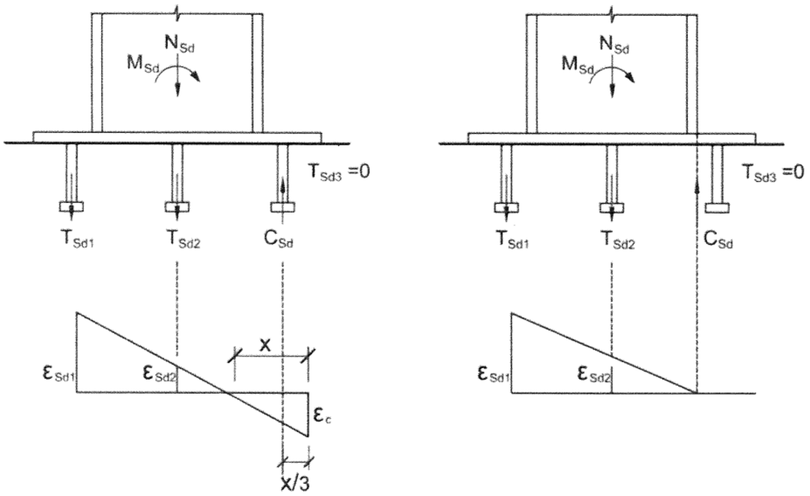


Figure B.4-11 – Example of elastic load distribution for an anchorage with rigid fixture loaded with moment and normal force (from [72]). Theory of elasticity (left) and assuming all compression at toe (right)

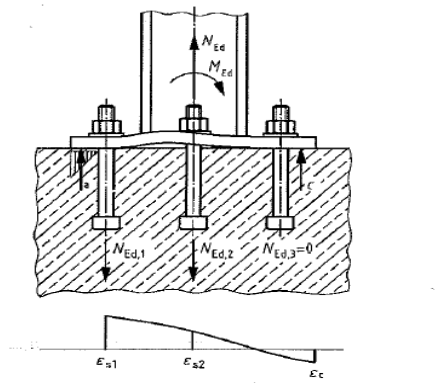


Figure B.4-12 – Distribution of forces in anchorage with deformable plate subjected to a normal force and a bending moment.

Appendix B.4.3.3 Obtaining of shear loads on anchors

For determining the shear forces on individual anchors from a group it is assumed that all anchors participate in the shear capacity with the same stiffness and that the fixture is rigid.

If the shear is not applied on the center of gravity of the group of anchors, an adequate torsional moment should be accounted for.

In case an edge is close to the anchorage, and the corresponding edge effects are to be included in the calculation, the total shear should be decomposed in its components parallel and perpendicular to the edge and acting on the center of gravity. In case the original shear force has an eccentricity, an additional torsional moment is to be considered.

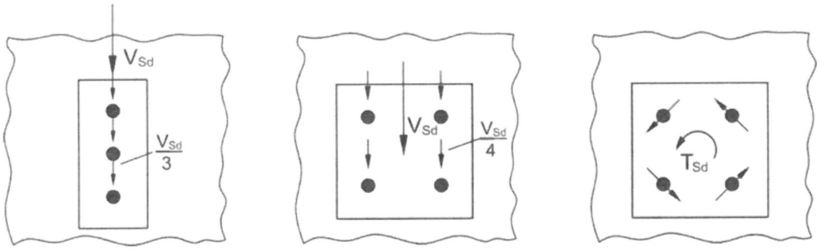


Figure B.4-13 – Distribution of shear forces for different anchorage configurations.

Appendix B.4.3.4 Concomitances

In many cases the anchor design requires the prior calculation of the global normal force (N), shear force(s) (V) and moment(s) (M). In some other cases (i.e. finite element model) the modelling permits to obtain the individual forces on each anchor.

In case global demands are available they have to be applied on to the anchor system in order to obtain the individual forces on anchors and on the concrete and go ahead with the corresponding verifications, as explained in the subsequent sections.

Seismic action is represented independently in each direction (two horizontal and one vertical). The resulting seismic forces in the anchors for each of these directions have to be combined appropriately. Besides the seismic action has to be combined with other actions that are expected to occur at the same time as the earthquake. So hereafter the term concomitance refers to:

- Concomitance of directions and of types of efforts
- Concomitance of different types of loads

In the case of a simplified analysis (i.e. static equivalent) or a modal spectral analysis, only the maximum value for each component is obtained. In case they correspond to different seismic directions they are not concomitant; in this case the Newmark rule as explained in section Appendix B.2.5.1.5 is also applicable here.

It is desirable to keep track of the concomitance of the three types of forces that can act in the anchor system, instead of combining the maximum values for each N, V and M, which can end up with a rather conservative design. So, for the seismic action instead of having a single group of (N, V, M) there will be three groups that need to be verified independently.

The load combinations to be considered are those already indicated in section Appendix B.1.4.

Appendix B.4.3.5 Non-linearity, ductility and reduction factors

The forces transmitted through the interface between the anchors and the support structure can depend on various nonlinearities developed on the equipment or its support. The non-linear behaviour can be associated to plastification in the equipment or in the steel supporting structure, to stretch of the anchor steel, to the flexibility of the fixture or to the uplift of the equipment (with or without anchor plastification):

- When the nonlinearities appear in the main (or secondary) supporting structure, their effect can be considered by means of equivalent elastic calculation either reducing the ground response spectrum or the floor response spectrum.
- When the nonlinearities appear in the equipment itself, the effects can be taken into account with an equivalent linear analysis considering an appropriate behaviour factor that permits to reproduce the limited inertial forces or acceleration that can affect the equipment. This coefficient depends on the frequency of the equipment compared to the main one of the main supporting structure. Some guidelines for its definition are presented here below.
- When the nonlinearities appear on the anchorage system, their effect lead to increase the behaviour factor of the equipment. These nonlinearities are also considered in the force distribution on the anchorage section, being the global demands affected by a behaviour

factor that might need to be increased for considering both nonlinearities, those of the equipment and of the anchor system.

These principles are not applicable if the modelling allows identifying directly in the calculation the nonlinearities that appear in the anchors during an earthquake. The same applies to other nonlinearities in the equipment or the supporting structure that may be also identified in the modelling.

The reduction of accelerations or spectra takes also account for the variations of the viscous damping that takes place when nonlinearities appear. In case a coupled modelling of equipment and support is employed, a global behaviour factor can be employed. This behaviour factor should be computed considering those of both, the equipment and the support structure.

Behaviour factors are widely employed in civil engineering for structures, however it is less common for equipment, the calculations of which generally rely on primary and secondary demands. Only the latter ones are translated under the form of imposed displacements rather than forces, which permits to represent the seismic action with a reduction of the elastic demands. This reduction can be treated following the same procedure as when behaviour factors are employed in buildings.

The wide variety of potential equipment and the fact that their fundamental frequencies may be considerable higher than that of the supporting structure leads to limit the behaviour factor when the equipment is too rigid with respect to the support and, as a consequence, the seismic action is a primary one for which it is not allowed to reduce its demands on the equipment. The behaviour factor of the equipment depends on the ratio of its fundamental frequency to that of the supporting structure. This dependency can be summarised by employing reduced FRS for the determination of demands on anchors, according to the following principles:

- If the equipment is rigid with respect to the supporting structure ($f_E > 2f_P$), the vibratory movement is an imposed forced, hence no reduction is allowed on the elastic demands ($\rho_E = 1$)
- If the equipment and the structure and the equipment have similar main frequencies, the vibratory movement is equivalent to an injected energy in the system and the reduction factor can be approximated by:

$$\rho_E = \sqrt{(2q_E - 1)}$$

  where

  q_E is the behaviour factor of the equipment.

- If the equipment is flexible with respect to the supporting structure (fE < fP/2), the vibratory movement is an imposed displacement on the equipment and the reduction factor equals the behaviour factor of the equipment:

$$\rho_E = q_E$$

The comparison between the fundamental frequency of the equipment (f_E) and that of the supporting structure (f_P) can be done considering the relative position of f_E with respect to the peak (or plateau) in the FRS (or in the design spectrum).

The value of the behaviour factors for equipment with zero frequency is conveniently defined in the corresponding guides. In the absence of this information, the minimum value that is accepted according to the behaviour requirements to the material are:

- $q_E = 1.5$ for stability and integrity requirement
- $q_E = 1.0$ for operability requirement

When the effects of the equipment nonlinearities combine with those of the support structure, it should be verified that the resulting reduction factor remains smaller than or equal the behaviour coefficient of the structure being verified.

Below is a representation of the variation of the factors ρ_P and ρ_E as a function of the frequencies of the equipment and its support structure:

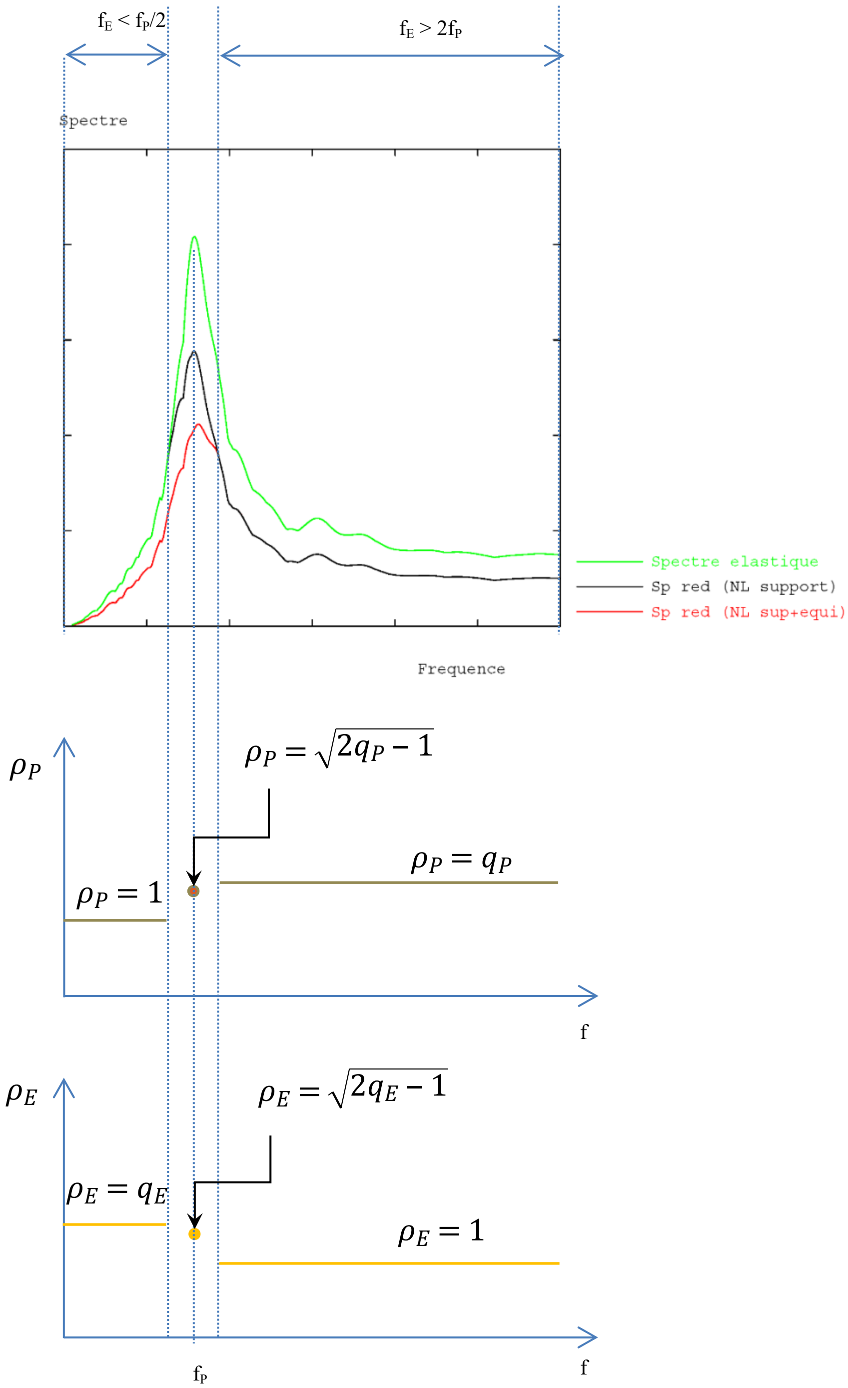


Figure B.4-14 – Variation of reduction coefficients ρ_E and ρ_P as functions of frequency.

When behaviour coefficients above 1.0 for the equipment or the structure are used for designing the anchorages, it should be verified that the ductility required during the seismic action corresponds to the appearance of nonlinearities that justify the behaviour coefficient because:

- the equipment or structure has been consistently dimensioned, or

- for the verification of an existing structure, the seismic level for which the structure must be verified leads to the dissipation of the energy expected.

On the basis of the demands defined following the above criteria, the anchorages can be designed taking into account their nonlinear behaviour. Otherwise the anchorages must display a sufficient ductility. A ductile anchorage is one in which the failure mode leading to its dimensioning is governed by criteria constraining the steel and not the concrete.

To guarantee the ductility of an anchorage system, it is necessary to compare its strength simultaneously with the concrete and steel criteria and it will be considered that the anchorage is ductile if the strength associated to the concrete modes of failure is at least 20% above that corresponding to the bolt steel failure. The comparison is to be made between characteristic values of the bolt steel strength and the concrete strength. In case the concrete is reinforced, the concrete resistance will be the characteristic resistance of the anchor reinforcement:

$$R_{k,concrete} < 1.2R_{k,bolt\_steel}$$

where:

$R_{k,concrete}$ is the characteristic resistance of the concrete, being that of the reinforcement;

$R_{k,bolt\_steel}$ is the characteristic resistance of the anchors with respect to steel failure.

The ductility of an anchorage can be achieved for example by reducing the section of the free part of the stem and increasing that of the part embedded in the concrete, leaving free a sufficient length to allow its elongation to provide the ductility level sought.

The dimensioning or verification of an anchorage from the global demands can therefore be performed:

- (a) considering a ductile behaviour of the anchorage if the above criteria are verified,
- (b) not considering a ductile behaviour of the anchorage even if the above criteria are verified, or
- (c) considering an elastic behaviour of the anchorage because the above criteria are not satisfied.

When assuming elastic behaviour of the anchorage, cases (b) or (c), the distribution of the global demands among the various elements of the anchorage system can be done using linear elastic assumptions. When assuming a ductile behaviour, case (a), the distribution must take account of the nonlinearities of the elements of the anchorage system, assumed to have reached their elastic limit.

A multiplying factor must be incorporated when dimensioning new anchorages for fragile failure modes (case (c)). For those modes the multiplying factor is taken equal to $\frac{(1+\rho_E)}{2}$ and never below 1.5, where ρ_E refers to the reduction coefficient of the elastic demands defined above, depending on the behaviour coefficient of the equipment.

In the other cases (a) and (b), a multiplying factor of 1.25 is applied to the demands considered for dimensioning new anchorages, whether or not the ductility of the anchorage is being taken into account in the evaluation of the demands, in order to ensure that the anchorage does not constitute a weak point of the equipment or the structure.

This multiplying factor can be reduced to 1.0 if it can be easily shown that the procedure for determining the demands to be considered for dimensioning the anchorage is sufficiently conservative (this is the case of a completely elastic calculation for the equipment and the support structure).

Appendix B.4.4 Failure modes

Appendix B.4.4.1 Failure modes for tension demand

The following failure modes can be distinguished for both, post-installed and cast-in place of anchorages, when subject to a tension demand:

- Steel Failure, the steel fails, as illustrated in Figure B.4-15 a.
- Pull-out (Figure B.4-15 b), or pull-through failure (Figure B.4-15 c), combined pull-out and concrete cone failure for bonded anchors (Figure B.4-15 d).
- Concrete cone failure (Figure B.4-15 e). When the cone of failure surrounds a group of anchors this mode is also referred to as group breakout or concrete breakout, or edge break out if the group is near an edge.
- Blowout failure (Figure B.4-15 f).
- Splitting failure (Figure B.4-15 g). Splitting failure is caused by the hoop stresses around the anchor. The hoop stresses originate from local load transfer and expansion forces.

Appendix B.4.4.2 Failure modes for shear demand

The failure modes than can arrive due to shear forces are the following ones:

- Steel failure, which is often accompanied by crushing and spalling of the concrete ahead of the anchor (Figure B.4-16 a).
- Pry-out failure is caused by rotation of the anchor and the catenary tension force generated in the anchor bolt as a result of lateral deformation and the eccentricity between the acting shear force and the resultant resisting force in the concrete (Figure B.4-16 b).
- Pull-out under shear load is generated by the catenary tension force when the pullout resistance of the anchor is insufficient to generate concrete breakout (Figure B.4-16 c).
- Concrete edge failure, which is characterised by the formation of a cone-shaped fracture surface originating at the anchor shaft and radiating towards the concrete edge (Figure B.4-16 d).

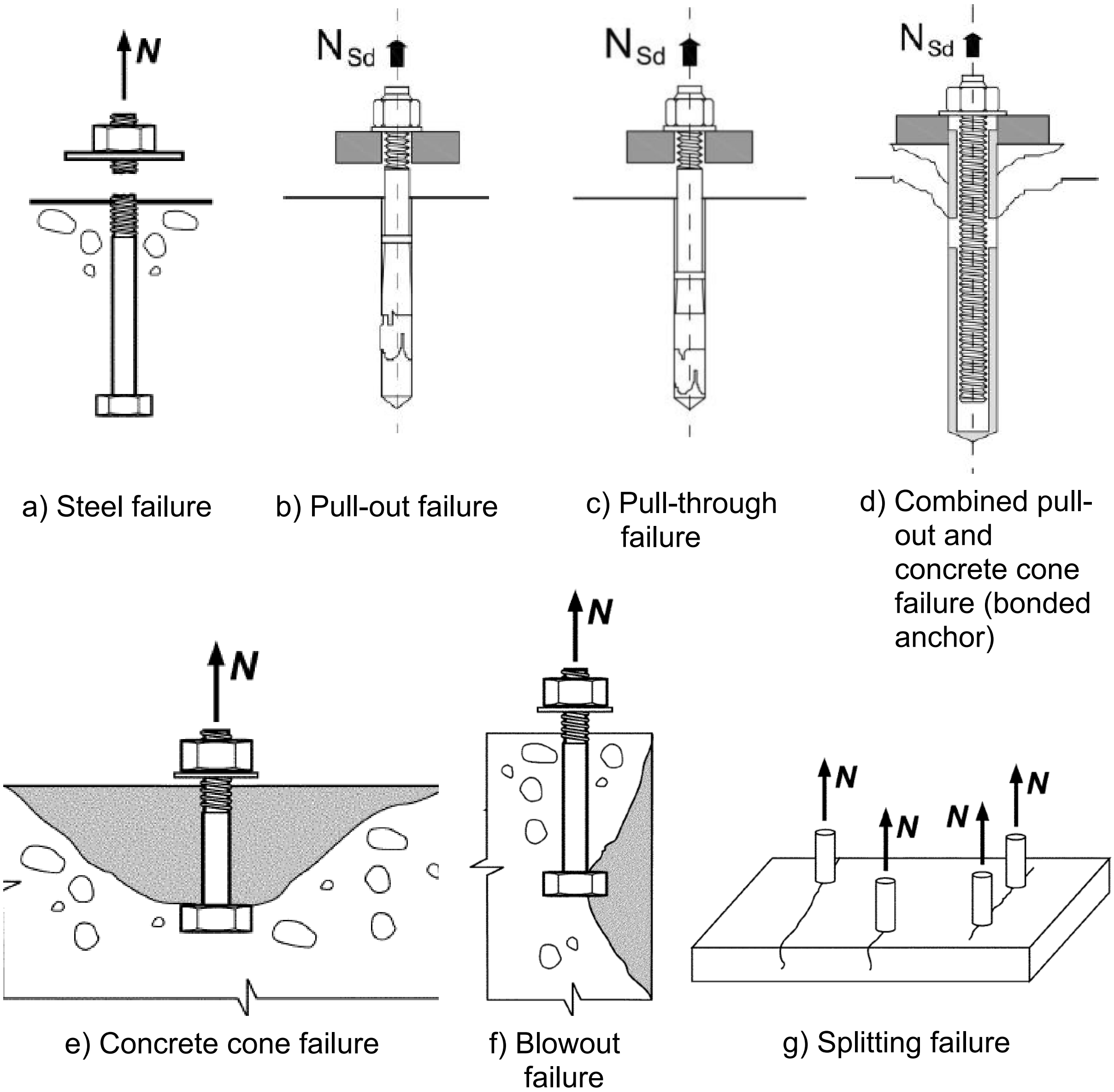


Figure B.4-15 – Failure modes of anchors under tension load (from [58] and [72]).

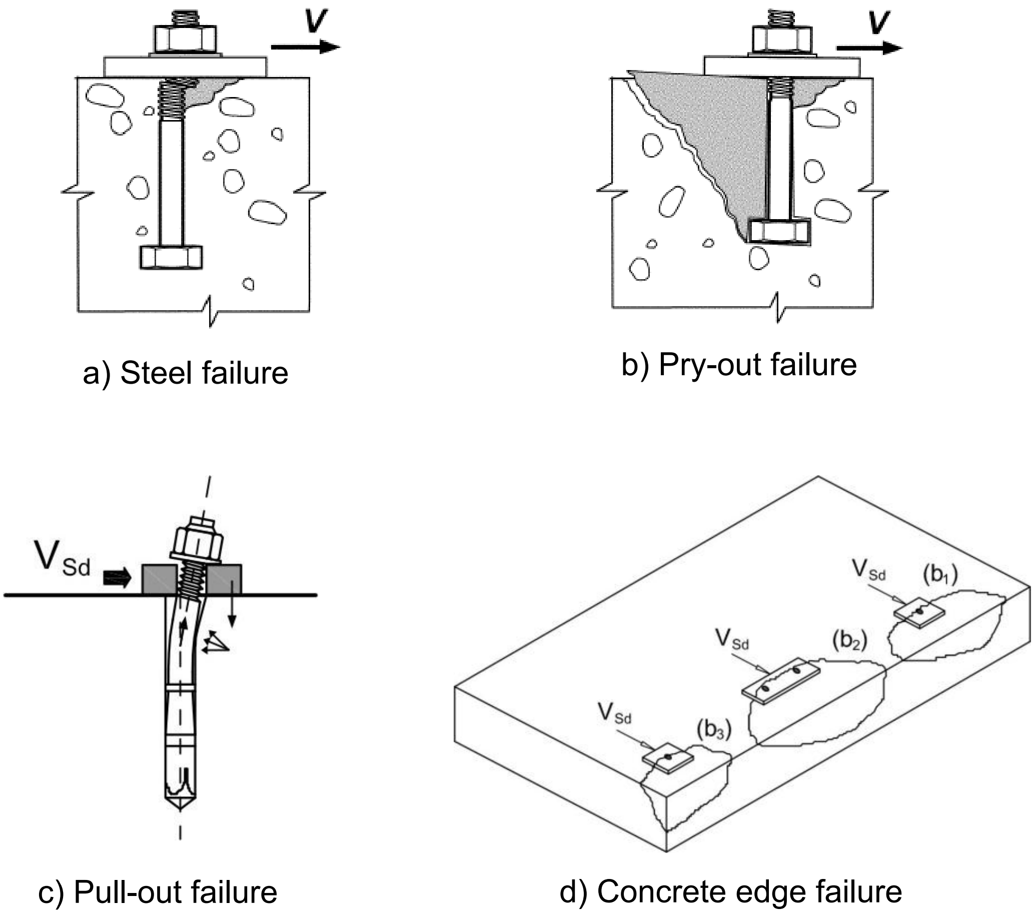


Figure B.4-16 – Failure modes of anchors under shear load (from [58] and [72]).

Appendix B.4.5 Verification of post-installed anchors

Appendix B.4.5.1 General principles

For post-installed anchors, the manufacturer's guidelines shall be followed. The principle of the verification lies with the verification of the capacity of the anchor in tension, in shear, and in combined tension and shear when it undergoes simultaneously both types of demands. The verifications can be written as follows:

- criteria for tensile demand: $\frac{N_E}{N_R} \leq 1$
- criteria for shear demand: $\frac{V_E}{V_R} \leq 1$
- criteria for combined tensile and shear demand: $\frac{N_E}{N_R} + \frac{V_E}{V_R} \leq 1$

where:

N_E and V_E represent the tensile and shear forces applied to the anchor, coming from the global demands exerted on the anchorage;

N_R and V_R represent the tensile and shear capacities of the anchor.

The strength capacities are obtained from the nominal ones using coefficients to take into account:

- the type of bolt,
- the configuration (spacing and edge effects),
- the concrete quality, and
- concrete cracking.

The latter may be replaced, for existing anchors, with the less conservative criterion:

$$\frac{N_E}{N_R} \leq 1, \text{if} \frac{V_E}{V_R} < 0.3$$

$$0.7\frac{N_E}{N_R} + \frac{V_E}{V_R} \leq 1, \text{if } 0.3 \leq \frac{V_E}{V_R} \leq 1$$

The corresponding domains are represented in Figure 9 17.

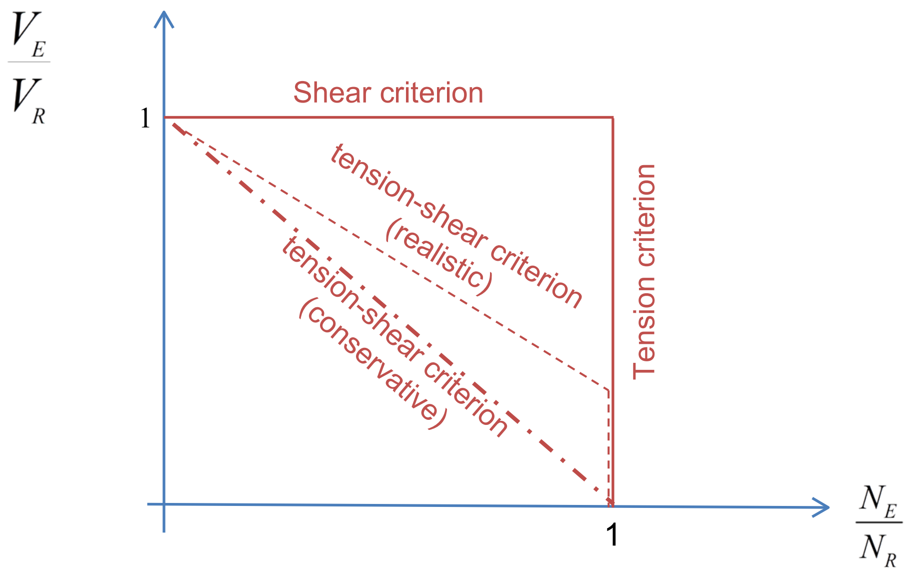


Figure B.4-17 – Tensile and shear criteria for post-installed anchors.

For sensitive materials, such as certain equipment with relays, depending on the functionality requirements imposed by the supplier or the designer, the strength capacity for dimensioning the bolts may be also affected by an additional factor.

The definition of the nominal capacities and the applicable coefficients mentioned above appear in section Appendix B.4.5.2.

Appendix B.4.5.2 Calculation of capacities

The strength capacity of bolts in tension (NR) and shear (VR) are defined as follows:

$$N_R = N_{nom} RT_N RS_N RE_N RC_N$$

$$V_R = V_{nom} RT_V RS_V RE_V$$

where:

N_{nom} and V_{nom} are nominal strength capacities in tension and shear;

RT_N and RT_V are reduction coefficients arising from the type of bolt;

RS_N and RS_V are reduction coefficients arising from bolt spacing

RE_N and RE_V are reduction coefficients arising from distance to a free edge

RCN is a reduction coefficients arising from concrete cracking

<u>Determination of the nominal strength</u>

The nominal capacity of the anchors will in general be taken from the manufactures specification ideally with information about the type of failure (steel failure or pullout failure). The nominal capacity is generally determined by dividing their mean resistance to failure by a safety factor of

3. The nominal strengths are only a function of the diameter of the anchor and are not affected by the type of bolt.

Determination of the specific strength of a bolt

For a given type of bolt, the manufacturers multiply the nominal strength defined above by coefficients for tension and shear in order to obtain the specific nominal strengths of each type of bolt using the factors RT_N and RT_V.

When the type of bolt is unknown, the coefficients take the following default values:

	Nominal diameter < M10	**Nominal diameter > M10**
$\boldsymbol{RT_N}$	0.5	0.6
$\boldsymbol{RT_V}$	0.75	0.75

Table B.4-1 – Default values of the coefficients to calculate capacity of bolts.

The specific strength must take into account the concrete strength. Usually the values provided by the manufacturers already include this effect.

Determination of the reduction coefficients arising from bolt spacing

As a function of D, the bolt diameter, and S, the spacing between axes, the coefficients RS_N and RS_V are determined as follows.

For the normal capacity:

$RS_N = 1 \quad \text{if} \quad S \geq 10D$

$RS_N = \frac{S}{10D} \quad \text{if} \quad 5D \leq S < 10D$

$RS_N = 0.5 \quad \text{if} \quad 2.5D \leq S < 5D$

For the shear capacity:

$RS_V = 1 \quad \text{if} \quad S \geq 2D$

$RS_V = 0.5 \quad \text{if} \quad S < 2D$

Determination of the reduction coefficients arising from distance to a free edge

As a function of E, the distance to a free edge, and D, the bolt diameter, the coefficients RE_N and RE_V are determined as follows:

For the normal capacity:

$RE_N = 1 \quad \text{if} \quad E \geq 10D$

$RE_N = \frac{E}{10D} \quad \text{if} \quad 4D \leq E < 10D$

For the shear capacity:

$RE_V = 1 \quad \text{if} \quad E \geq 10D$

$RE_V = \left(\frac{E}{10D}\right)^{1.5} \quad \text{if} \quad 4D \leq E < 10D$

Determination of the reduction coefficients arising from concrete cracking

The crack opening should remain below 0.25 mm. If more than half of the anchorages are affected by concrete cracking, the factor RCN becomes 0.75. This value should also be used if the concrete support structure has been verified or dimensioned using a behaviour coefficient above 1.5, in particular for the determination of the global forces acting on the anchor. In other cases, $RC_N = 1.0$ will be used.

<u>Other reductions</u>

For equipment particularly sensitive to dynamic excitations, the designer may specify additional reduction factors of the strength capacity of post-installed anchors in order to ensure the survival of the equipment. Such reductions should be established on case-by-case basis using specific studies.

Appendix B.4.6 Determination of capacity for cast-in place anchors

Appendix B.4.6.1 General principles

For all the configurations, the anchor has to be verified with respect to the following modes of failure:

- Tension, that can affect the bolt steel, the adherence of the bolt to the concrete (or the corresponding interfaces if there is any kind of grouting between the bolt and the concrete) or the concrete cone mode of failure in case it is not foreseen in the design of the concrete reinforcement;
- Shear, that can affect the bolt steel, the concrete failure due to diametric compression or the concrete cone failure in case there is an adjacent edge;
- Localised compression at the level of the washer plate, the fixture or the shear key if it exists;
- Bending of the fixture or of the free part of the anchors.

Some of the verifications have to do with combined modes of tension and shear.

When the shear demand is important the anchors have to be provided with a shear key welded to the fixture. In this case the shear demand is completely undertaken by the shear key and the concrete next to it should be conveniently verified against the concentrated stress next to the shear key.

For the anchor verification, the three following inequalities have to be met:

- criteria for tensile demand: $\frac{N_E}{N_R} \leq 1$
- criteria for shear demand: $\frac{V_E}{V_R} \leq 1$
- criteria for combined tensile and shear demand: $\frac{N_E}{1.4N_R} + \frac{V_E}{V_R} \leq 1$

where:

N_E and V_E represent the tensile and shear forces applied to the anchor, coming from the global demands exerted on the anchorage;

N_R and V_R represent the tensile and shear capacities of the anchor.

In case the anchorage is subject to a shear demand and the bolt undergoes a bending moment as a result of the eccentricity of the shear demands (V_E or $F_{v,Ed}$) acting on a free part of the bolt, the bending moment is considered by increasing the tensile force N_E with an equivalent tensile demand on the bolt $N_{V,Eq}$. Depending on the case, the eccentricity can be calculated as indicated in Figure B.4-18.

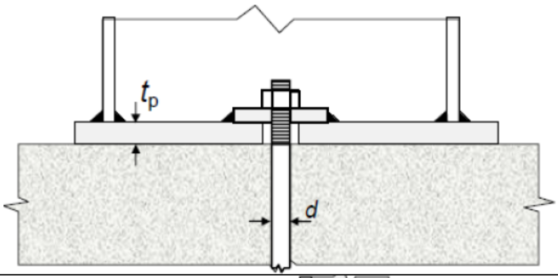
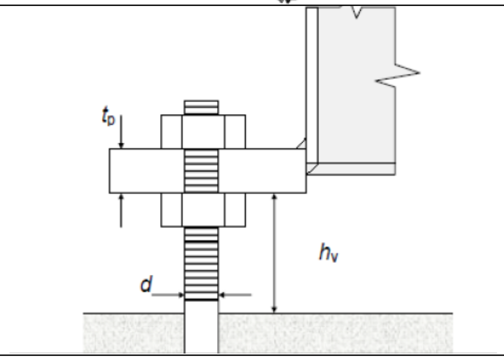
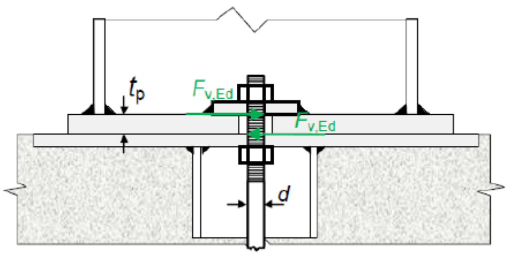
Description	Sketch	Eccentricity
Single plate with an overdimensionned hole		$e = t_p + \frac{d}{2}$
Flange with no levelling concrete		$e = h_v + \frac{d}{2}$
Plate with over dimensioned holes		$e = t_p$

Figure B.4-18 – Calculation of the excentricity for shear demands.

The equivalent tensile demand is given by: $N_{E,V,Eq} = V_{Ed}\frac{e5\pi}{d\,6}$.

For existing installations, the criteria for combined tensile and shear demand can be replaced by: $\left(\frac{N_E}{N_R}\right)^{\alpha} + \left(\frac{V_E}{V_R}\right)^{\alpha} \leq 1$ with $\alpha = \frac{5}{3}$.

Note that special provisions regarding the criteria for combined tensile and shear demand within the Tokamak Complex are given by the Architect Engineer in [38] based on CEB-FIP [72].

The comparison of both criteria for combined tension and shear demand is presented in Figure B.4-19.

The calculation of strength capacities of cast-in anchors for tension and shear and for the different modes of rupture is indicated in section Appendix B.4.6.2. These modes of rupture have already been discussed in section Appendix B.4.4.

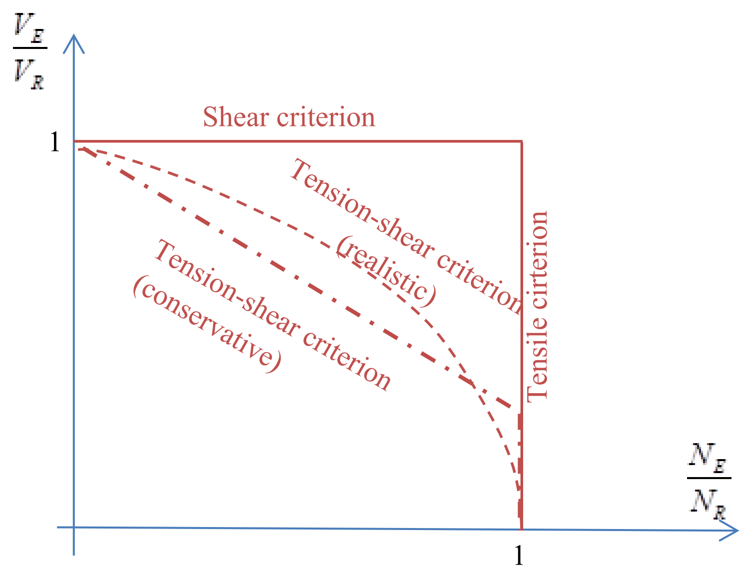


Figure B.4-19 – Calculation of the excentricity for shear demands.

Appendix B.4.6.2 Calculation of capacities

For the design of anchorages in new installations the capacities in tension and shear are concluded from the principles in Eurocodes 2 and 3, completed with the guide of CEB FIP [72] for the design of anchors and other related documentation that will be cited along the text. In particular, additional provisions for the Tokamak Complex are given by the Architect Engineer in [38] based on CEB-FIP [72].

The calculation of capacities is detailed here below.

Appendix B.4.6.2.1 Steel tensile resistance

The tensile resistance of the threaded section of a bolt is given by:

$$F_{t,Rd} = \beta_P \min\left\{\frac{0.9 f_{ub} A_s}{\gamma_{M2}}; \frac{f_y A_s}{\gamma_{M0}}\right\}$$

where:

A_s is the effective area of the threaded section;

f_{ub} is the ultimate steel resistance in tension;

f_y is the steel elastic limit;

β_P is a quality coefficient that takes the value of 1 if the bolts comply with the requirements in EN 15048 and 0.85 otherwise.

For the partial coefficients the adopted values are: $\gamma_{M0} = 1$ and $\gamma_{M2} = 1.25$.

Appendix B.4.6.2.2 Resistance against pullout failure: straight anchor

The ultimate adherence strength is given by the following expression:

- For a plain bar: $f_{bd} = \frac{0.36\sqrt{f_{ck}}}{\gamma_c}$
- For a high adherence bar: $f_{bd} = \frac{2.25\, f_{ck}}{\gamma_c}$

where:

f_{ck} is the characteristic compression resistance of concrete in cylindrical sample according to Eurocode 2 [46];

γ_c is the partial coefficient for the concrete resistance and adopts the value of 1.3.

For a straight bolt with diameter d and anchored along a length lb, the tensile force that can be mobilized by adherence is: $F_{t,c,Rd} = \pi d l_b f_{bd}$.

Appendix B.4.6.2.3 Resistance against pull-out failure: hooked anchor

In the general case of a hooked anchor, defined by a bolt with a diameter d, the length of the straight part being $\lambda_1 d$, with a curvature radius ρ_d along a cumulative length $\rho_d \vartheta$ and extended by a straight length $\lambda_1 d$, the tensile force that can be mobilised by adherence can be written by summing up the terms of friction and adherence:

$$F_{t,c,Rd} = \pi d \lambda_1 d f_{bd} + \psi(\theta) \pi d \rho d f_{bd} + \psi^{'}(\theta) \pi d \lambda d f_{bd} \qquad \text{with } \psi(\theta) = \frac{1}{\mu}(e^{\mu\theta} - 1)$$

where: μ is the friction coefficient between concrete and steel taken by default as 0.3.

For the particular case of an anchorage J-shaped ($\vartheta = 90^{\circ}$) the previous expression can be written:

$$F_{t,c,Rd} = \pi d\,(\lambda_1 + 6.4\rho d + 3.5\lambda d) f_{bd} \qquad \text{with } 1.5 \leq \lambda \leq 2$$

In case $\rho \geq 3$, the concrete failure does not need to be verified. For smaller radius of curvature, a decrease in the adherence capacity in the area of curvature is expected; hence the previous expression needs to be modified consistently. In any case the concentrated compression in the concrete has to be verified.

Specific concrete reinforcement can be placed in order to limit the risk of pull-out failure. A guidance is presented in Figure B.4-20.

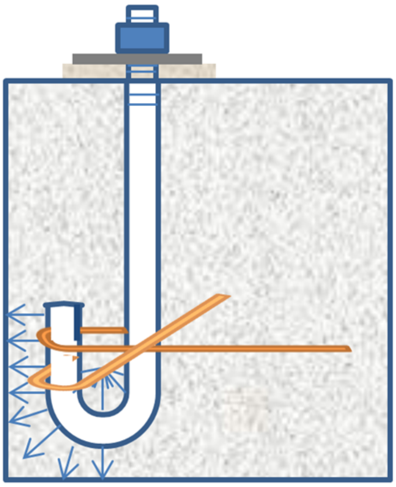


Figure B.4-20 – Calculation of the excentricity for shear demands.

Appendix B.4.6.2.4 Tensile resistance of an anchor with washer plate

An anchor with a washer plate at a depth l, with thickness tr and radius r, the tensile capacity is mainly controlled by the compression resistance of the concrete under the action of the washer plate. This strength can be expressed as:

$$F_{t,c,Rd} = k f_{cd} \pi \left(r^2 - \frac{d^2}{4}\right)\left(1 - \frac{r}{v}\right) \text{ being } v = \min\,(l, d_1, p)$$

where:

d_1 is the distance of the anchor to the free edge;

p is the distance between anchor axes;

k is a coefficient for increasing the concrete compression strength accounting for its confinement, varying between 2.55 and 3.3;

f_{cd} is the concrete strength.

This verification is under the assumption that the plate is rigid enough for transmitting the tensile demands onto the concrete with negligible deformations. The following condition guarantees this assumption:

$$t_r \geq 0.3r$$

In general, the adherence of the straight part of the anchor is neglected. For the verification of existing structures it could be included in the previous expression by adding the term: $\pi d l f_{cd}$.

For this type of anchor it is enough verifying the effects of the tensile demand on the concrete with the above expression for $F_{t,c,Rd}$, as long as the concrete is reinforced appropriately in order to avoid the appearance of a concrete cone starting on the washer plate up to the surface. Additionally the anchors should go down to a depth below the reinforcing rebar level of the opposite concrete face. If any of these dispositions is not verified, the concrete cone mode of failure should be verified. This resistance can be calculated as:

$$F_{t,cc,Rd} = 3C_p l^2 \frac{f_{ct}}{\gamma_c} \quad \text{with } C_p = \left(\frac{1}{2} + \frac{p}{2l}\right) \leq 1$$

where: f_{ct} is the concrete tensile strength.

If the concrete cone interacts with a free edge or with the interface between a grouting product and concrete already in place, the previous expression has to be adapted for considering the interaction between cones or with the interfaces between the concrete and the grouting products.

If several concrete cones interact, the group effect has to be verified. Provisions given by the Architect Engineer in [38] based on CEB-FIP [72] for the Tokamak Complex can be used as reference.

The side-face blowout failure (section Appendix B.4.4.1, Figure B.4-15 f) can be prevented by providing enough edge distance. In general terms, if the distance of the anchor axis to the nearest edge is at least 0.5 times the effective embedment depth lb, this failure mode does not need to be verified. In case it is needed, a detailed guidance can be found in section 7.2.1.8 of the draft EC2-4 [48] and in the provisions given by the Architect Engineer in [38].

Regarding the concrete splitting failure mode, again the draft EC2-4 [48] and the provisions given by the Architect Engineer in [38] give guidance on when it needs to be verified and how.

Appendix B.4.6.2.5 Steel shear resistance

According to Eurocode 3 the shear resistance of the anchor steel is given by:

$$F_{vb,Rd} = \frac{(0.44 - 0.0003\, f_{yb}) f_{ub} A_s}{\gamma_{M2}} \quad (f_{yb} \text{ in MPa})$$

If there is no shear key, the shear demands are transmitted to the concrete by diametric compression of the bolt against the concrete. The maximum strength that can be mobilised by this mechanism is given by the following expression:

$$F_{vb,c,Rd} = 0.28 d^2 \frac{\sqrt{f_{ck} E_c}}{\gamma_c} \quad \text{if } \frac{l}{d} = 3$$

$$F_{vb,c,Rd} = 0.36 d^2 \frac{\sqrt{f_{ck} E_c}}{\gamma_c} \quad \text{if } \frac{l}{d} \geq 4.2$$

where:

E_c is the concrete Young modulus;

f_{ck} is the characteristic resistance of concrete.

For intermediate values of l/d, a linear interpolation can be done.

Under shear demands it is also needed to verify the blowout failure of a concrete cone in case there is a free edge close to the anchor and no adhoc reinforcement has been foreseen. Provisions given by the Architect Engineer in [38] based on CEB-FIP [72] for the Tokamak Complex can be used as reference. This verification can be done under the same hypothesis that for the concrete cone failure due to tension demands, requiring a capacity for the concrete cone mode of failure of $F_{vb,c,Rd}$.

Regarding the pry-out, pull out and concrete edge shear failure modes the draft EC2-4 [48] and the provisions given by the Architect Engineer in [38] give guidance on when it needs to be verified and how.

Appendix B.4.6.2.6 Final values of resistance

The final tensile strength is given by:

$$N_R = F_{t,anchorage,Rd} = \min(F_{t,Rd}, F_{t,c,Rd}, F_{t,cc,Rd})$$

And the one for shear strength to:

$$V_R = F_{vb,anchorage,Rd} = \min(F_{vb,Rd}, F_{vb,c,Rd}, F_{vb,cc,Rd})$$

These values are to be considered in the formulae indicated in section Appendix B.4.6.1 together with the demands calculated according to section Appendix B.4.3.

Appendix B.4.6.3 Acceptable anchor length and capacity of standards anchors

In practice, the acceptable values of anchor length for seismic loads are between 10 and 20 times the anchor diameter. Tables of anchor length and capacity are given hereafter.

See AirLiquide document for the following anchors:

Bolt Type	Full description
PL	Plate type anchor bolt
HK	Hooked anchor bolt
LN	Linear anchor bolt
CH	Chemical anchor bolt
TR	Transverse anchor bolt
SP	Special type anchor bolt

Table B.4-2 – Anchors length requirement.

Appendix B.4.7 Design of Anchor reinforcement

Appendix B.4.7.1 Design Philosophy

As indicated previously, the system is required to have adequate ductility, which in general terms means that the anchorage strength should be governed by ductile yielding of a steel element. For this to happen, failure modes associated with concrete failure modes need to be treated with a more conservative approach than in a conventional design.

This idea was already introduced in section Appendix B.4.3.5. In some cases, it is not possible to achieve this requirement, given geometrical constraints (i.e. anchor depth) or simply because the anchorage provider indicates the anchor capacity with no further information about the failure modes.

If there is no guarantee that the anchorage is design to have a ductile failure mode, a way to achieve this goal is by the introduction of supplementary reinforcement. In general terms, the reinforcement should be proportioned to develop the strength of the anchor.

When reinforcement is used to restraint concrete breakout (concrete cone failure when a group of anchors is involved, see section Appendix B.4.4), the overall anchorage design should ensure that there is sufficient strength corresponding to the other failure modes described in section Appendix B.4.4.

Appendix B.4.7.2 Reinforcement for tension forces

In general terms, for a single anchor, the total area of needed reinforcement A_{st}should be calculated as follows:

$$A_{st} \geq \frac{A_s f_{ub}}{f_{yr}}$$

where: f_{yr} is the yield strength of the reinforcement bars.

This is the proposal in [80], which is in agreement with what is indicated in CEB FIP [72]. Other standards related to anchor design, such as draft EC2-4 [47] or the American ACI 318 Appendix D [58], do not provide specific guide on the dimensioning of this reinforcement.

The supplementary reinforcement should comply with the following requirements:

- The reinforcement bars should be distributed around the anchor, symmetrically, to minimize the effect of eccentricity, and all with the same diameter;
- It should preferably enclose the surface reinforcement;
- They should be as close as possible to the anchor bolt and in any case at a distance from the anchor axis not greater than 0.75 l_b;
- The maximum reinforcement bar diameter is 16 mm, according to draft EC2-4 [47], with a characteristic yield strength not higher than 600 MPa, again as per draft EC2-4 [47];
- Ideally, the supplementary reinforcement should extend all along the anchor length; however if this is not possible due to interferences with other reinforcement needs, a the needed anchorage length l1 of supplementary reinforcement in the concrete cone failure cone has to be adequately calculated according to the guidance given below;
- The supplementary reinforcement shall be anchored outside the assumed failure cone with an anchorage length according to EC2-1 [46], or the applicable code in the context of ITER.

The provisions given in EC2-4 [47] regarding minimum values of l1, although valid for a conventional design, do not necessarily guarantee that the needed capacity is reached. The force for one leg (straight bar) of a hanger reinforcement, like the one presented in Figure 9 21, can be calculated, according to EC2 [46], also referred to in [79]:

$$F_{ru} = \frac{\pi \, \phi_s \, l_1 \, f_{bm}}{\alpha_b}$$

Where:

ϕ_s is the reinforcement diameter;

f_{bm} is the average bond stress, which can be considered in this case equal to 2.25f_ctd, according to EC2 [46], which is the design value of the concrete tensile strength;

α_b is a factor that takes into account the influence of hook and can be considered equal to 0.7, according to EC2 [46].

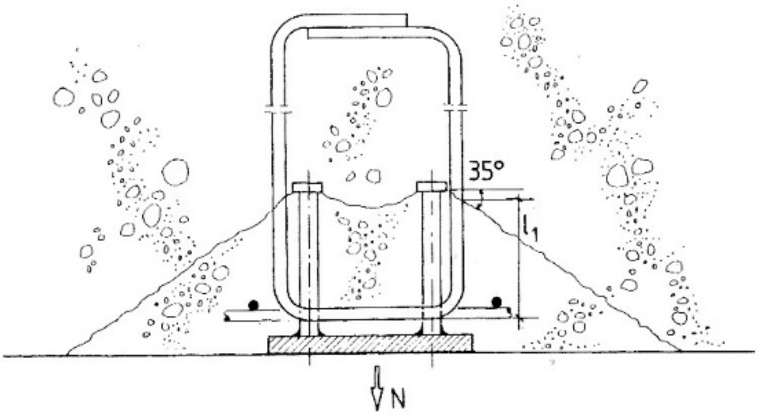


Figure B.4-21 – Anchors with hanger reinforcement (stirrups) from [79].

This means that for the reinforcement being able to develop the rebar capacity according to its area and tensile strength, the length l_1 of supplementary reinforcement within the concrete cone should be given by the following formula, which is easily derived by making equal the previous value of F_{ru} to the bar axial strength:

$$l_1 \geq \frac{\alpha_b \, \phi_s \, f_{yr}}{4 \, f_{bm}}$$

Figure B.4-22 shows another example, from EC2-4 [47] of anchorage with supplementary reinforcement designed for tension loads.

The above formulae are valid if the supplementary reinforcement is close to the anchor. This is implicitly needed, in practical terms, for being able to have the needed value of l_1.

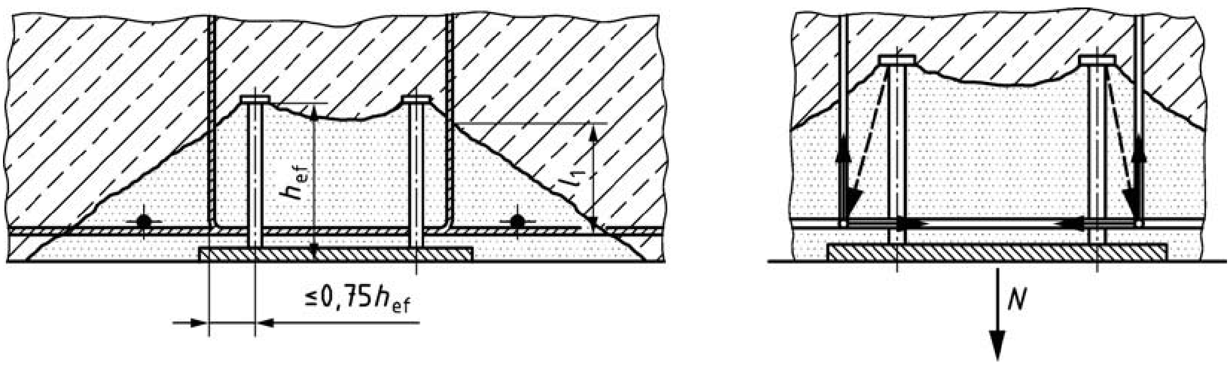


Figure B.4-22 – Anchorage with supplementary reinforcement (h_{ef} in the picture is named l_b in the text) [47].

A distribution can consist, for instance of four bars equally distant and equally spaced around the anchor, providing the total area of needed reinforcement. In case the needed area cannot be accomplished with four bars, the condition of having a maximum diameter of 16 mm, eight bars, grouped two by two, can be disposed. A typical disposition is presented in Figure B.4-23.

The concrete cone failure can be assumed to have a surface rupture that forms an angle of 35º with the normal to the bolt axis (or 55º with the anchor axis). When the anchorage has a base plate (headed anchors, see section Appendix B.4.2.2.2) the concrete cone is assumed to be formed from the exterior tip of the base plate (Figure B.4-23).

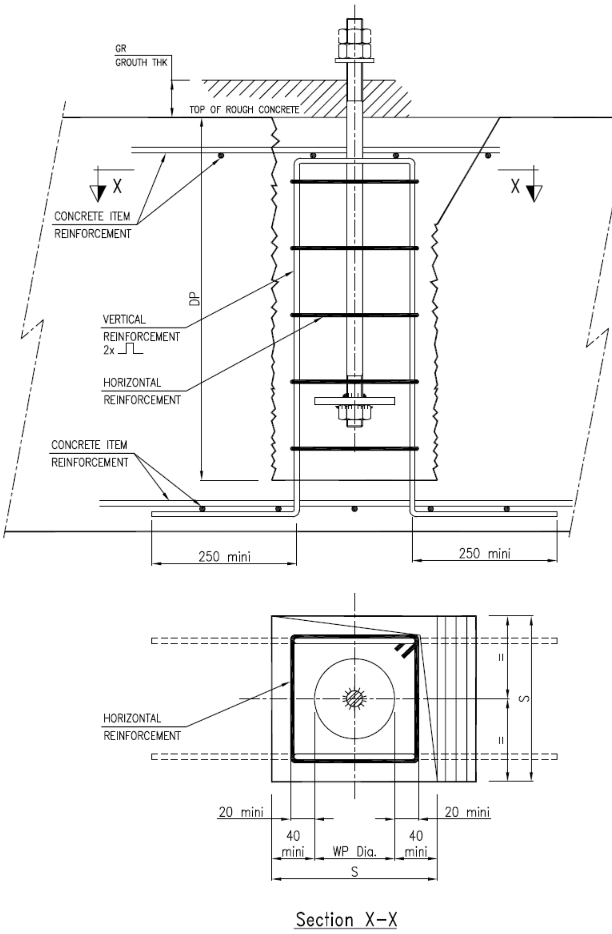


Figure B.4-23 – Typical reinforcement for cast-in headed bolt.

Appendix B.4.7.3 Reinforcement for shear forces

The supplementary reinforcement shall be designed to resist the shear strength of the anchor steel.

The shear supplementary reinforcement may be in the form of a surface reinforcement or in the shape of stirrups or loops.

Strut-and-tie models can be used to analyse shear transfer to concrete and conclude from it the bar diameter needed; this way of calculation is

The maximum reinforcement bar diameter is 16 mm, according to draft EC2-4 [47], with a characteristic yield strength not higher than 600 MPa, again as per draft EC2-4 [47].

As indicated in draft EC2-4 [47], when the design relies on supplementary reinforcement, concrete edge failure need not to be verified but the supplementary reinforcement shall be designed to resist the total load. The supplementary reinforcement may be in the form of a surface reinforcement or in the shape of stirrups or loops.

Appendix B.5 Design and Verification of Vessels

Appendix B.5.1 Main failure modes induced by the seismic action on vessels

Some of the more typical failure modes induced by seismic action on vessel tanks are presented in this section. Knowing the potential failure modes, as well as the mechanisms that generate them, helps having a better understanding of the design and verification process.

The ground motion can induce liquefaction in the soil around and beneath the tanks. The phenomenon is most often observed in saturated, loose, non-cohesive soils. During liquefaction, due to the excess pore pressures generated, the soil loses its effective pressures, it has no shear resistance and hence it behaves as a liquid. The more frequent consequences for tanks are differential settlements and bearing failures. Figure B.5-1 and Figure B.5-2 show two cases of tanks with differential settlements due to the liquefaction phenomena experienced in the surrounding soil.

Figure B.5-1 – Soil liquefaction around tanks. Kobe 1995 earthquake, M 6.7.

Figure B.5-2 – Tank settlement due to liquefaction around the tank foundation.

Buckling is a form of instability of a structural member that can be triggered by compressive stresses below the ultimate stress that the material is capable of withstanding. Two types of buckling can be observed at the lower part of the tank, depending on the internal pressure:

- Elastic-plastic buckling of the tank wall near the bottom edge, caused by the compressive forces associated with overturning moments; this mode of buckling is usually referred to as "elephant's foot" buckling due to its characteristic shape. This type of buckling typically occurs under high internal pressure accompanied by axial compression in the shell structure. Examples of this mode of failure are presented in Figures 10-3 and 10-4. Since the tank shell courses decrease their thickness with increasing elevation, this phenomenon can also occur at an intermediate level, known in this case as "elephant's knee" buckling (see Figure B.5-5).
- Elastic buckling of the tank wall, also known as diamond shaped buckling modes. This mode of failure is presented in Figure B.5-6.

When the fluid inside the vessel is a liquid, its sloshing motions can damage the roof and the top of tank wall. Examples of this type of damage are presented in Figure B.5-7 and in Figure B.5-8.

In case of floating roofs, they can slide off the tank, as can be seen in Figure B.5-9.

Finally, anchorages can be damaged or even ruptured by the seismic action. A couple of examples are presented in Figure B.5-10.

Figure B.5-3 – Elephants's foot buckling. San Fernando earthquake in 1971, M 6.6.

Figure B.5-4 – Elephants's foot buckling. Anchorage earthquake in 1964, M 9.2.

Figure B.5-5 – Elephants's knee and foot buckling. Haiti earthquake in 2010, M 7.0.

Figure B.5-6 – Diamond-shaped buckling.
Livermore (California) earthquake in 1980, M 5.9.

Figure B.5-7 – Sloshing damage to upper shell of a tank.
San Fernando (California) earthquake 1971, magnitude 6.6.

Figure B.5-8 – Sloshing damage to upper shell and roof of a tank.
Marmara (Turkey) earthquake 1999, M 7.4.

Figure B.5-9 – Sliding of floating roof. Tokachi-oki (Japan) earthquake 2003, M 8.3.

Figure B.5-10 – Damage and rupture of anchorages.
Kobe (Japan) earthquake 1995, M 7.3.

References from [61] to [77] cover the background information considered in the preparation of the present Chapter.

Seismic input motions leads to an increase of generated stresses, liquid pressures, forces and displacements. In principle, this type of consequences can be handled simply by increasing material quantities; in this case the seismic effects on quantities and costs are gradual, growing with the size of the design motions. However, there are some thresholds beyond which the strengthening process cannot be pursued continuously; at those points, either a new feature must be incorporated to the design or, in some cases, the construction of the tank becomes impossible.

One of the consequences of the earthquake is sloshing, the generation of standing waves in the free surface of the liquid. The predicted wave height must be incorporated as additional freeboard of the tank in order to prevent spills or, alternatively, the consideration of the hydrodynamic pressures exerted on the roof structure.

Apart from sloshing, which involves the so-called convective liquid mass, the movements of the rest of the liquid mass, the impulsive mass, entail important pressure variations in the liquid. The same occurs with the vertical ground movements, which also excite the liquid mass. All of them imply departures from the pre-existing hydrostatic pressures, which the tank must be able to deal with (see Figure B.5-11).

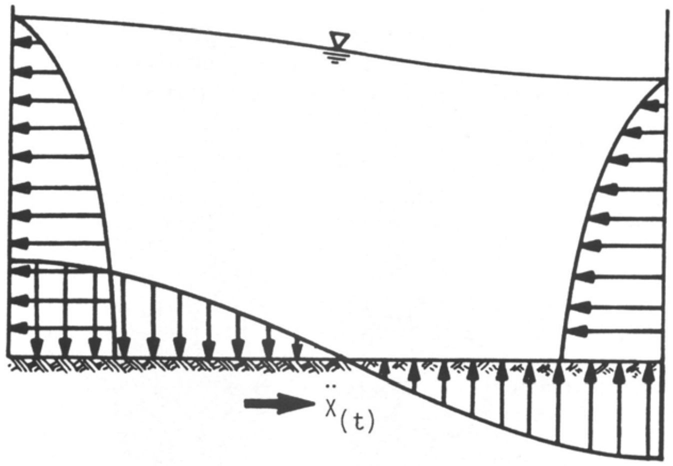


Figure B.5-11 – Hydrodynamic pressure induced by ground acceleration.

Additionally rocking excitations may lead to excessive compression of the tank wall (producing the elephant's foot buckling) and/or lift-off at the opposite corner of the tank; anchor straps may provide some help in respect of the latter problems, although that strategy is not totally free of uncertainties in their performance during the seismic movements. The problems linked to rocking are of course alleviated by a flatter aspect ratio of the tank, though this strategy has adverse implications on space occupancy.

Another undesirable response of the tank would be gross sliding. The horizontal demands, coupled with a dynamically decreased vertical weight, may lead to gross sliding of the tank. There is little that the designer can do to avoid this problem if it does tend to occur, even a flatter aspect ratio would not improve the situation.

Finally, if the vertical accelerations were sufficiently high, the upward vertical forces might exceed the static weight, whereupon the tank and the liquid would lift off globally. Again, no practical solution exists for this problem.

In summary, the following problems, as developed by a gradually increasing seismic input, will be taken into account together with their solutions when they are feasible:

- Larger liquid pressures and increased compression of the wall. The traditional solution is to use a thicker shell and/or provide additional stiffening.
- Corner uplift. When expected, a common solution strategy is to anchor the inner tank in spite of the possible disadvantages already mentioned.
- Gross sliding of the tank. This problem has no known solution, even changing the aspect ratio of the tank will not resolve it.
- Global uplift of the tank. Again, when this is expected, no practical solution is known.

The first two items above are not fatal, in the sense that they simply require additional expenditure. The last three, however, may be alleviated if seismic isolation can be incorporated, otherwise the tank can no longer be built according to the current standards.

It is clear that the use of seismic isolation can decrease the seismic demands developed in the tank structure. Its beneficial action can be exerted in two ways. The first way is reducing the spectral acceleration, under the assumption that the fundamental frequency lays on the plateau region of the design spectra. A second way in which the isolation can be beneficial is by dissipating energy in the case of systems that have this capability.

Appendix B.5.2 Hydrodynamic pressures and forces

Appendix B.5.2.1 Horizontal Earthquake – Rigid Tank:

Unless a more rigorous analysis is undertaken, the earthquake induced horizontal hydrodynamic forces on a tank with rigid walls shall be determined from the spring-mass analogy show in Figure B.5-12. The mass m_1 at height h_1 represents the hydrodynamic effects of the first convective (or sloshing) mode of vibration and the mass m_0, rigidly attached to the walls, represents the hydrodynamic effects of the impulsive mode or rigid body displacement of the tank wall.

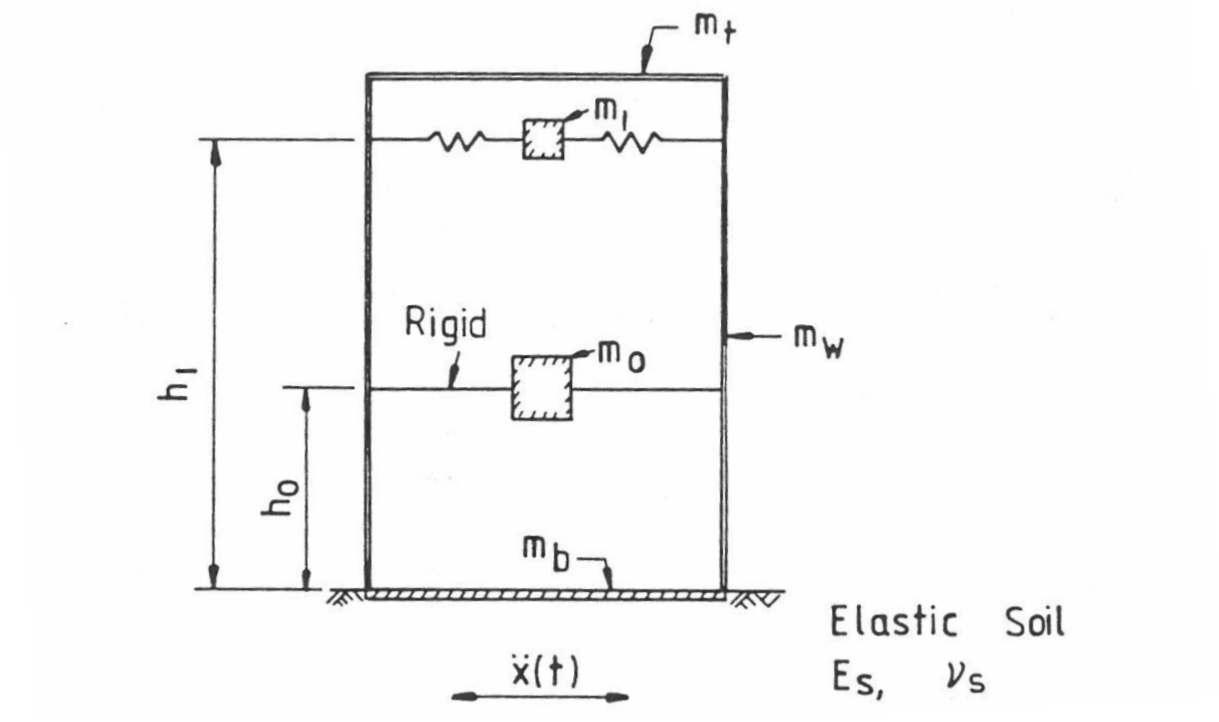


Figure B.5-12 – Spring-mass analogy for horizontal earthquake rigid tank.

Appendix B.5.2.2 Horizontal Earthquake –Flexible Tank:

Unless a more rigorous analysis is undertaken, the earthquake induced horizontal hydrodynamic forces on a tank with flexible walls shall be determined from the spring-mass analogy show in Figure B.5-13. The mass m_1 at height h_1 represents the hydrodynamic effects of the first convective mode of vibration; the mass m_r at height h_r represents the hydrodynamic effects of the rigid body displacement of the tank wall, and the mass m_f at height h_f represents the influence of the deformation of the tank wall relative to the base. The mases m_r and f_r correspond to two separate impulsive modes of vibration. Mass m_r is given by:

$$m_r = m_0 - m_f$$

where: m_0 is the rigid tank impulsive mass.

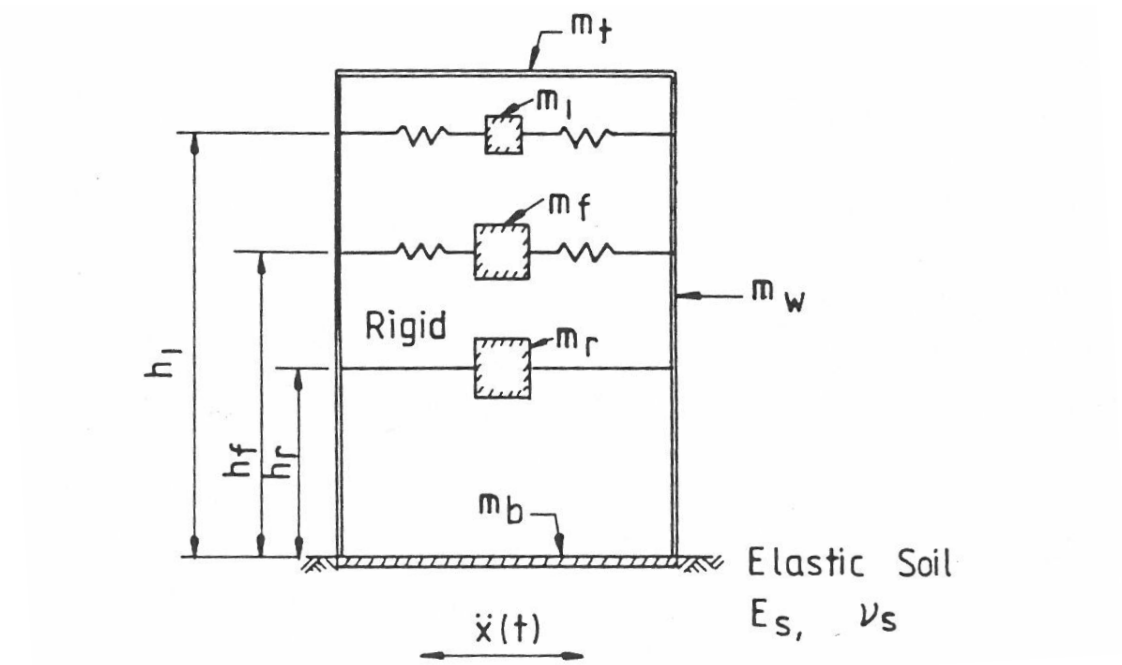


Figure B.5-13 – Spring-mass Analogy for horizontal earthquake flexible tank.

Appendix B.5.2.3 Vertical Earthquake – Rigid Tank:

Unless a more rigorous analysis is undertaken, the hydrodynamic pressures induced by the vertical earthquake shall be determined from the spring-mass analogy show in Figure B.5-14. The mass m_l shall have the total mass of the fluid and shall be rigidly connected to the shell base. The effect of the vertical earthquake on the hydrodynamic pressures shall be considered as a factored increase in the static pressures. The additional pressure is the product of the vertical spectral acceleration divided by g and the static pressures.

Appendix B.5.2.4 Vertical Earthquake – Flexible Tank:

The hydrodynamic pressures induced by the vertical earthquake shall be determined from the spring-mass analogy show in Figure B.5-15. The mass ml shall have the total mass of the fluid and shall be connected to the shell base by a spring representing the flexibility of the tank walls. The hydrodynamic pressures from vertical earthquake shall be determined in a manner similar to that given above for the rigid tank. However for the flexible tank case the influence of the wall flexibility on the vertical or breathing mode period shall be considered.

Appendix B.5.2.5 Base Pressures

The hydrodynamic pressures acting on the base of the tank (as well as on the walls) shall be included in the analysis of the tank support system and the soil foundation.

Appendix B.5.2.6 Foundation flexibility

The influence of foundation soil-structure interaction on the response of both horizontal and vertical spring-mass systems shall be considered.

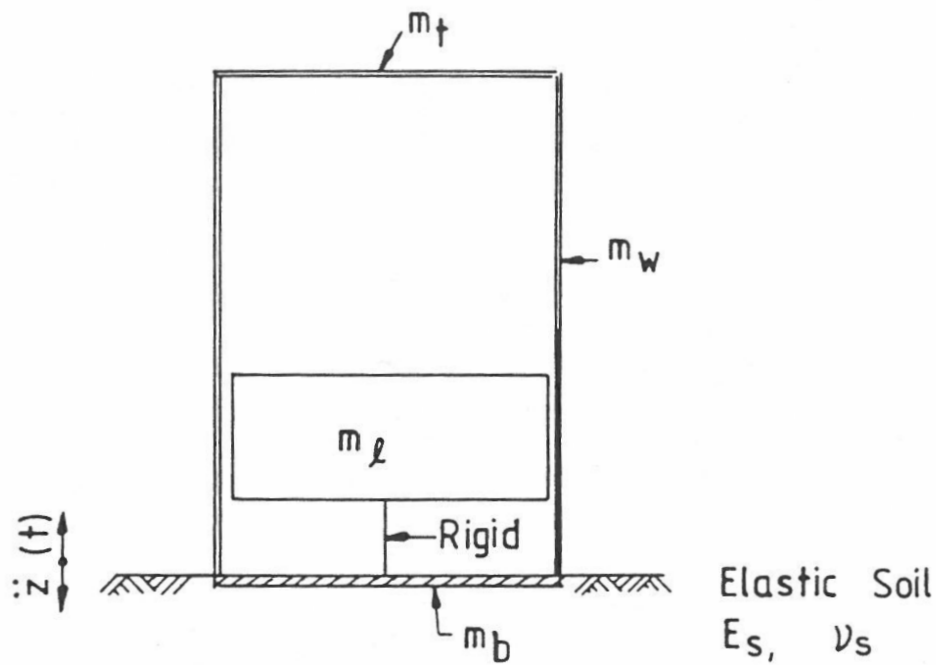


Figure B.5-14 – Spring-mass analogy for vertical earthquake rigid tank.

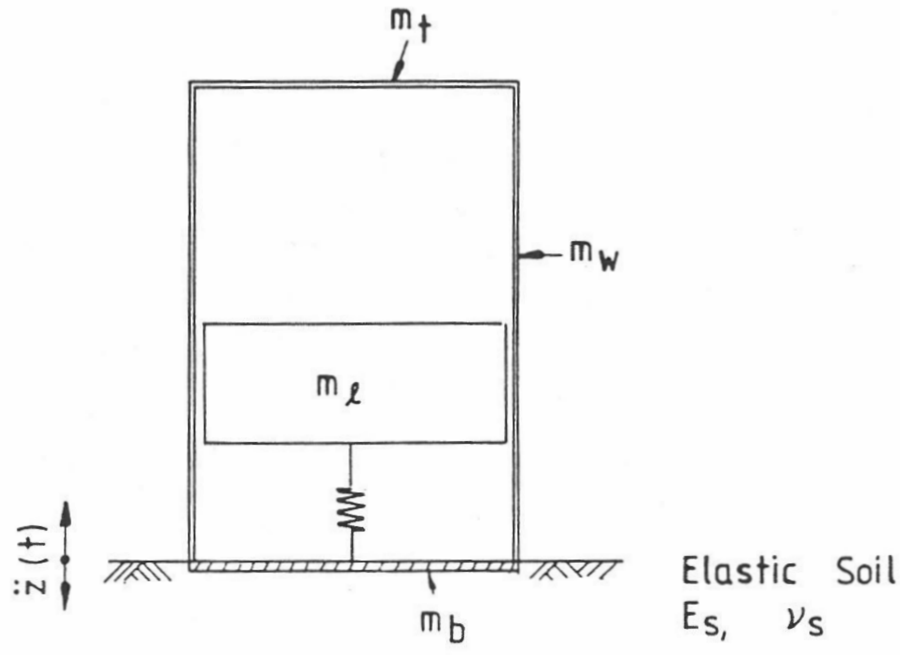


Figure B.5-15 – Spring-mass analogy for vertical earthquake flexible tank.

Appendix B.5.3 Shell inertia forces

Appendix B.5.3.1 Horizontal Earthquake – Rigid Tank:

The earthquake forces from the horizontal inertia of the tank walls and roof shall be included in the analysis of the earthquake stresses in the tank shell and foundation. The inertia loads from the tank base and any support system or structural foundation shall be included in the analysis of the stresses in the foundation.

The earthquake forces arising from the inertia of the shell, base and structural foundation act at the centre of gravity of the respective masses. The effect of these masses on the impulsive mode period shall be considered.

Appendix B.5.3.2 Horizontal Earthquake – Flexible Tank:

For flexible tanks, other than concrete tanks, the mass of the shell may be neglected. Where a more rigorous analysis is undertaken, the mass of the shell shall be included in the hydrodynamic mass (m_f) that represents the influence of the deformation of the tank wall. To calculate periods of vibration and stresses in the foundation, the mass from the tank base and any support system or structural foundation shall be included.

Appendix B.5.3.3 Vertical Earthquake – Rigid Tank:

The vertical inertia loads on the tank roof, walls, base and any support system or structural foundation shall be included in the analysis of the earthquake stresses in the foundation. These masses shall also be used in the calculation of the period of the vertical mode of vibration.

The vertical inertia forces from the structural masses may be neglected for the analysis of the stresses in the tank shell.

Appendix B.5.3.4 Vertical Earthquake – Flexible Tank:

A detailed analysis of the vertical response of a flexible tank on a flexible foundation requires a two mass analogy. However, this refinement adds considerable complexity to the analysis and will generally be unnecessary for design purposes. Unless a more rigorous analysis is justified, the single mass analogy shall be used with the vertical inertia loads on the tank roof, walls, base and any support system or structural foundation included in the analysis of the earthquake stress in the foundation. The influence of theses masses on the vertical period of vibration may be neglected and the inertia forces on them may be estimated by assuming a vertical force coefficient corresponding to zero period on the acceleration response spectrum.

As for the rigid tank case, the vertical inertia forces form the structural masses may be neglected for the analysis of the stresses in the tank shell.

Appendix B.5.4 Base shears and moments

The shear force components, just above the tank base, arising from the horizontal inertia forces associated with the masses representing the hydrodynamic effects on the shell and the shell mass, are given by:

$$Q_1 = m_1 . S_a(T_1, \xi)$$

$$Q_r = m_r . S_a(T_0, \xi)$$

$$Q_f = (m_f + m_w + m_t) . S_a(T_f, \xi)$$

where:

T_1 is the period of the first convective mode of vibration;

T_0 is the period of the impulsive mode of vibration for the tank-foundation system ($T_0 = 0$ for rigid foundation);

T_f is the period of the first horizontal the tank-liquid mode including the effects of foundation flexibility ($T_f = T_0$ for a rigid tank);

$m_r = m_0 - m_f$

m_t is the roof mass.

m_w is the wall mass.

$S_a(T, \xi)$ is the spectral acceleration for period T and damping factor ξ.

The overturning moments, just above the tank base, arising from the horizontal inertia forces associated with the mass components, are given by:

$$M_1 = m_1.h_1.S_a(T_1,\xi)$$
$$M_r = (m_0.h_0 - m_f.h_f).S_a(T_0,\xi)$$
$$M_f = (m_f .h_f + m_w.h_w + m_t .h_t).S_a(T_f,\xi)$$

where:

h_w is the height to the center of gravity of the wall mass;

h_t is the height to the center of gravity of the roof mass.

Appendix B.5.5 Natural periods of vibration

The deformation of the soil foundation shall be included in the calculation of the periods of vibration of the impulsive modes for both the horizontal and vertical directions. For flexible tanks, this effect shall be included in the estimation of the periods associated with both the impulsive mode mases m_r and m_f.

The mass of the shell, base and any support system or structural foundation shall be included with the impulsive masses to calculate the impulsive modes periods.

The influence of soil deformation and tank wall flexibility may be neglected when calculating the periods of vibration of the convective modes.

Appendix B.5.6 Damping

The damping factor for convective modes in all types of tanks shall be taken as 0.5% [44]. A damping factor assessed by a consideration of material damping, foundation radiation and soil hysteretic damping shall be used for the impulsive modes of vibration.

Appendix B.5.7 Convective wave height

Unless more rigorous analysis is undertaken the maximum height of convective mode waves in the tank shall be estimated from spectral mode analysis involving at least the first two antisymmetric modes.

Appendix B.6 Seismic sizing, verification and qualification of Piping systems

Appendix B.6.1 Seismic behaviour of piping systems

Piping systems normally show a good behaviour under seismic loads, however, certain issues may appear with connecting elements (Figure B.6-1), long distances between supports (Figure B.6-2), soil liquefaction affecting piping support (Figure B.6-3) or support failure (Figure B.6-4).

Figure B.6-1 – Pipe elbow failure at the base of a water tank. Edgecumbe (New Zealand) 1987, M 6.3.

Figure B.6-2 – Displacement of a piping system. Izmit (Turkey) 1999, M 7.4.

Figure B.6-3 – Movement of supports due to soil liquefaction. Northrigde 1994, M 6.7.

Figure B.6-4 – Support break. Peru 2001 M 8.4.

Appendix B.6.2 Seismic verification of piping

Appendix B.6.2.1 Introduction

The evaluation and verification of piping systems under seismic loading conditions is given according to four procedures [52]:

- The first procedure allows the designer to avoid seismic verification for the piping system by evaluating the maximum acceleration the pipe can be subject to (see section Appendix B.6.2.3.1);
- The second verification proposes a methodology based on the distance between supports according to a table from which the seismic safety is assured (see sections Appendix B.6.2.3.2 and Appendix B.6.2.3.3);

- When the two above procedures cannot be followed, two additional methodologies are proposed to evaluate the stresses developed in the piping system and the criteria to be met.

These methodologies ensure that:

- longitudinal stresses in piping, forces and moments in supports and on the connected equipment (tanks, pumps, etc.) are acceptable;
- the forces and deformations of the non-permanent assemblies (bolted flange assemblies, screwed assemblies, etc.) are acceptable;
- displacements/deformations of the piping do not lead to shocks and/or interference with neighbouring equipment.

Appendix B.6.2.2 Calculation assumptions and special conditions

The recommendations given AFPS [52] cover ferrous and non-ferrous material subject to compliance with the following conditions, taken from the AFPS guideline for piping [52] where additional references for particular values can be found:

- The material used must have enough ductility at the operation temperature.
- For the exemption rules and the analytical calculations, the operating temperature must be ⩽ 110 ºC.
- For the response spectrum and for the equivalent static analysis, the operating temperature must be less than or equal to the creep temperature and the range of stress variation shall comply with the requirements of the technical reference system used for the evaluation under consideration.
- The only permanent assemblies covered by these recommendations are the welded ones.
- For the first three procedures, the piping is considered as decoupled from the secondary support structure. This assumption will be respected if the weight of the supported pipes is less than or equal to 25% of the sum of the weight of the supporting structure and the supported pipes. If this is not the case, the rules proposed in Chapter 3.4 of the AFPS Support Structures Guide [51] may justify decoupling. If decoupling cannot be justified, then the response spectrum and the equivalent static method procedure should be considered. The assumptions used to define these rules are conservative (linear elastic behaviour). More detailed analyses (nonlinear behaviour) can reduce this conservatism.
- In relation with the interaction between components of different diameters of the same piping, it is assumed that two connected components can be studied independently of each other if the ratio of their diameters is greater than or equal to three or if the ratio of their inertias is greater than 25.

Appendix B.6.2.3 Exemption of analysis

When a piping system satisfies any of the following two points, no further seismic analysis is required as per AFPS guidelines [52]. These calculations can be employed for the justification of NSC components and investment protection.

The intensity of seismic load is given by the parameter γ_S (see definitions below).

Appendix B.6.2.3.1 Case 1. Verification of the maximum acceleration to which the system is subjected

<u>Exemption of analysis on a given horizontal pipe.</u>

When the flexibility of the piping systems is ensured and the maximum acceleration on a horizontal axis pile fulfils the conditions described below (as per AFPS [52]) based on the performance criteria defined below, no further analysis is needed. Each case has an associated value of k (as in EN 13480-3:2012 [48]) that would be needed as a multiplying factor for deriving allowed stresses in case specific calculations are needed (section Appendix B.6.2.4).

- $k = 1.8$ when the only requirement is in-situ maintenance of the piping system under consideration or it is required that both, the piping support integrity and the confinement of the inside product are required, this is typically the situation for SL-2 earthquake (see section 12 in [48])

$$\gamma_s = \sqrt{(\gamma_h)^2 + (\gamma_v)^2} \leq 1.6\ g$$

- $k = 1.5$ when all three, in-situ maintenance of the piping, confinement of the product inside the piping and maintaining the functional capacities of the piping are required

$$\gamma_s = \sqrt{(\gamma_h)^2 + (\gamma_v)^2} \leq 1.0\ g$$

- $k = 1.3$ when retaining the piping, keeping the product handled inside the piping, maintaining the functional capacities of the piping in question and maintaining the operability of all or part of the operating elements of the piping are required

$$\gamma_s = \sqrt{(\gamma_h)^2 + (\gamma_v)^2} \leq 0.6\ g$$

where γ_h and γ_v are the maximum horizontal and vertical acceleration to which the section may be subjected.

<u>Exemption of analysis on a given vertical pipe.</u>

When the flexibility of the piping system is ensured and the maximum acceleration on a given pipe with vertical axis fulfils the conditions based on the performance criteria and associated values of k (see above descriptions), no further analysis is needed according to the following criteria:

- $k = 1.8$ $\quad\quad$ $\gamma_h \leq 1.1\ g$
- $k = 1.5$ $\quad\quad$ $\gamma_s = \sqrt{(\gamma_h)^2 + (\gamma_v)^2} \leq 0.7\ g$
- $k = 1.3$ $\quad\quad$ $\gamma_s = \sqrt{(\gamma_h)^2 + (\gamma_v)^2} \leq 0.4\ g$

Despite of the exemptions above mentioned, if a section is subjected to seismic loads generating differential displacements at its ends, it is necessary to evaluate their influence as defined in Section Appendix B.6.2.5.1.

As part of these exemptions, displacements, support reactions and stresses at the joints are considered to be sufficiently low that they do not need to be object of specific checks. In case of doubt, however, it is recommended for specific items that may be relevant to refer to section Appendix B.6.2.3.2 below.

Appendix B.6.2.3.2 Case 2. Estimation of the seismic response from verification of the maximum distance between supports

Sections meeting the conditions in section Appendix B.6.2.2 as well as those detailed below, and distance between supports is less than or equal to the values specified in Table B.6-1, Table B.6-2 and Table B.6-3 below, then this section is verified and does not need to be further analysed. This approach proposed by AFPS [52] can be employed to get an order of magnitude, in particular for an independent verification.

Distances between supports given in Table B.6-1 and Table B.6-2 were calculated according to the following conditions in AFPS [52] for three different seismic actions:

- Horizontal section
- Material: Steel, allowable stress f_f in MPa (see section Appendix B.6.2.5.1 for its definition):

$$f_f = \begin{cases} 176, & e \leq 16 \text{ mm} \\ 170, & 16 < e \leq 40 \text{ mm} \end{cases}$$

- Material density $\rho = 7860\ kg/m^3$

- Fluid mass density $\rho_{fluid} = 1000\ kg/m^3$
- Stress intensity factor $i = 1$ (defined in Annex H in EN 13480 part 3 [48])
- No supported equipment
- Earthquake:
  - EC-8 results in Table B.6-1
    - Horizontal acceleration: 0.48 g (maximum spectral acceleration for 5% damping EC-8 see section Appendix B.3.2.1)
    - Vertical acceleration: 0.32 g
  - SL-2 results in Table B.6-2
    - Horizontal acceleration: 0.74 g (maximum spectral acceleration for 5% damping SL-2, see section Appendix B.3.2.2)
    - Vertical acceleration: 0.49 g
- Performance requirements/seismic input
  - k = 1.3 for EC-8
  - k = 1.8 for SL-2
- Reduction coefficients: 1.5 for horizontal earthquake and 1.5 for vertical earthquake.

This procedure assumes that the horizontal straight lines of piping have a support in the axial direction, as long as the straight length is more than twice the distance given in the table below (from FEMA [66]), and that the vertical straight lines of the pipes are supported in the vertical direction.

The distances provided in the Tables are determined for each diameter for the normalized minimum and maximum thickness such that the ratio of outer diameter to inner diameter is approximately equal to 1.17.

For any other conditions as those established above, Table B.6-3 gives factors for obtaining new modified distances between supports (L*), deflections (X, Y) and reactions (H, V) from those given in Table B.6-1, Table B.6-2 and Table B.6-3. Furthermore, for one or more changes of direction (elbows) between two supports, the developed length shall be at most equal to the distance determined for a straight length divided by 1.5.

OD	Thickness	Distance between supports L_0	Deflection		Reactions		Pipe mass per unit length, m_{pipe}	Fluid mass per unit length, m_{fluid}
			Horizontal X_0	Vertical Y_0	Horizontal H_0	Vertical V_0		
mm	mm	m	mm	mm	kN	kN	kg/m	kg/m
26.9	2	5.5	35.72	98.23	0.021	0.058	1.22	0.41
26.9	5.6	5.75	43.76	120.35	0.042	0.116	2.92	0.19
33.7	2.2	6	33.07	90.95	0.034	0.092	1.7	0.67
33.7	7.1	6.5	45.66	125.56	0.075	0.207	4.63	0.3
42.4	2.6	6.5	29.20	80.29	0.055	0.152	2.54	1.09
42.4	8.8	7.25	44.49	122.35	0.132	0.363	7.25	0.48
48.3	2.6	6.75	27.09	74.49	0.069	0.191	2.91	1.46
48.3	10	7.75	44.74	123.04	0.183	0.502	9.39	0.63
60.3	2.9	7.25	23.95	65.85	0.109	0.301	4.08	2.33
60.3	12.5	8.75	46.66	128.31	0.322	0.885	14.6	0.98
76.1	2.9	7.5	18.75	51.56	0.160	0.441	5.2	3.88
76.1	16	9.75	45.31	124.60	0.576	1.583	23.6	1.53
88.9	3.2	7.75	16.04	44.11	0.220	0.605	6.72	5.35
88.9	17.5	10.5	44.02	121.04	0.813	2.236	30.6	2.28
114.2	3.6	8.25	13.24	36.41	0.364	1.001	9.76	8.99
114.2	22.2	12	45.39	124.81	1.522	4.184	50.1	3.83
139.7	4	8.5	10.44	28.71	0.539	1.481	13.3	13.6
139.7	28	13.25	45.37	124.77	2.562	7.044	76.6	5.5
168.3	4.5	9	9.36	25.74	0.805	2.213	18.1	19.9
168.3	32	14.5	44.36	121.98	3.939	10.832	107	8.54
219.1	6.3	10.75	10.84	29.80	1.679	4.616	31.8	31.3
219.1	40	16.75	46.25	127.20	7.520	20.679	168.7	13.7
273	6.3	10	5.87	16.14	2.222	6.112	41.2	53.3
273	55	18.5	45.20	124.30	13.701	37.679	293	20.9
323.9	7.1	10.25	4.74	13.03	3.147	8.654	55.1	75.3
323.9	65	20.25	46.06	126.66	21.061	57.918	412	29.5
355.8	8	11.25	5.62	15.44	4.206	11.568	68.2	90.7
355.8	70	21.25	46.09	126.75	26.348	72.457	490	36.6
406.4	8.8	11.5	4.80	13.21	5.534	15.219	85.7	118.7
406.4	80	22.75	46.42	127.64	36.814	101.239	639.9	47.7
457	10	12.25	4.86	13.37	7.482	20.577	110	150
457	90	24	45.47	125.04	49.123	135.088	809	60.3
508	11	12.75	4.64	12.77	9.587	26.364	133.4	186
508	100	25.5	46.89	128.95	64.475	177.307	1000	74.5
610	12.5	13.25	3.88	10.67	14.089	38.746	183	269
610	100	28	45.82	126.01	91.058	250.410	1250	132
711	25	22	16.45	45.23	39.529	108.704	420	343
711	100	30.5	46.75	128.55	122.199	336.048	1497	205

Table B.6-1 - Determination of the maximum distance between supports, EC-8 (γ_h = 0.48; γ_v = 0.32; q = 1.5).

OD	Thickness	Distance between supports L_0	Deflection		Reactions		Pipe mass per unit length, m_{pipe}	Fluid mass per unit length, m_{fluid}
			Horizontal X_0	Vertical Y_0	Horizontal H_0	Vertical V_0		
mm	mm	m	mm	mm	kN	kN	kg/m	kg/m
26.9	2	6	77.87	157.31	0.035	0.072	1.22	0.41
26.9	5.6	6.25	94.02	189.95	0.071	0.143	2.92	0.19
33.7	2.2	6.5	70.11	141.64	0.056	0.113	1.7	0.67
33.7	7.1	7	94.53	190.96	0.125	0.252	4.63	0.3
42.4	2.6	7.25	69.55	140.51	0.095	0.192	2.54	1.09
42.4	8.8	8	101.52	205.09	0.224	0.452	7.25	0.48
48.3	2.6	7.5	63.54	128.37	0.119	0.240	2.91	1.46
48.3	10	8.5	99.65	201.31	0.308	0.623	9.39	0.63
60.3	2.9	8.25	61.80	124.84	0.192	0.387	4.08	2.33
60.3	12.5	9.5	99.78	201.59	0.538	1.086	14.6	0.98
76.1	2.9	8.5	47.61	96.19	0.280	0.565	5.2	3.88
76.1	16	10.5	93.80	189.49	0.954	1.928	23.6	1.53
88.9	3.2	9	44.90	90.70	0.393	0.795	6.72	5.35
88.9	17.5	11.5	97.48	196.94	1.371	2.769	30.6	2.28
114.2	3.6	9.75	39.75	80.31	0.662	1.338	9.76	8.99
114.2	22.2	13	96.22	194.39	2.537	5.126	50.1	3.83
139.7	4	10.25	33.98	68.65	1.000	2.020	13.3	13.6
139.7	28	14.5	100.15	202.33	4.315	8.717	76.6	5.5
168.3	4.5	11	32.15	64.95	1.514	3.059	18.1	19.9
168.3	32	16	101.21	204.47	6.690	13.515	107	8.54
219.1	6.3	13	35.67	72.07	3.124	6.312	31.8	31.3
219.1	40	18.25	100.33	202.69	12.611	25.476	168.7	13.7
273	6.3	12.75	23.87	48.23	4.361	8.811	41.2	53.3
273	55	20.25	99.87	201.76	23.084	46.634	293	20.9
323.9	7.1	13.5	21.95	44.33	6.380	12.888	55.1	75.3
323.9	65	22	98.76	199.51	35.218	71.148	412	29.5
355.8	8	14.25	22.25	44.96	8.201	16.568	68.2	90.7
355.8	70	23	97.36	196.69	43.894	88.674	490	36.6
406.4	8.8	15	21.40	43.23	11.110	22.445	85.7	118.7
406.4	80	24.75	100.08	202.18	61.644	124.535	639.9	47.7
457	10	16	21.77	43.99	15.042	30.388	110	150
457	90	26.25	100.15	202.33	82.697	167.065	809	60.3
508	11	16.75	21.29	43.02	19.385	39.162	133.4	186
508	100	27.75	101.22	204.48	107.994	218.171	1000	74.5
610	12.5	17.5	18.17	36.71	28.642	57.863	183	269
610	100	30.75	102.59	207.26	153.919	310.949	1250	132
711	25	25.5	45.69	92.31	70.520	142.466	420	343
711	100	33.25	101.63	205.31	205.044	414.232	1497	205

Table B.6-2 - Table for the determination of the maximum distance between supports, SL-2 (γ_h = 0.74; γ_v = 0.49; q = 1.5).

Modified parameter	Modified distance between supports, L^* m	Modified Deflections, X*, Y*		Modified Reactions, H*, V*	
		Horizontal mm	Vertical mm	Horizontal kN	Vertical kN
Accelerations	$L_0\left(\frac{(a_{0,h}^2 + a_{0,v}^2)^{0.5}}{(a_{*,h}^2 + a_{*,v}^2)^{0.5}}\right)^{0.5}$	$\left(\frac{L^*}{L_0}\right)^4 X_0$	$\left(\frac{L^*}{L_0}\right)^4 Y_0$	$\left(\frac{L^*}{L_0}\right) H_0$	$\left(\frac{L^*}{L_0}\right) V_0$
Allowable stress	$L_0\left(\frac{f_f}{170\ MPa}\right)^{0.5}$	$\left(\frac{L^*}{L_0}\right)^4 X_0$	$\left(\frac{L^*}{L_0}\right)^4 Y_0$	$\left(\frac{L^*}{L_0}\right) H_0$	$\left(\frac{L^*}{L_0}\right) V_0$
Density	$L_0\left(\frac{\mathrm{m}_{0\mathrm{pipe}} + \mathrm{m}_0^{\mathrm{fluid}}}{m_{\mathrm{pipe}}^* + \mathrm{m}_0^{\mathrm{fluid}}}\right)^{0.5}$	$\left(\frac{L^*}{L_0}\right)^4 X_0$	$\left(\frac{L^*}{L_0}\right)^4 Y_0$	$\left(\frac{L^*}{L_0}\right) H_0$	$\left(\frac{L^*}{L_0}\right) V_0$
Stress intensity factor	$L_0\left(\frac{1}{\mathrm{i}}\right)^{0.5}$	$\left(\frac{L^*}{L_0}\right)^4 X_0$	$\left(\frac{L^*}{L_0}\right)^4 Y_0$	$\left(\frac{L^*}{L_0}\right) H_0$	$\left(\frac{L^*}{L_0}\right) V_0$
Fluid mass density	$L_0\left(\frac{\mathrm{m}_{0\mathrm{pipe}} + \mathrm{m}_0^{\mathrm{fluidd}}}{\mathrm{m}_{0\mathrm{pipe}} + \mathrm{m}_{\mathrm{fluid}}^*}\right)^{0.5}$	$\left(\frac{L^*}{L_0}\right)^4 X_0$	$\left(\frac{L^*}{L_0}\right)^4 Y_0$	$\left(\frac{L^*}{L_0}\right) H_0$	$\left(\frac{L^*}{L_0}\right) V_0$
Concentric mass $\mathrm{M_m}$	$L_0\left(\frac{\mathrm{M_m}}{\mathrm{m_{pipe}} + \mathrm{m_{fluid}}}\right)$				
If part of the mass $\mathrm{M_m}$ is eccentric. Mass $\mathrm{M_{m,ecc}}$ at distance d_{ex}	$\left(\frac{(L_0)^2 - 4(\mathrm{M_{m,ecc}} d_x)}{\mathrm{m_{pipe}} + \mathrm{m_{fluid}}}\right)^{0.5}$				
k=1.5		$\frac{5}{3} X_0$	$\frac{5}{3} Y_0$	$\frac{5}{3} H_0$	$\frac{5}{3} V_0$
k=1.8		$\frac{8}{3} X_0$	$\frac{8}{3} Y_0$	$\frac{8}{3} H_0$	$\frac{8}{3} V_0$
Vertical axis pipes	$0.9 L_0$	$0.75\ X_0$	$0.75\ Y_0$	$1.05\ H_0$	$1.05\ V_0$

Table B.6-3 - Distances, deflections and reactions as a function of modified parameters.

Appendix B.6.2.3.3 Determination of the maximum distance between supports

The values of the maximum distances between supports defined in Table B.6-1 were determined using relatively conservative assumptions.

However, when these distances cannot be respected, it is possible to reduce the conservatism of these requirements by doing a specific calculation using the analytical methodologies described in section Appendix B.6.2.4.

Appendix B.6.2.4 Seismic evaluation methods

Appendix B.6.2.4.1 Determination of the seismic response from equivalent static analysis

The determination of seismic response on the piping system by equivalent analysis must be applied as defined in section Appendix B.2.3. The results obtained from the equivalent static analysis must satisfy the requirements specified in section Appendix B.6.2.1.

In order to provide tools for the designer, Figure B.6-5 presents equations for the determination of the first frequency of different supported systems. Knowing the first frequency, the acceleration to be considered can be easily obtained from the corresponding ground response spectra or FRS.

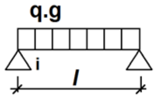
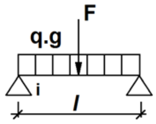
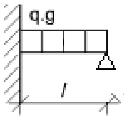
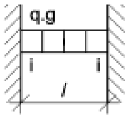
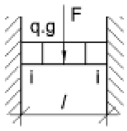
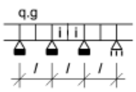
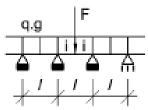
Case	System	Load	First frequency
A		q [kg/m]	$f_{1,A} = \frac{9.87}{2\pi}\sqrt{\frac{EIg}{ql^4}}$
B		q [kg/m] F [kg]	$f_{1,B} = \frac{6.93}{2\pi}\sqrt{\frac{EIg}{Fl^3 + 0.486ql^4}}$
C		q [kg/m]	$f_{1,C} = \frac{15.4}{2\pi}\sqrt{\frac{EIg}{ql^4}}$
D		q [kg/m]	$f_{1,D} = \frac{22.4}{2\pi}\sqrt{\frac{EIg}{ql^4}}$
E		q [kg/m] F [kg]	$f_{1,E} = \frac{13.86}{2\pi}\sqrt{\frac{EIg}{Fl^3 + 0.383ql^4}}$
F		q [kg/m]	$f_{1,F} = \frac{6.93}{2\pi}\sqrt{\frac{EIg}{ql^4}}$
G		q [kg/m] F [kg]	$f_{1,G} = \frac{6.93}{2\pi}\sqrt{\frac{EIg}{Fl^3 + 0.486ql^4}}$

Figure B.6-5 – Approximation for the first frequency of different beam structures [78].

Appendix B.6.2.4.2 Determination of seismic response from response spectrum analysis

The determination of seismic response on the piping system from a response spectrum analysis must be applied as defined in section Appendix B.2.5.1. The results obtained from the response spectrum analysis must satisfy the requirements specified in section Appendix B.6.2.1. In particular, the coupling or decoupling of the dynamic behaviour between the piping system and the supporting structure has to be studied and the modelling in agreement with its conclusions. The flexibility of the piping system shall be well predicted by the numerical model considering the flexibility of the supports and the elbows.

Appendix B.6.2.5 Flexibility analysis

Appendix B.6.2.5.1 General conditions

When a piping system is subjected to a seismic load event the stresses determined by methods described above must meet the following requirements.

The primary stresses due to internal pressure and the resultant moment from weight and other permanent mechanical loads must satisfy the following equation:

$$\sigma_1 = \frac{p\,D_o}{4\,e_f} + \frac{0.75\,i\,M_A}{Z} < f_f$$

where:

p is the internal pressure;

D_o is the external diameter of the pipe;

e_f is the thickness of the pipe;

i is the stress intensification factor;

M_A is the resultant moment from permanent sustained mechanical loads obtain for the most unfavourable combination;

Z is the elastic section modulus

$f_f = \min\left(f; f_{cr}\right)$ is the design stress for flexibility analysis as defined in EN 13480 part 3 [48]. For time independent not austenitic steels f is defined as (For other types of steels see clause 5 in EN 13480 part 3 [48]:

$$f = \min\left(\frac{R_{eHt}}{1.5}\text{ or}\frac{R_{p0,2t}}{1,5};\frac{R_m}{2.4}\right)$$

$f_{cr} = \frac{S_{RTt}}{Sf_{cr}}$ is the design stress in the creep range;

where:

R_{eHt} is the minimum specified value of upper yield strength at calculation temperature when this temperature is greater than the room temperature;

$R_{p0,2t}$ is the minimum 0,2 % proof strength at temperature of pipe;

R_m is the tensile strength;

S_{RTt} is the mean value of creep rupture strength as indicated by the standards, for the material in question at the considered temperature, t, and for the considered lifetime T (in hours) whereby the dispersion band of the results does not deviate by more than 20% from the mean value;

Sf_{cr} is a safety factor which depends on the time and shall be in accordance with Table 5.3.2-1. in EN 13480 part 3 [48].

The primary stresses due to internal pressure, bending moment due to permanent mechanical loads and bending moment due to occasional loads must satisfy the following equation:

$$\sigma_2 = \frac{p\,D_o}{4\,e_f} + \frac{0.75\,i\,M_A}{Z} + \frac{0.75\,i\,M_b}{Z} < kf_f$$

where:

M_b is the bending moment produced by occasional loads;

$k = 1.8$ when only the retention of the piping in question is required or where the holding of the piping in question and the containment of the product handled inside the piping are required;

$k = 1.5$ when retaining the piping, keeping the product handled inside the piping and maintaining the functional capacities of the piping in question are required;

$k = 1.3$ when retaining the piping, keeping the product handled inside the piping, maintaining the functional capacities of the piping in question and maintaining the operability of all or part of the operating elements of the piping are required;

Appendix B.6.2.5.2 Relative displacements between supports

When the supports of a section are subjected to relative displacements, the effect of the latter can be verified on the basis of the following formula [47]:

$$M_D = \frac{6\Delta_{max} E\, I}{l_{\text{supports}}^2}$$

where:

- $k = 1.3,\ \sigma = \frac{i\, M_D}{Z} \leq \text{Min}\ (3f,\ 2R_{p0,2t})$
- $k = 1.5,\ \sigma = \frac{i\, M_D}{Z} \leq \text{Min}\ (4.5f,\ 3R_{p0,2t})$
- $k = 1.8,\ \sigma = \frac{i\, M_D}{Z} \leq \text{Min}\ (6f,\ 4R_{p0,2t})$
- Δ_{max} is the relative displacement between the two supports considered;
- E is the Young's Modulus;
- I is the moment of inertia of the section considered;
- $l_{supports}$ is the distance between supports.

Appendix B.6.2.6 Seismic verification for inelastic piping systems (Hinge method)

Piping is ductile and can accommodate inelastic strains under cyclic conditions. The inelastic energy absorption capacity of a piping system causes the actual response to the seismic event at higher load levels to be significantly below the response predicted by elastic calculation methods. For these reasons, EPRI has developed a simplified inelastic analysis method [65]. The basic idea is to use, at an acceptable cost, an incremental hinge solution to address multi-degree-of-freedom (non-uniform) plasticity and to consider the plastic system deformations predicted by this solution to define the effective load reduction due to inelastic energy dissipation. A brief overview of the method is provided in Figure B.6-6.

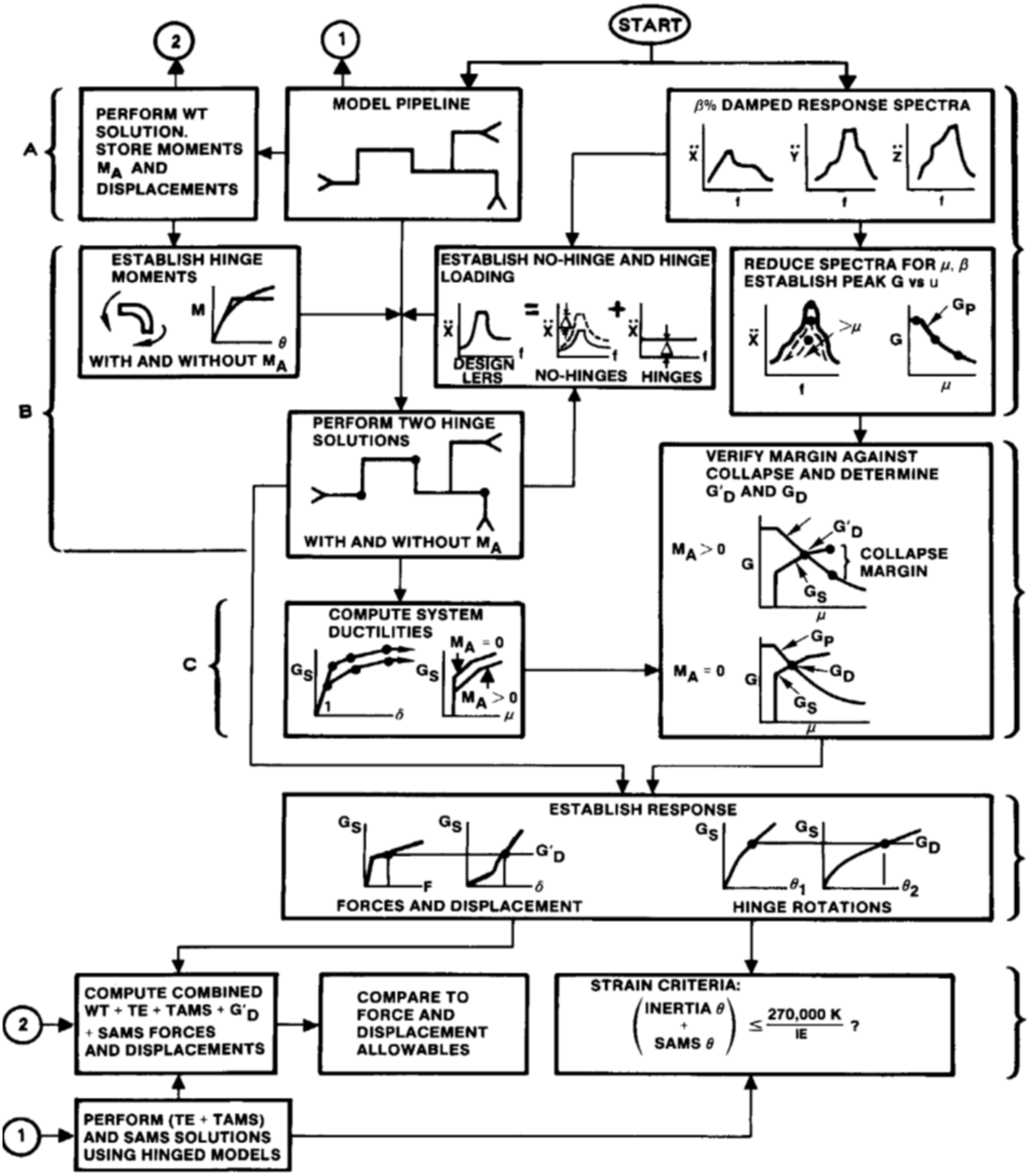


Figure B.6-6 – Flow chart of the Incremental Hinge Method (from [65]).

Appendix B.6.3 Seismic verification for active mechanical equipment

The verification for active mechanical equipment installed on piping systems here developed is based on AFPS Guideline [52] and FEMA guide [71]. Due to the wide variety of equipment that can be installed on pipe lines it is difficult to establish general rules covering all possible situations, however depending on the criteria to be met some calculations may need to be done, according to AFPS [52].

In FEMA guide [71], more specific considerations to different mechanical equipment can be found.

Appendix B.6.3.1 Only the stability of the piping under consideration is required.

The main issue to solve when verifying the stability of the piping is the mass of the equipment installed and the possible effects of an eccentric part of the mass. These possible effects must be considered when the flexibility of the piping is checked.

No further analysis of stresses in the equipment is normally required due to its robustness. However, for some components, especially when they have eccentric parts, it is necessary to check that the stresses present in the material remain acceptable. For these calculations, the allowable stresses can be taken up to a maximum of 1.8 f_f.

Appendix B.6.3.2 When the integrity of the piping and the confinement of the product inside the piping under consideration is required

In these conditions, in addition of the requirements above mentioned, non-permanent assemblies should be checked for leakage of the product inside the piping.

Appendix B.6.3.3 When retaining the piping, confinement of the product inside the piping and maintaining the functional capacities of the piping in question are required

The same requirements as the two above need to be fulfilled. When calculations are needed, the allowable stresses can be taken up to a maximum of 1.5 f_f.

Appendix B.6.3.4 When the stability of the piping, confinement of the product inside the piping, maintaining the functional capacities and the operability of all or part of the piping operating members are required

The requirements of the preceding paragraphs shall be accomplished; where calculations are necessary, the allowable stress can be taken up to a maximum of 1.3 f_f.

Appendix B.6.3.5 Valves and valve operators

The manual valve is considered as concentrated load, in-line with the pipe span in the stress analysis of piping according to section Appendix B.6.2.5.

The need to include the operator valve in the stress analysis of the piping system depends on the operator weight. The FEMA guide [71] recommends including the operator as an eccentric mass only if its weight is 10% more than the weight of the valve. Specific considerations for heavy valve operators can be found in [71].

Appendix B.7 Justification of Conceptual Model

The physical principles adopted in the conceptual model generally idealize and hence simplify the physical reality. Such simplifications include, amongst others:

- The application of fixed displacement constraints. Here the stiffness of the surrounding structure is assumed to be infinite. This includes displacements imposed on the boundaries of sub models.

- Parts or the entire model are assumed to have linear or constant material properties, e.g. those parts are assumed not to yield or their properties are assumed not to be dependent on temperature.
- When beam or shell theory is used it is typically assumed that that the cross section remains plane. Deformation due to shear may or may not be taken into account.
- When hinges are assumed, the resisting moment at the hinge is often assumed to be negligible.
- When a structure is assumed to act as a truss, the elements in the truss are usually assumed to transfer axial forces but no bending moments.
- The application of loads in an idealised manner, e.g. point loads or uniform pressure distributions.
- The manner in which structural damping is considered.

In particular the most important simplifications that the chosen model contains shall be listed. Examples include:

- Simplifying assumptions related to contact, e.g. uniform friction coefficient, thermal contact conductance independent of contact pressure, corrected contact geometry.
- Neglected nonlinearities, e.g. small gaps, damping, the application of the load to the initial un-deformed structure, linear isotropic material properties.
- Neglected dynamic amplification.
- Neglected geometrical imperfections and tolerances (e.g. use of nominal dimensions).
- Neglected thermal expansion.
- Neglected or simplified loads, e.g. radiation heat, dead weight or coolant or hydrostatic pressure.
- Simplified mass distributions, for example by assuming that mass is located at a single point.
- Structural elements of the model assumed to have infinite stiffness, e.g. when linking the degrees of freedom of two non-coincident nodes with constraint equations.
- Geometric simplifications, e.g. neglected holes, chamfers, fillets.

Justification shall be given for each simplification or assumption. Short, logical arguments are often sufficient to meet this requirement, but sensitivity studies may be necessary in some cases. Examples of justification could include:

- Non-consideration of fillets: The effect of the fillets is only to reduce peak stresses, which are not within the scope of this analysis. Their contribution to the global stiffness of the structure is negligible and beneficial.
- Consideration of a part A as a point mass element: The single mass element representing part A has the same influence on the intended results of this analysis as a detailed model of part A would have since the intended results are not from within part A or near the interface between part A and the rest of the system.

Appendix B.8 Mesh Discretisation

With the exception of stress and strain at geometric discontinuities, results of FE analyses usually tend toward the correct value as mesh density increases. To verify that the mesh density is sufficient it is suggested to obtain results for several different mesh densities. Based on such sensitivity analyses it is usually possible to determine what uncertainty is associated with mesh discretisation. One method for doing this is Richard extrapolation, with ASME V&V 10.1 giving a good example, see [13].

Beyond that, this section also contains some guidance for minimum mesh densities and related uncertainty factors. The given recommendations are based on the experience with ANSYS FE models, but other FE codes will tend to require similar mesh densities.

Note that the verification of the mesh density cannot be avoided even if the recommendations given in this section are followed. It remains the responsibility of the performer to verify the FE model. Such benchmarking always requires the generation of multiple meshes with different mesh densities. Different uncertainty factors than those given here may be applied if justified by an appropriate benchmarking of the mesh density, which shall be reported in the analysis report.

Appendix B.9 Analysis Verification and Accuracy of Results

Having selected and justified the conceptual model and analysis method, the accuracy of the results obtained with an FE model shall be verified by the performer of the analysis. Generally speaking, determining whether or not the FE model accurately represents the conceptual model is done in two ways:

- Eliminating characteristics of the FE model that could cause imprecision or errors in the results.
- Comparing obtained results from the FE analysis with those from alternate calculations.

The purpose of the verification of the FE model is two-fold. The first is to ensure that there are no errors in the analysis. In this sense, errors are deficiencies in any phase or activity of modelling or experimentation that is not due to lack of knowledge.

The second purpose of the verification process is the quantification of the maximum expected uncertainty associated with the different types of results that are intended to be obtained with the FE model and, if applicable, the different types of loads.

It is common practice to apply an uncertainty factor to the obtained results to account for possible uncertainties in the input data or the conceptual model. The accuracy requirement for the FE model depends on the intended use and usually requires engineering judgement. A general rule is that the required accuracy is the minimum accuracy for which meaningful conservative results can be obtained through the application of a defined uncertainty factor.

As stated in Appendix A.4.12, the result values given in the report shall consider the uncertainty of the analysis. It may be that the accuracy goal is met only partially, e.g. for certain types of loads or for certain types of results only.

Errors and uncertainties in FE results can occur due to a number of causes and it is the duty of the performer to prevent errors and keep uncertainty to an acceptable level. Since it is often difficult to exclude all possible sources of errors and quantify all uncertainties in an FE analysis, results from FE analyses shall be verified by comparing them with results from an alternate calculation.

The purpose of these requirements is to support the performer to produce reliable results by excluding certain well-known sources of errors in FE analyses. Individual checks may be dropped if the consequent effects on the analysis results are quantified and considered in the analysis conclusions. Engineering judgement combined with the results of the verification checks can be used to determine the uncertainty in an analysis.

Note that compliance with the verification requirements does not release the performer from his/her responsibility to report correct and reliable results.

Appendix B.10 Element Types and Shapes

One cause of wrong results is use of inappropriate element types or element shapes. This section gives some guidance for common pit-falls to avoid.

- Using shell and beam elements often means that the effective length of structural elements is overestimated. For example, in a shell model of a building, the length of a slab between two walls would typically be measured from mid-plane to mid-plane. This would overestimate the length of the span by the width of the wall. Beam and shell

elements are therefore most appropriate in slender structure. A good rule of thumb is that structures with slenderness ratios over 20 should be modelled using shells or beams.
- When using shell and beam elements the performer should be aware of whether or not shear deformation is likely to be important, and whether or not the element formulation includes shear deformation. Shear deformation can usually be ignored in slender structures.
- Many FE codes struggle to give accurate results when using linear (i.e. without mid-side nodes) degenerate (e.g. triangular or tetrahedral) elements.
- In ANSYS, the shell element type SHELL181 typically needs a very fine mesh to converge to the correct solution for bending stress. However, this will become apparent during a mesh-sensitivity study. SHELL281 has been found to require a much smaller number of nodes to reach the converged solution. A study highlighting this problem can be found [81].
- In solid models, parabolic tetrahedra, parabolic hexahedra and linear hexahedra are roughly equivalent terms of the solution accuracy that can be achieved for a given number of DoFs. Linear tetrahedra tend to perform less well.

Appendix B.11 Boundary Conditions

The choice of BCs is linked to the conceptual model and its justification. The following points are a few examples of potential issues with BCs:

- The use of rigid links between nodes (in ANSYS e.g. CP or CERIG) that are based on degrees of freedom whose direction does not follow the deformation of the structure is generally not appropriate in large-deformation analyses. If such links are used in large-deformation analyses, their use shall be justified.
- Unless there is a specific requirement to have free rigid body motion it should be verified that all parts of the structure are globally constrained in all degrees of freedom. Problems with under-constrained models are usually spotted during the load checks described in Appendix A.2.5.3 and Appendix A.2.5.4.
- Inappropriate BCs are a common source of unrealistic thermal stresses. Therefore, when assessing thermal stresses in a local model, care has to be taken to avoid restraining displacements in all directions if a structure is in reality free to expand.

Appendix B.12 Solution Settings

Solution settings can have significant effect on analysis results. It is therefore important for the performer to understand these effects and control their impact on the analysis results. This section highlights some common issues:

- Many FE software packages offer several different solver types (e.g. sparse, preconditioned conjugate gradient) for both shared memory and distributed memory solvers. These vary in robustness and solution speed. The performer needs to be able to determine which solver is most applicable for a particular task.
- The solution settings cover whether or not non-linear geometry is assumed.
- In transient analyses, the following settings are important:
  - The size of the time step shall be verified. A good rule of thumb for implicit solvers is that the time step shall not be longer than 1/20th of the period of the highest natural frequency important to the results of the analysis.
  - Whether loads are ramped or stepped.
  - Whether the analysis is fully transient or mode-superposition transient.
- In non-linear analyses, results are often sensitive to the convergence criteria. In ANSYS, this is especially true when friction is present. In this case it is recommended to run analyses with two different sets of convergence criteria (e.g. CNVTOL) to verify that the convergence criteria are suitable.

Appendix B.13 Contacts

It is not uncommon for incorrect contact properties (standard, rough, frictionless, etc.) to be set by mistake, especially when multiple contact surfaces with different properties are specified. In addition, it is quite common for contact pairs to 'miss' each other due to inappropriate pinball settings. To minimise the probability of these errors, it is recommended to:

- Check the contact status (open, closed, sliding, etc.) of each contact pair at significant steps throughout the analysis.
- Check that the contact stiffness is reasonable. It should be borne in mind that the default contact stiffness values chosen by FE software are often not appropriate. Whilst it is relatively simple to determine what the contact stiffness is in an FE model, it is often difficult to know whether or not it is appropriate without real-world tests. If contact stiffness has a significant impact on the result of the FE analysis it may be necessary to either perform a sensitivity study or experimental validation. In cases where the analysis is used to calculate stresses due to imposed displacements (e.g. thermal) it is often conservative to minimise contact penetration by maximising the contact stiffness.
- Check that the shear stress in a contact pair is consistent with the normal stress and the coefficient of friction.

Appendix B.14 Damping

Choosing an appropriate value of damping is very important in dynamic analyses (response spectrum, time-history of PSD analysis). Suitable damping values can vary depending on the type of component, the materials used and the severity of the load. The damping values for most ITER systems are defined in the ITER Load Specifications [17].

Special attention shall be required for components made by several materials with different values of damping ratio. The use of composite modal damping is possible.

In case Rayleigh damping is used, the chosen values of α and β can be justified by comparing the desired damping ratio with the effective damping ratio for the significant modes of the structure. This is often best achieved in a tabular format. Using plots similar to the one below can also be very helpful, perhaps adding in the main modes and their effective modal mass. The values of the frequencies ω_1 and ω_2 should be given. These are the frequencies for which the effective damping ratio is equal to the desired one.

If α and β are used to simulate a constant damping ratio, care must be taken to avoid overdamping significant modes.

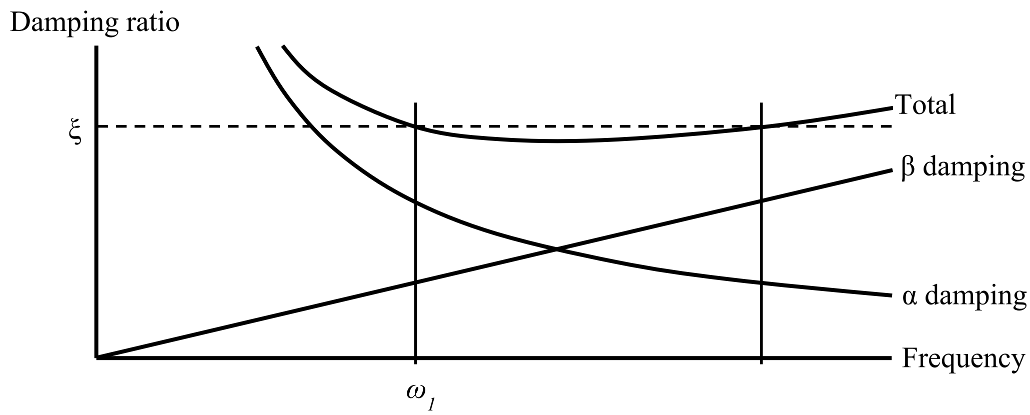


Figure B.14-1 – Rayleigh damping.

The most appropriate manner of checking that the intended damping has been applied correctly varies depending on the nature of the model and the type of analysis. However, looking at the response of a model to a simple load is often helpful (transient or harmonic load). For transient analysis, one example of this is the decaying response to an applied step function of acceleration. Looking at relevant transfer functions may also give a good indication that damping has been applied correctly.

Appendix B.15 Unintended Internal Stiffness Check

The purpose of this check is to identify any unintended rigid links between nodes. The check is conducted by applying a uniform temperature of 100°C to all parts of the model. Uniform material properties are applied to the whole model, with Poisson's ratio of 0.3 and a thermal expansion coefficient larger than zero. All support constraints are removed, except for one single node which is constrained in all degrees of freedoms. It is recommended to check that all resulting stresses are negligible.

Appendix B.16 Use of Figures and Plots

Figures can be a very useful way to communicate. However, it is worthwhile for the performer to be conscious of what the purpose of each figure is, and then satisfy him or herself that the relevant figure serves the intended purpose. The following suggestions may be helpful:

- If the report compares results from different contour plots, it is usually easier for the reader if the same contour definitions are used in all of the plots.
- Text in the plot should have an appropriate size.
- The resolution/compression should be appropriate. For example, if the purpose of the plot is to show the details of a mesh, the mesh should be clearly visible in the plot.
- It is usually easier for the reader to understand plots of mesh or geometry if the image is orientated such that the vertical axis is pointing upwards.

Appendix B.17 Internal References

If the report is written using Microsoft Word it is strongly recommended that the in-built cross-reference function is used for references to Tables, Figures, Sections, References, etc. This makes it less likely that internal references become incorrect as the report is written and updated. It also allows readers to navigate the report more easily.

For references to external documents, i.e. like those made in Chapter 4, several suitable features are available in MS Word. It is recommended to make use of one following:

- Numbered list.
- Bookmarks.
- Bibliography.

Appendix B.18 Mesh reporting

Some examples are given below of how to report details of an FE mesh, for some of the most common software packages within IO.

Appendix B.18.1 FE MESH (for ANSYS APDL)

Appendix B.18.1.1 OVERVIEW

A description of the FE model is given in this section, along with pictures of the FE mesh. Several figures may be necessary to show all relevant details. An example overview table of the FE mesh is shown in Table B.18-1.

The summary of the shape check performed on the FE model shall be reported here.

Parts	Element Properties in ANSYS					Number of elements
	ETYPE	ENAME	REAL	MAT	SEC	
Inner cylinder of top lid	85	SHELL63	809	1	1	720
Base section cylinder	88	SHELL181	1	88	88	576
UCTS inner mass	1022	MASS21	1022	1	1	1

Table B.18-1 – Overview of the element types and attributes used in the analysis (APDL example).

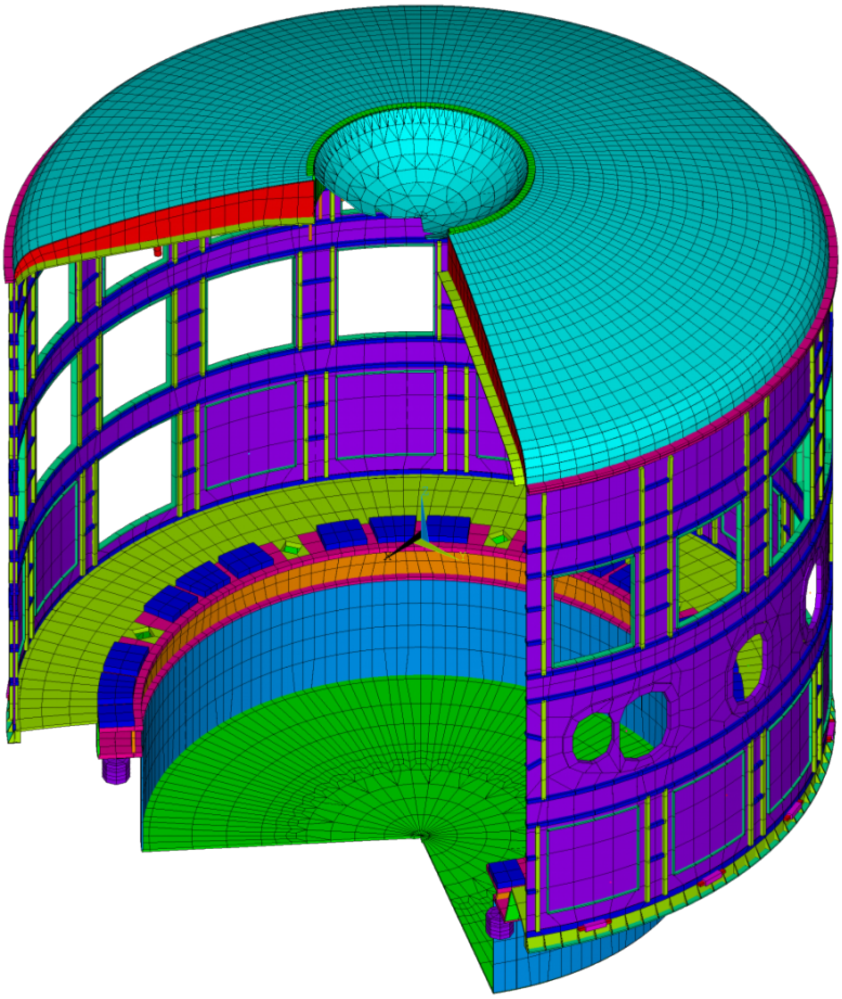


Figure B.18-1 – Cut-away view of the mesh of the simplified Cryostat model.

Appendix B.18.1.2 REAL CONSTANTS

A suitable tabular format should be chosen to present relevant real constant parameters. Two examples are shown below.

Parts	Real number	Real 1	Real 2
		OD (m)	TKWALL (m)
Outer UCTS attachments	905	0.273	0.0254
Inner UCTS attachments and truss for outer UCTS attachments	909	0.114	0.0112

Table B.18-2 - Real constants for elements of type PIPE16. Non-listed REALs are left at their default values (APDL example).

Parts	Real number	Real 1 MASSX (m)	Real 2 MASSY (m)	Real 3 MASSZ (m)	Real 4 IXX (kgm^2)	Real 5 IYY (kgm^2)	Real 6 IZZ (kgm^2)
VVGS master node	120	0	0	0	0	0	0
UCTS inner mass	122	21903	21903	21903	452330	421175	828274

Table B.18-3 - Real constants for elements of type MASS21 (APDL example).

Appendix B.18.1.3 SECTION PROPERTIES

A suitable tabular format should be chosen to present relevant section parameters. An example is shown below.

Parts	Section number	Width (m)	Height (m)
Toroidal ribs in lower cylinder	1017	0.244	0.021
Vertical ribs in lower cylinder	1021	0.021	0.601

Table B.18-4 – Sections used for elements of type BEAM188, SUBTYPE = RECT. The number of cells along the width and height is set to the default value of 2. The section's centroid is offset to the beam node locations (default) (APDL example).

Appendix B.18.1.4 FINITE ELEMENT OPTIONS OR KEYOPTIONS

The element options or keyoptions shall be listed for every element type. This information can be listed in the 'Overview' table described in Paragraph Appendix B.18.1.1 if desired. An example is shown below.

Use of Elements	ETYPE	ENAME	KEYOPT(3)	KEYOPT(8)
Plate	53	SHELL181	2 - Full integration with incompatible modes.	2 - Store data for TOP, BOTTOM, and MID.

Table B.18-5 – Keyoptions of SHELL181 elements. Non-listed keyoptions are left at their default values (APDL example).

Appendix B.18.1.5 CONTACTS

This paragraph is mandatory if contact elements are used in the FE model.
For every contact pair used in the analysis, the two associated parts of the FE model should be defined in the analysis report. The settings of each contact pair (keyoptions and real constants of contact element) should be listed here. It is recommended to show a figure of each contact pair, highlighting the geometry that is part of the contact. Examples of all this are shown below.

Use of Elements	Real number	Target elements		Contact elements	
		ENAME	ETYPE	ENAME	ETYPE
Contact between boss and insulation ring	58	TARGE170	59	CONTA174	58
Contact between boss and bolt	62	TARGE170	63	CONTA174	62

Table B.18-6 – Contact pairs used in the model (APDL example).

Use of Elements	Real number	Real 3	Real 6
		FKN	PINB
Contact between boss and insulation ring	58	10	-0.10E-02
Contact between boss and bolt	62	$-1.34 \cdot 10^{12}$	-0.10E-02

Table B.18-7 - Real constants for contact elements of type TARGE170 and CONTA174. Non-listed REALs are left at their default values (APDL example).

Contact element options		CONTA174 element type	
KEYOPT	Description	58	62
1	Element degrees of freedom	UX/UY/UZ	UX/UY/UZ
2	Contact algorithm	Augmented method	MPC algorithm
3	Contact stiffness units	F/(L^3)	F/(L^3)
4	Contact detection	Node: Normal from contact	Node: Normal from contact
5	Auto CNOF/ICONT adjustment	No auto. Adjustment	Close gap
6	Auto contact stiffness change	Standard	Standard
7	Contact time/load prediction	No predictions	No predictions
8	Asymmetric contact selection	No	No
9	Initial penetration/gap	Exclude	Exclude
10	Contacting stiffness update	Each iteration	Each iteration
11	Shell thickness effect	Include	Include
12	Behaviour of contact surface	Standard	Bonded (always)
15	Effect of contact stabilization damping	Active in 1^{st} load step	Active in 1^{st} load step
18	Sliding behaviour	Small sliding	Small sliding

Table B.18-8 – Settings for contact element CONTA174. Non-listed keyoptions are left at their default values (APDL example).

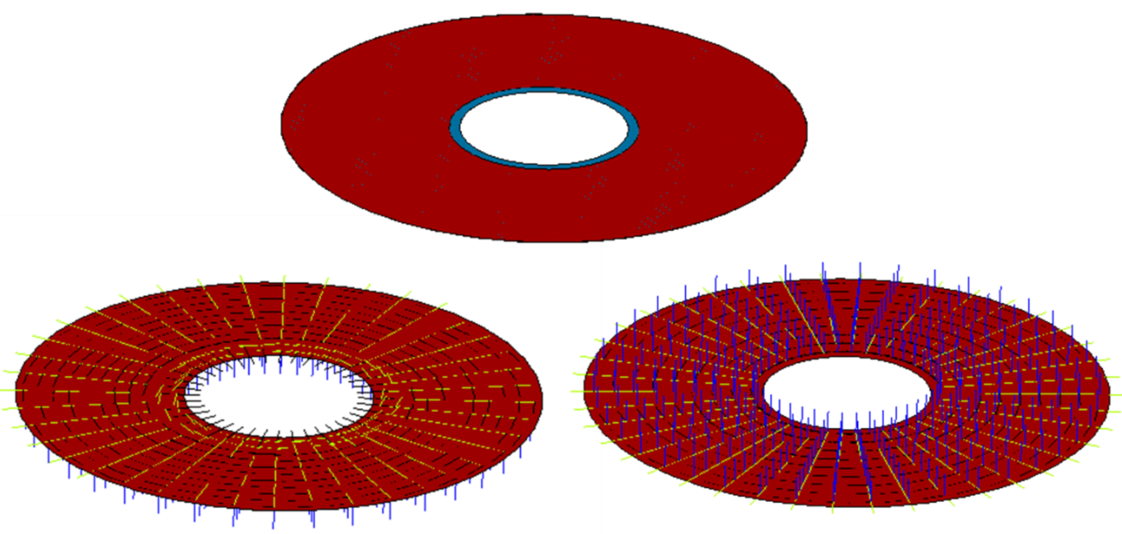


Figure B.18-2 – Contact pair (top) with real constant 58 between boss and insulation ring. Standalone view of CONTA174 elements (bottom left) and TARGE170 (bottom right).

Appendix B.18.2 FE MESH (for ANSYS Workbench)

Appendix B.18.2.1 OVERVIEW

A description of the FE model is given in this section, along with pictures of the FE mesh. Several figures may be necessary to show all relevant details. An example overview table of the FE mesh is shown in Table B.18-9. Parts shall be clearly and unambiguously identifiable, especially if the names in Workbench do not match the ones presented in the geometry section of the report.

The summary of the shape check performed on the FE model shall be reported here. An example is shown in Figure B.18-4.

Parts	Element Name	Number of elements
Contacts	TARGE170	4930
Contacts	CONTA174	2456
Plate	SHELL181	10060
Bosses, bolts and insulation ring	SOLID186	15582

Table B.18-9 – Overview of the element types used in the analysis (Workbench example).

Notes:

- Starting from version 19.0 of ANSYS Workbench, the number of elements of each type can be obtained under *Material and Element Type Information* option in the Solution Summary Worksheet. This can be displayed by either selecting the *Worksheet* button on the *Home* tab or by right-clicking on the *Solution* object and selecting the option *Worksheet: Result Summary*.
- The element name used for each body (either solid or shell/beam) can be displayed by plotting the *User Defined Result* defined by the expression *PNUMENAM*. An example of this plot is shown in Figure B.18-3.
- A mesh check can be performed by plotting one or more element quality metrics:
  - o Click the *Mesh* object in the Tree Outline.
  - o In the Details View, expand the *Quality* folder.
  - o For the *Mesh Metric* control, select the metric of interest from the drop-down menu. One example is shown in Figure B.18-4.

A global picture of the mesh metrics can also be obtained by changing the *Display Style* of the *Mesh* object to the desired mesh metric (e.g. *Element Quality*). Particular attention should however be paid to the mesh quality inside solid elements since they are not visible from the exterior. Figures shall show the location of the worst elements and section plane may be used if those are inside a body.

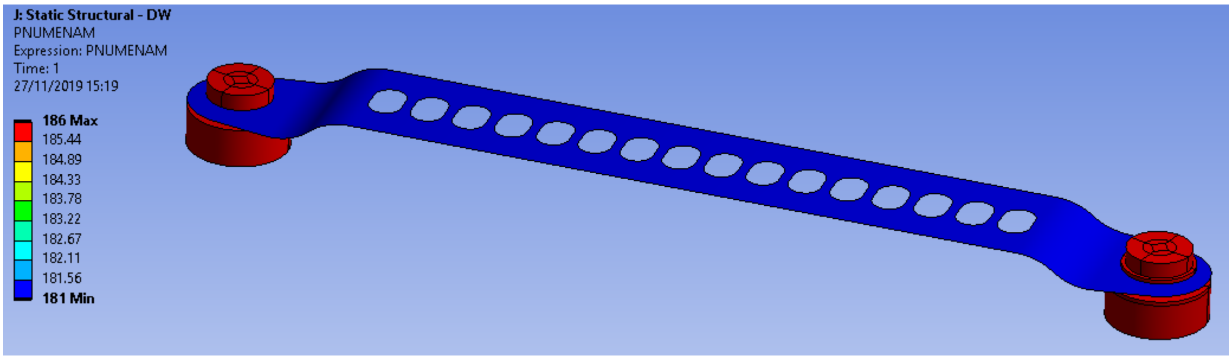


Figure B.18-3 – Plot of user defined result PNUMENAM that shows the parts made of SOLID186 (186 in the legend) and SHELL181 (181 in the legend) (Workbench example).

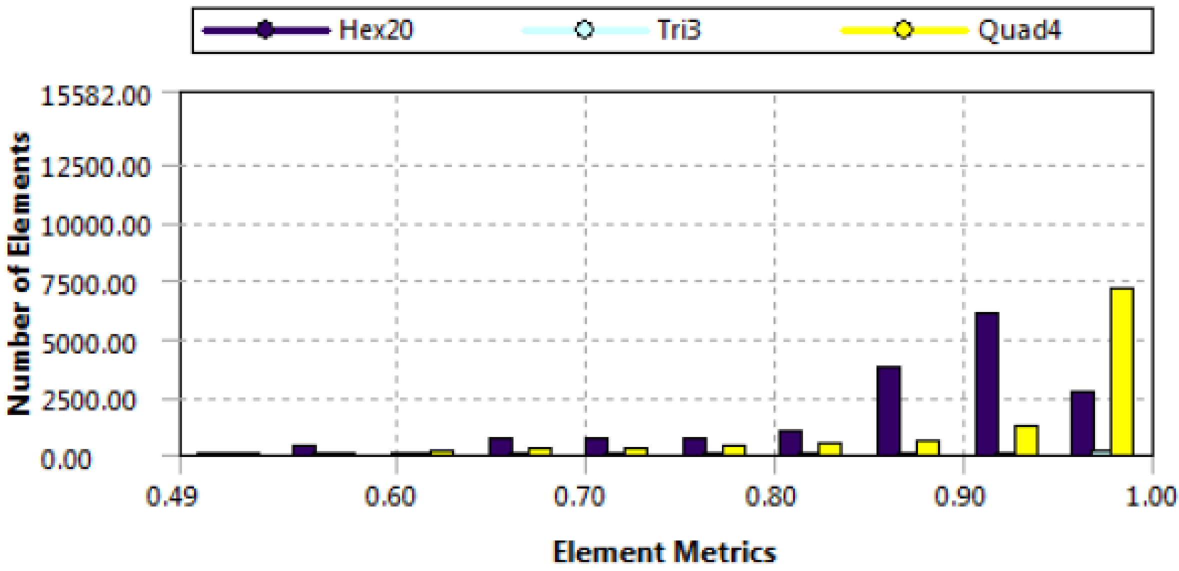


Figure B.18-4 – Plot of Jacobian ratio at Gauss points (Workbench example).

Appendix B.18.2.2 FINITE ELEMENT OPTIONS

All the relevant element options shall be listed for every element type. This information can be retrieved from the automatic report generated by clicking the *Report Preview* option, selected from the *Tools* group or *Home* tab. Some examples of these tables are shown below. Note that another option for reporting some element properties is to plot them by changing the *Display Style* in the *Geometry* object to the desired element property (e.g. *Body Type*, *Shell Thickness, Material*).

If any APDL commands are inserted with the purpose to modify the mesh, these shall be mentioned here (see Table B.18-14).

Line parts	Model type	Coordinate system	Offset type	Cross section
Toroidal ribs in lower cylinder	Beam	Local coordinate system 1	Centroid	LowerCylRib
Outer UCTS attachments	Pipe	Local coordinate system 2	Centroid	UCTS_outer

Table B.18-10 – Element options for line bodies. Non-listed settings are left at their default values (Workbench example).

Cross section	Type	Area (m^2)	I_{yy} (m^4)	I_{zz} (m^4)
LowerCylRib	RECT	$5.124\cdot10^{-3}$	$1.8831\cdot10^{-7}$	$2.5422\cdot10^{-5}$
UCTS_outer	CTUBE	$1.9757\cdot10^{-2}$	$1.5298\cdot10^{-4}$	$1.5298\cdot10^{-4}$

Table B.18-11 – Cross sections defined for line body elements (Workbench example).

Surface parts	Thickness (m)	Offset type
Outer_shell	$1.5\cdot10^{-3}$	Middle
Flange	$2\cdot10^{-3}$	Top

Table B.18-12 – Element options for surface bodies. Non-listed settings are left at their default values (Workbench example).

Point masses	Coordinate System	X coord (m)	Y coord (m)	Z coord (m)	Mass (kg)	I_{xx} (kg·m^2)	I_{yy} (kg·m^2)	I_{zz} (kg·m^2)	Behavior
Mass1	Global coordinate system	0	0	0.408	30.4	0	0	0	Rigid

Table B.18-13 - Element options for point masses. Non-listed settings are left at their default values (Workbench example).

Use of Elements	ETYPE	ENAME	KEYOPT(3)	KEYOPT(8)
Plate	53	SHELL181	2 - Full integration with incompatible modes.	2 - Store data for TOP, BOTTOM, and MID.

Table B.18-14 – Keyoptions of SHELL181 elements. Non-listed keyoptions are left at their default values (Workbench example).

Appendix B.18.2.3 CONTACTS

This paragraph is mandatory if contact elements are used in the FE model.

For every contact pair used in the analysis, the two associated parts of the FE model should be defined in the analysis report. The settings of each contact pair should be listed here. It is recommended to show a figure of each contact pair, highlighting the geometry that is part of the contact. This information can be retrieved from the automatic report generated by clicking the *Report Preview* option, selected from the *Tools* group or *Home* tab. Some examples of these tables are shown below.

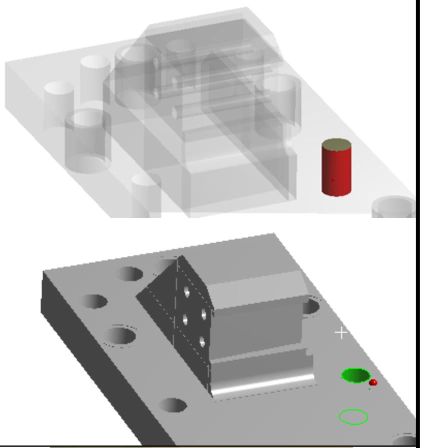
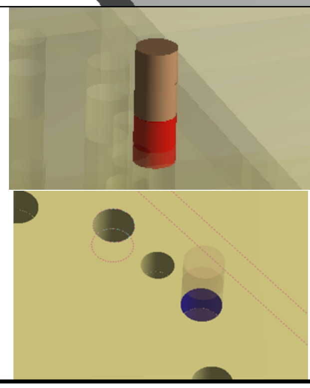
Contact pair	Type	Behavior	Formulation	Normal contact stiffness (GN/m³)	Tangential contact stiffness (GN/m³)	Figure
Shear pins-Female lug	Bonded	Auto asymmetric	MPC	Rigid	Rigid	
Shear pins-Embedded plate	Frictional (υ=0.1)	Auto asymmetric	Augmented Lagrange	1340	511	

Table B.18-15 – Contact pairs used in the model. Non-listed contact settings are left at their default values (Workbench example).

APPENDIX C Completion of Checklists

The completion of Reviewer and Independent Peer Review checklists shall be performed following the requirements listed in Table C-1.

Check	Requirements and Guidance
R1	**Report format.** The requirements of 0 (not including sub-paragraphs) are met, including that: • IO analyses reports follow the template for seismic analysis reports, [8]. Reports from DAs and subcontractors either follow the template, or contain all of the contents described in 0. • Reports stored in IDM are in the Microsoft Word format (.doc or .docx). If not, a Word version is stored in IDM as an attached file.
R2	**Abstract, purpose and scope.** The requirements of the following paragraphs are met: • Appendix A.4.1. The abstract of the seismic analysis report contains the following information: o The ITER SSC to which the seismic analysis is related. o The assessed components or parts of the system. PBS codes should be used where practical. o The type of failure modes that were assessed, e.g. buckling or fatigue, if applicable. o The loads that were considered in the seismic analysis. o Any recent significant changes of design, structural design criteria or load specification. o A statement that the report was written following these instructions and that the loads applied in the assessments are consistent with the system load specification. • Appendix A.4.4. The “purpose” section outlines the aim of the report. • Appendix A.4.5. The abstract defines the applicability of the report, typically in terms of geometry, loads, results and context.
R3	**Scope of reviewers.** The requirements of Section 7.3 and Appendix A.4.3 are met, including that: • For PIA analyses, the SRO has been assigned as an Observer. • One or more Reviewers have been assigned that between them cover at least all of the points listed in Paragraph 7.3.2. • Where required by [1], an Independent Peer Reviewer has been assigned that covers at least all of the points listed in Paragraph 7.3.3. • A Technical Checker has been assigned to cover at least all of the points listed in Paragraph 7.3.3. • The scope of the review is specified for each reviewer, either in IDM or in the report itself.
R4	**Definitions and abbreviations.** The requirements of Appendix A.4.6 are met, i.e. all definitions and abbreviations used in the report are listed, in alphabetic order.
R5	**Units.** The requirements of Appendix A.1.5 and Appendix A.4.7.1 are met, i.e. analyses are performed using S.I. base and derived units.

Check	Requirements and Guidance
R6	**Geometry (excluding applicability).** The requirements of the following paragraphs are met: • Appendix A.1.2, including that: o Analyses are based on geometry that is unambiguously traceable. Where relevant, the references are approved. o The uncertainty in the geometry, e.g. due to tolerances, has been considered. o Analyses performed during or after the construction phase has considered any relevant non-conformances. o Deviations from the current approved design geometry have been justified. o The quantitative effect of the deviations on the results has been estimated, and considered in the conclusions of the analysis • Appendix A.4.7.2, including that: o A figure of the geometry used in the seismic analysis has been shown. o Special attributes of the geometry that cannot be easily recognized in a figure and that are relevant to the analysis has been described. o The magnitudes of geometrical imperfections considered in the analysis through any modification of the initial FE mesh have been stated.
R7	**Applicability of geometry (*).** Analyses are based on applicable geometry, i.e. the current approved design.
R8	**Material properties (excluding applicability).** The requirements of the following paragraphs are met: • Appendix A.1.3, including that analyses consider the uncertainties in material properties. • Appendix A.4.7.4, including that: o The physical material properties of the analysed SSC are listed, and traceable to approved references. o It is clearly documented which parts are made from which materials.
R9	**Applicability of material properties (*).** Analyses are based on physical material properties that are consistent with the procured materials of the analysed SSC.
R10	**SDCs (excluding applicability).** • The requirements of 0 are met, including that: o The design code applicable to the analysis of the system is stated. o The design code is consistent with the definition in the relevant SRD. o The rules and limits from the design code applicable to this seismic analysis are extracted from the design code and summarized in the analysis report. o Relevant service limits applicable to the system are listed, and linked to approved references. • The SDCs are appropriate.
R11	**Applicability of SDCs (*).** Analyses are based on the applicable SDCs and service limits.
R12	**Loads (excluding applicability) (**).** The requirements of Appendix A.1.4 and Appendix A.4.7.7 are met, including that: • All input loads used for analyses are listed and described clearly and unambiguously. • All listed input loads come from the relevant approved System Load Specification. • Any uncertainties in the loads are reported, and considered in a conservative manner. • For transient loads, the time functions of all loads are given in the form of either a table or a diagram that allows the identification of characteristic magnitudes of the time functions.
R13	**Applicability of loads (*).** The approved System Load Specification on which the analysis is based is still consistent with the current design of the SSC.

Check	Requirements and Guidance
R14	**Conceptual model and analysis methodology.** The requirements of the following paragraphs are met: • Appendix A.1.1, including that: ○ The chosen conceptual model represents the physical reality sufficiently accurately to cover the intended purpose of the analysis. ○ Appropriate analysis method(s) are used. • Appendix A.4.8, including that: ○ The principle of the analysis approach is described. ○ The conceptual model is justified, in particular the inherent simplifications compared to the physical reality. ○ Justification is provided that the analysis methods are used in their validated domains. • If the linear equivalent static analysis method is selected, the associated requirements in Appendix B.2.3 shall be met, including: ○ The applicability of the method is justified, particularly for irregular structures. ○ The amplification factor α_{st} and the equivalent static acceleration a_{eq} are appropriately defined. • If the response spectrum analysis method is selected, the associated requirements in Appendix B.2.5.1 are met, including: ○ The cut-off frequency f_c is correctly selected for the modal analysis. ○ The modal damping ratios are defined in an appropriate manner. ○ The modal combination method is appropriately selected. ○ The response to the pseudo mode (representing modes with natural frequencies higher than f_c) is considered and correctly combined with the combined maximum response to all other modes, e.g. the missing mass option in ANASYS is on. ○ The spatial combination method is appropriately selected. • If the transient method is selected, the associated requirements in Appendix B.2.5.2 are met. • If some other method introduced in Appendix B.2 is selected, the associated requirements in Appendix B.2 are met. • In case of multiple supported SSCs, the associated requirements in Appendix B.2.6 are met.

Check	Requirements and Guidance

R15	**Description of FE analyses (only applicable for FE analyses).**
	The FE analyses well documented, and are an appropriate implementation of the conceptual model and analysis methodology. The requirements of the following paragraphs are met:
	• Appendix A.2.1 and Appendix A.4.9.2, including that:
	  o The name and version number of any software package used to perform FE analyses is stated.
	  o Any software package used is validated.
	  o It has been justified that software packages have been used in their validated domain.
	  o It is stated what uncertainties, if any, are associated with the use of the validated Finite Element software package for the reported analysis.
	  o If a validated Finite Element software package has non-negligible uncertainties when used properly, the uncertainties are covered either by performing sensitivity studies or by applying a suitable safety factor to the results.
	• Appendix A.2.2 and Appendix A.4.9.3, including that:
	  o All coordinate systems used in any FE analyses are defined
	  o The global coordinate system for FE models has its positive z-axis pointing vertically upward.
	• Appendix A.2.3, including that:
	  o The choice of element types and shapes is justified.
	  o Shape checking has been performed and reported. If poor quality elements have been identified by the check, their use is justified.
	• Appendix A.2.4, including that:
	  o The solution settings chosen for the analysis are documented and justified.
	• 0 (excluding paragraphs covered above), including that:
	  o The type of analysis is stated.
	  o All material properties used in the FE model are listed
	  o A description of the FE model is given, including pictures of the mesh, and details of element properties (e.g. element types and options, real constants and section properties.
	  o Each set of boundary conditions (BCs) is described, including degrees of freedom and the coordinate system. The BCs are also shown on a figure.
	  o In case the BCs are not constant throughout the analysis, the changes are described.
	  o Internal constraints (e.g. coupled equations) are described, and shown on one or more figures.
	  o The report describes how the defined loads are applied to the FE model. The described application of loads to the FE model is consistent with the System Load Specification.
	  o The solution settings are listed and justified.

R16	**Hand calculations (only applicable for hand calculations).** The hand calculations are well documented, and are an appropriate implementation of the conceptual model and analysis methodology. The requirements of Appendix A.3 are met, including that: • All equations used in the calculation are shown. • A reference is given for any non-trivial analytical formulas used. • Equations are referenced • The result of calculations shall be given including the unit. • All symbols used in the equation shall be defined in the report.

R17	**Results.**
	• Results are reasonable for the given inputs and assumptions.
	• The requirements of Appendix A.4.10 are met, including that:
	  o All results values are given with their units.
	  o Clear titles are given when presenting graphs.
	  o All relevant results to meet the scope of the seismic analysis are given.
	  o Results shall be given corresponding to the design criteria.
	  o When giving reaction forces or moments the direction of a positive reaction force shall be specified or shown in a figure, unless it is obvious. As the direction is dependent on the component the load is acting on, the latter shall be specified.
	  o The point of summation of moments shall be explicitly stated.
	  o The coordinate system used for the results shall be specified.
	  o If modal damping is used for response spectrum or mode-superposition transient analyses, the damping ratio for each mode is reported.

Check	Requirements and Guidance
R18	**Verification of FE analyses (only applicable for FE analyses).** • Appendix A.2.5.1, including that analysis software and its installation of the computer used for the analysis shall be qualified according to [3]. • Appendix A.2.5.2, including that: o A mass check is performed and reported if inertial effects are relevant to the analysis. o The total mass, centre of gravity and if possible the inertia of the FE model is reported and compared to the values listed in the system load specification. Any significant differences are justified in the report. • Appendix A.2.5.3, including that: o A gravity load check is performed and reported if gravity is part of the load case considered in the analysis. o It has been checked whether the reaction forces on the constraints of the FE model correspond to the weight of the FE model. o In case the FE model has more than one constraint it has been checked whether the distribution of the reaction forces is roughly consistent with the centre of gravity of the FE model. • Appendix A.2.5.4, including that: o A static structural load check has been performed and reported, checking that the total reaction forces and moments on the supporting constraints correspond to the total applied structural loads. o In case the FE model has more than one constraint it has been checked whether the distribution of the reaction forces is reasonable. o If loads are transferred from other models the resulting reaction forces and moments from the different models have been compared. o When complex structures made of several sub-structures are considered, the resulting forces and moments have been checked for each sub-structure. o The point(s) considered for the calculations of resulting moment(s) are reported. o In the special case that the applied structural loads cancel each other out it has been verified that the: ▪ sum of the support reaction forces is close to zero. ▪ magnitude of the applied loads, including bolt prestress, is approximately correct. o Appendix A.2.5.5, including that it is demonstrated that the behaviour of the contact elements is as intended. o Appendix A.2.5.6, including that it is demonstrated that damping has been applied as intended • Appendix A.2.5.7, including that: o The mesh density is justified, e.g. by means of a mesh sensitivity study. o The sensitivity of the results to the mesh density is considered in the interpretation of the results. • Appendix A.2.5.8 and Appendix A.4.11, including that the results of FE analyses have been verified by comparing them to those of alternative calculations.
R19	**Conclusions.** • The conclusions are reasonable and representative of the outputs. • The conclusions properly meet (or cover) the scope and purpose. • The requirements of Appendix A.4.12 are met, including that: o The conclusions summarize the most significant findings, and are comprehensible for persons familiar with the design and loads of the system, with an engineering background but not necessarily with expertise in seismic analyses. o The result values given in the conclusions consider the uncertainty of the seismic analysis. o Results for which the FE model cannot meet accuracy requirements are either not reported, or marked as "preliminary" or "best estimates". o Result values are not be given without a judgement.
R20	**References.** The requirements of Appendix A.4.13 are met, including that: • All documents that are referenced by the analysis report are listed. • References are stored in IDM or are publically available. • References in IDM are approved. • References to IDM documents include the version numbers. • An approver and at least one reviewer must be assigned to IDM references.

Table C-1 – Requirements for completion of Reviewer and Independent Peer Reviewer checklists.

(*) This check shall be performed by the RO of the SSC.

(**) On the Reviewer Checklist, this check shall be performed by the RO of the SLS. On the Independent Peer Reviewer checklist this requirement does not apply.

The completion of Technical Checker checklists shall be performed following the requirements listed in Table C-2.

Check	Requirements and Guidance

TC1	**Conceptual model and analysis methodology.**
	The requirements of the following paragraphs are met:
	• Appendix A.1.1, including that:
	  ○ The chosen conceptual model represents the physical reality sufficiently accurately to cover the intended purpose of the analysis.
	  ○ Appropriate analysis method(s) are used.
	• Appendix A.4.8, including that:
	  ○ The principle of the analysis approach is described.
	  ○ The conceptual model is justified, in particular the inherent simplifications compared to the physical reality.
	Justification is provided that the analysis methods are used in their validated domains.

TC2	**Mathematical model.** The mathematical model is described properly, and is appropriate given the analysis methodology. This includes the: • Coordinate system(s) used. • FE material properties. • FE mesh. • BCs. • Load application. • Solution settings.
TC3	**The analysis model is properly stored in the analysis database.** The models are stored in the ITER analysis database, and respect the requirements of the MQP Instructions for the Storage of Analysis Models (ITER_D_U34WF3), including that they: • Include all files necessary to get the reported results (e.g. including macros & spreadsheets). • Are linked to the analysis report, and their metadata is filled properly. • Are stored in a sensible and organized folder of IO's Analysis Model Database. • Are in a ready-to-run state. The technical checker shall rerun the analyses to verify this. • Are commented/organised to be clearly and unambiguously understandable by a third party. Proper storage formats are used, i.e. that privileges robustness and exhaustiveness.
TC4	**The model in the database matches the report.** The geometry, mesh, element types, material and element properties, BCs and loads of the model in the database match the description in the report.
TC5	**The results of the model in the database match the description in the report.** Important results match those described in the report. Given that results are often not stored in the database this check may require rerunning the analyses, see TC3.
TC6	**Analysis results are reasonable, and hand calculations are correct.** • Results are reasonable for the given inputs and assumptions, i.e. no observations are made that indicate that there are errors in the analysis. • The analysis uncertainties are judicious. • The numerical evaluations of hand calculations are correct.

Table C-2 – Requirements for completion of Technical Checker checklists.

APPENDIX D Compliance Matrix for Checking Requirements from [1]

The compliance matrix below demonstrates that all the requirements for checking of analysis reports required by [1] are covered by requirements given in these Instructions. In this table 'R#' means 'Reviewer check number #'; 'TC#' means 'Technical Checker check number #'. They correspond to the checks listed in Subsections 7.3.2, 7.3.3 and 7.3.3, which are propagated to [5], [6] and [7].

	Requirement from [1]	Covered by
Review / Technical Check	Check that the requirements defined in the specifications are met including the scope and purpose as defined in the technical specification.	A contractual issue, not in the scope of these Instructions.
	Check that the calculation model data appropriately reflect the geometrical data and interfaces of the object under investigation.	R6, R7, R14, TC1
	Check the basic approach, assumptions, subject-specific data (such as loads), and any equations or formulas applied are appropriate.	R8, R9, R12, R13, TC1
	Check that input data are consistent with requirements or validated by referenced sources.	R6, R7, R8, R9, R10, R11, R12, R13
	Check the calculations are mathematically correct.	TC6
	Check the requirements and acceptance criteria are appropriate and used correctly.	R10, R11
	Check the conclusions reached are reasonable and consistent with the analysis or calculation approach, assumptions, input, and acceptance criteria.	R17, R19, TC6
	Check that the software is validated for the scope and purpose of the analysis.	R15
Independent Peer Review	Design or analysis philosophy is sound	R14, TC1
	Structural system, materials, acceptance criteria, and other pertinent factors are considered.	R6, R7, R8, R9, R10, R11
	Analysis or calculation approach is reasonable and appropriate.	R14, R15, R16, TC1
	Inputs are reasonable and correct.	R6, R7, R8, R9, R10, R11, R12, R13
	Assumptions are reasonably substantiated and justified.	R14, TC1
	Mathematical formulations and/or computer models (see def.) are appropriate and contain sufficient detail.	R15, R16, TC2
	Outputs are reasonable for the given inputs and assumptions	R17, TC6
	Acceptance criteria used are appropriate.	R10
	Conclusions are reasonable and representative of the outputs	R19

Table D-1 - Compliance Matrix for "Analysis and Calculations", [1].

APPENDIX E Compliance Matrix for INB Order [4]

The compliance matrix between these instructions and the INB order dated 7 February 2012 [4] is shown in Table E-1 and Table E-2 below. By following these Instructions and the references in the table below, all requirements from [4] relevant to seismic analyses are met.

For Article 3.8 of the INB order, the requirements have been taken from the summary table of [18], rather than directly from the INB order.

Article in [4]	Requirement	Covered by
2.2.1	Surveillance of external interveners. The operator informs all external interveners of the provisions required for implementing the Ministerial Order hereof.	Chapter 2, which states that the rules governing the propagation of the requirements specified in these Instructions are specified in [19].
2.2.2	The operator surveys external interveners.	Outside the scope of these instructions. The requirements for surveillance plans are defined in [1].
2.5.2.II	The protection-important activities are carried out in accordance with procedures and using means for meeting *a priori* the requirements defined for these activities and for the protection-important components concerned, and to ensure them *a posteriori*.	These Instructions represent the procedures that shall be followed to meet *a priori* the requirements for PIA analyses. The reviews required by Section 7.3 ensure *a posteriori* that analyses under direct control of ITER IO meet the defined requirements. The surveillance plans required by [1] ensure *a posteriori* that analyses performed by external interveners meet the defined requirements.
2.5.3	Each protection-important activity undergoes technical monitoring, to ensure that: the activity is carried out in compliance with the requirements defined for the activity and, if necessary, for the protection-important components concerned; appropriate corrective and preventive actions have been defined and implemented.	Subsection Appendix A.2.5, Section 7.3, the surveillance requirements defined in [1].
2.5.5	Protection-important activities, their technical monitoring and the checking and assessment actions are carried out by individuals with the appropriate skills and qualifications.	Section 7.2, the surveillance requirements defined in [1].
2.5.6	Protection-important activities, their technical monitoring and the checking and assessment actions are documented and are traced to demonstrate *a priori* and to …	The *a priori* definition of PIAs is outside the scope of these Instructions. The surveillance requirements are defined in [1].

Article in [4]	Requirement	Covered by
	… check a *posteriori* that they comply with defined requirements.	The reviews required by Section 7.3 ensure *a posteriori* that analyses under direct control of ITER IO meet the defined requirements. The surveillance plans required by [1] ensure *a posteriori* that analyses performed by external interveners meet the defined requirements.
	The documents and corresponding recordings are kept updated, are easily accessible and legible, protected, kept under appropriate conditions and archived for an appropriate and justified period of time.	Chapter 8.

Table E-1 - Compliance Matrix for the INB Order [4], excluding Article 3.8.

Group	Requirement	Covered by
Input data	Use of referenced, updated and validated input data.	Appendix A.1.2 (geometry), Appendix A.1.3 (materials), Appendix A.1.4 (loads).
	Use of controlled assumptions.	Appendix A.1.1 (justification of conceptual model and analysis method).
	Assessment of the uncertainties in input data.	Appendix A.1.4 (loads).
Methods	Establishment of a range of assumptions and sensitivity studies when assumptions include uncertainties.	Appendix A.1.1 (justification of conceptual model and analysis method).
	Verification of the consistency with safety demonstration.	Subsection 7.3.1 (SRO review).
	Establishment of a list of validated and appropriate methods.	Appendix A.1.1 (justification of conceptual model and analysis method).
	Use of methods in their validation domain.	Appendix A.1.1 (justification of conceptual model and analysis method).
	Verification of the methods consistency with safety demonstration.	Subsection 7.3.1 (SRO review).
	Sensitivity studies to be performed for covering methods uncertainties or additional safety factor in the results.	Appendix A.1.1 (justification of conceptual model and analysis method). Appendix A.4.12
Codes and calculation tools	Establishment of a list of validated and appropriate codes.	Outside the scope of these Instructions.
	Use of methods in their qualification domain.	Appendix A.2.1 (software package).
	Verification of the methods' consistency with safety demonstration.	Subsection 7.3.1 (SRO review).
	Sensitivity studies to be performed for covering code uncertainties or additional safety factor in the results.	Appendix A.2.1 (software package), Appendix A.4.9.1 (type of analysis).
Reports and results	All input data, methods, codes and their validity domain and uncertainties to be included.	Reporting of input data: Appendix A.4.7.2 (geometry), Appendix A.4.7.4 (physical material properties), Appendix A.4.7.7 (loads). Reporting of analysis method, including its validity and associated uncertainties: Appendix A.4.8 (methodology). Reporting of FE software packages, including their validity and associated uncertainties: Appendix A.4.9.2 (software package).
	Intermediate and final results to be expressed in international units.	Appendix A.4.7.1 (units).
	Sensitivity studies to be performed for covering uncertainties or additional safety factor in the results.	Appendix A.4.8 (methodology). Appendix A.4.12 (conclusions).
Acceptance criteria	The acceptance criteria to be substantiated and checked against potential safety limits and when applicable design margins brought by codes.	Appendix A.4.10 (results).
	Margins and safety factor to be expressed with regards to safety limits.	Appendix A.4.12 (conclusions).

Table E-2 - Compliance Matrix for Article 3.8 of the INB Order [18].