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EXTERNAL REFERENCE / VERSION

Rules or Handbooks or Guidelines

ITER Structural Design Code for Buildings (I-SDCB) - Part1: Design Criteria

The present specification defines the design criteria for the ITER facility buildings that have safety requirements.

Approval Process			
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Change Log			
ITER Structural Design Code for Buildings (I-SDCB) - Part1: Design Criteria (283B24)			
Version	***Latest Status***	***Issue Date***	***Description of Change***
v1.0	In Work	06 Feb 2008	ITER Structural and Design Code for Buildings
v2.0	Approved	30 May 2008	ITER Structural and Design Code for Buildings
v2.1	Approved	27 Aug 2008	ITER Structural and Design Code for Buildings
v2.2	In Work	11 Jun 2009	
v2.3	In Work	16 Jun 2009	Author resubmitted document - formatting changes made
v2.4	In Work	01 Jul 2009	Author changed according to comments
v2.5	In Work	28 Aug 2009	Document Updated - see change bars
v2.6	Approved	03 Sep 2009	Document Updated - see change bars
v2.7	Approved	10 Nov 2009	updated version for AE PA FFRs Refer change bars/ tracked changes for details of updates
v2.8	Signed	07 Sep 2011	Updated Version
v2.9	Approved	13 Oct 2011	Consideration of the PCR-M100 - Updates to baseline documents (for design of Buildings with Safety Requirements) (5XSHKS) and PCR-397 - Revision of the Load Specification Document PR annex (ITER_D_222QGL). It shall be noted that it was agreed at the CCB-077 that given the impact on building design of PCR 397, it would be beneficial to the project if BSI immediately updated the three impacted documents (‘’Safety Requirements for Buildings", “Load Specification for building with Safety Requirements” and “ITER Structural Design Code”) on the basis of the draft of the Load Specification (PR Annex) as prepared already by SYSA section. The chairman approved this approach and asked BSI to update their three documents for issue to F4E as quickly as possible CCB-077 (29 September 2011) Executive summary (6LTPHW)
v2.10	Approved	13 Jun 2012	this version of the document includes updates to the list of load combinations that are to be considered (PCR-397) and some minor updates to address minor comments on the previous version
v3.0	Approved	13 Oct 2017	New version to include the Tokamak Assembly Preparation Building (PBS 62.22)
v3.1	Signed	05 Aug 2022	New version to integrate the combinations and the criteria for design associated to SL-3 (Design extension Hazard). This update is needed in the frame of the Tokamak Assembly Hold Point Task Force and in particular as an outcome of WP4 Design Review held on the 28th July 2022. See as justifications of the need of this update: the Term of Reference ITER_D_7H9JAE - ToRs Assembly Hold Point Task Force the Design Plan of the Assembly Hold Point WP4: Tokamak Complex Building - Assembly Hold Point - Complementary Evaluation of the Tokamak Support Structure - Design Plan (7HCULL)
v3.2	Signed	26 Sep 2022	VCD ITER Structural Design Code for Buildings (I-SDCB) - Part1: Design Criteria (283B24) detailing all changes from v3.0 to 3.2 This update is required within the frame of the Tokamak Assembly HP Task Force and in particular as an outcome of the WP#4 design delivery and associated design review. See as justifications of the need of this update in ToRs Assembly Hold Point Task Force (7H9JAE v3.1) (current) and Tokamak Complex Building - Assembly Hold Point - Complementary Evaluation of the Tokamak Support Structure - Design Plan (7HCULL v1.6) (current) . PCR-1414 updates cover exclusively the civil works of the Tokamak Support Structures, extended to the full B2 slab, lower basemat and ASBs/plinths within the Tokamak Complex. There are also answers to comments raised on previous versions and implementation of PCRs as noted below.

			1. IDM Comments (v3.0) requesting clarifications Section 3.1.2: • Add note to explain "Qmfield – See section 7.1.3 of [3] for an explanation of the treatment of these loads" • Correcting the ? index (implementing the RFI-AE PA-127 - TK complex - Live load during seismic event; inversion of 0 and 2 • Change (IDM comment linked SDR #85), B15, B37, B44, B47 & B71 removed from Combination 34 (SL-2 + Int. Fire) 2. PCR-1414: Update of Load Specifications and Safety requirements for Buildings to include the SL-3 description, criteria and an explanation on temperature effects Section 3.1.2: • Text added to describe the live load cases in the different seismic conditions (SL-1, SL-2 & SL-3) Section 3.4: • (Note above Matrix 3.2) Note added to confirm no margin required in combinations that include SL-3 seismic loads (Design extension combinations) • Matrix 3.2 – add new Design Extension combinations (combination 56, 57 & 58) for SL-3, HIG and LOCA loading actions. (including footnotes to the table) Section 4.5.2.5: • New section added to describe the SLS.R5 Limitation of Stress and Strain to be considered for Design Extension situations Section 4.5.3: • No change compared to the v3.0 of this document (at v3.1 a new column was included in Table 4, but this change is rejected). Section 4.6.1: • Third criteria added describing the confinement criteria that are to be applied in the design extension situations (SL-3) Appendix A3.3: • Text added to confirm the associated live load mass that shall be considered during SL-3 seismic 3. Answering to IDM comment from v3.1 review (Tyge Schioler) Section 5.3 Table 5: • No change from v3.0 of this document. Returning partial factors (gamma.m) to those presented in v3.0 (at v3.1 update there was a misinterpretation of an IDM comment)
v3.3	Approved	13 Oct 2022	VCD ITER Structural Design Code for Buildings (I-SDCB) - Part1: Design Criteria (283B24) detailing all changes from v3.0 to 3.3 This update is required within the frame of the Tokamak Assembly HP Task Force and in particular as an outcome of the WP#4 design delivery and associated design review. See as justifications of the need of this update in ToRs Assembly Hold Point Task Force (7H9JAE v3.1) (current) and Tokamak Complex Building - Assembly Hold Point - Complementary Evaluation of the Tokamak Support Structure - Design Plan (7HCULL v1.6) (current) . PCR-1414 updates cover exclusively the civil works of the Tokamak Support Structures, extended to the full B2 slab, lower basemat and ASBs/plinths within the Tokamak Complex. There are also answers to comments raised on previous versions and implementation of PCRs as noted below. 1. IDM Comments (v3.0 and v3.2) requesting clarifications Section 3.1.2:

			• Add note to explain "Qmfield – See section 7.1.3 of [3] for an explanation of the treatment of these loads" • Correcting the PSI index (implementing the RFI-AE PA-127 - TK complex - Live load during seismic event; inversion of 0 and 2 Section 3.1.3: • Add Design Extension Earthquake (E.DE) to table, and notes to link other seismic levels (SL-1, SL-2, SMHV) Section 3.4: • In matrix 2, add a row for Design Extension Earthquake (with footnote about applicability) • Change (IDM comment linked SDR #85 and SDR #572), B15, B37, B46, B47, B71 & B75 removed from Combination 34 (SL-2 + Int. Fire), 12 & 19 added. Description of Combination 34 changed to say "SL-2 followed by Fire, followed by SMHV". Reference [200] included in Section 2.4 (link to SDR text to explain which rooms of Tokamak Complex consider Combination 34) • Delete B71 and B15 from Combination 48 (previously indicated as removed by strikethrough font, now removed completely) • Delete "see note over page" as the note is now on the same page • Typo corrected in note below Matrix 3.2, now reads "… given in Matrix 3.2 above" 2. PCR-1414: Update of Load Specifications and Safety requirements for Buildings to include the SL-3 description, criteria and an explanation on temperature effects Section 3.1.2: • Text added to describe the live load cases in the different seismic conditions (SL-1, SL-2 & SL-3) updated at v3.3 and moved to section 3.1.3 Section 3.4: • (Note above Matrix 3.2) Note added to confirm no margin required in combinations that include SL-3 seismic loads (Design extension combinations) • Matrix 3.2 – add new Design Extension combinations (combination 56, 57 & 58) for SL-3, HIG and LOCA loading actions. (including footnotes to the table) Section 4.5.2.5: • New section added to describe the SLS.R5 Limitation of Stress and Strain to be considered for Design Extension situations Section 4.5.3: • No change compared to the v3.0 of this document (at v3.1 a new column was included in Table 4, but this change is rejected). Section 4.6.1: • Third criteria added describing the confinement criteria that are to be applied in the design extension situations (SL-3) Appendix A3.3: • Text added to confirm the associated live load mass that shall be considered during SL-3 seismic 3. Answering to IDM comment from v3.1 review (Tyge Schioler) Section 5.3 Table 5: • No change from v3.0 of this document. Returning partial factors (gamma.m) to those presented in v3.0 (at v3.1 update there was a misinterpretation of an IDM comment)
v3.4	Approved	16 Sep 2025	Updated in accordance with PCR-001614 (creation of B20, K3, LC20) and PCR-001619 (creation of B18, K2, LC19): • Figure 1 updated to include New Buildings in extract of Site Master Plan. • Matrix 3.2 – Combinations 34, 39, 43, 44 & 48. B18, B20, K2 and

			K3 added to these combinations

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1 Scope

1.1 General organization of the design

1.1.1 Design specifications

Design Specifications for the ITER facility buildings that have safety requirements are made of the three following documents:

- Safety Requirements for Buildings (2E4KSJ),
- Load specification for buildings with safety requirements (2ERTXQ), which describes the design actions,
- ITER building code I-SDCB (283B24), which defines the design criteria for the civil works structures.

1.1.2 Buildings in scope of this specification

The present specification applies for the buildings listed in the chapter 1 of the Safety Requirements for Buildings (2E4KSJ).

Buildings are identified in the general layout drawing of the ITER site in figure hereafter.

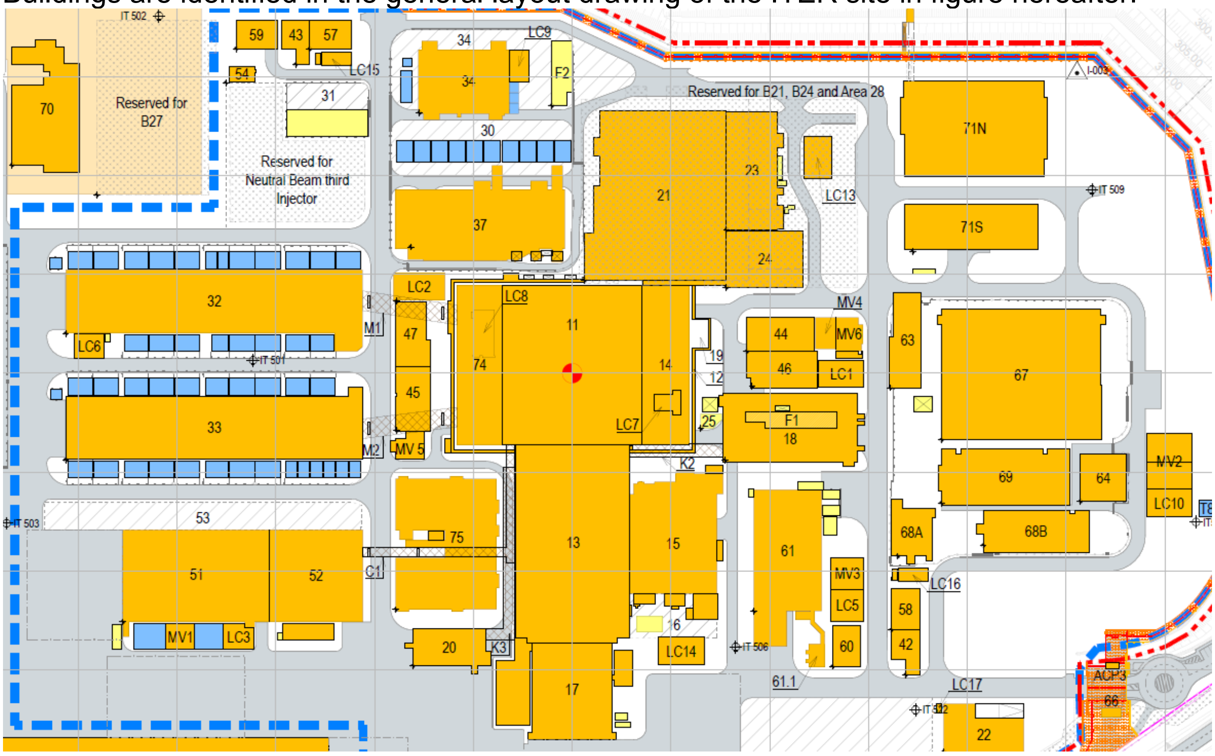


Figure 1 : General layout of the central part of the ITER site

1.1.3 Items beyond the scope of this specification

Following items are beyond the scope of the present document:

- Geotechnical aspects;
- Finishing works, coating and painting;
- Non-structural elements;
- Steel liners, if any.

1.2 Codes and Standards

The design principles are based on the Structural Eurocodes:

– The basic principles for structural design are those of Eurocode EN 1990 (EC0), in particular for the concept of limit states, the definition of partial safety factors and load combinations.
– Euronorm EN 1991: Eurocode n°1 gives basic requirements related to loads.
– Euronorms EN 1992, EN 1993, EN 1994, EN 1995, EN 1996: Material Eurocodes n°2 to 6 (EC2 to EC6) shall be used for the design of structural elements, except when otherwise specified in the present document.
– Although geotechnical aspects shall be dealt with in specific studies, the principles of Euronorm EN 1997 - Eurocode 7 (EC7) - and of Euronorm EN 1998 - Eurocode 8 - part 5 may be referenced to.
– Principles and application rules proposed by Euronorm EN 1998 - Eurocode 8 are applicable; however specific analysis procedures and requirements are enforced by the present document.
– In the present document, unless otherwise specified, reference to a Eurocode part includes the corresponding Euronorm and the French National Annex.

In case of contradictory requirements, the order of applicability of the specifications is as follows:

– the present design specification,
– the Eurocodes or other normative European standards,
– the French national standards, when corresponding Euronorms do not exist.

All construction works, including structures, shall be designed with suitable structural strength, serviceability and durability in conformity with the design requirements and for the corresponding acceptance criteria defined in this document, including referenced standards.

The present Part 1, dedicated to the design criteria, is intended to be used with the construction rules developed in Part 2 [4].

1.3 Design life time

The design life time of the buildings is defined in [2]. The design of structures, in terms of resistance and durability, shall take into account this design lifetime.

2 References

2.1 Project specifications

[1] *void*
[2] Safety Requirements for Buildings (2E4KSJ)
[3] Load specification for buildings with safety requirements (2ERTXQ)
[4] ITER Building Code II-SDCB (2E2U9X)

2.2 Regulatory rules, norms and standards

[10] Structural Eurocodes transposed in French standards
NF EN 1990 Structural Eurocodes – Basis of structural design

NF EN 1990/A1 Structural Eurocode – Basis of structural design Amendement A1
NF EN 1990 /NA French National Annex
NF EN 1991 Eurocode 1 – Actions on structures
NF EN 1991 /NA French National Annex
NF EN 1992 Eurocode 2 – Design of concrete structures
NF EN 1992 /NA French National Annex
NF EN 1993 Eurocode 3 – Design of steel structures
NF EN 1993 /NA French National Annex
NF EN 1994 Eurocode 4 – Design of composite steel and concrete structures
NF EN 1994 /NA French National Annex
NF EN 1997 Eurocode 7 – Geotechnical design
NF EN 1997 /NA French National Annex
NF EN 1998 Eurocode 8 – Design of structures for earthquake resistance
NF EN 1998 /NA French National Annex

[11] Fundamental safety rules (RFS), *Règles Fondamentales de Sûreté*
RFS I.1.a for risks associated with aircraft crashes
RFS I.1.b for risks associated with industrial environments and communication routes
RFS I.2.e for external flood hazards
RFS I.3.c for geological and geotechnical site studies, soil characterization and terrain behaviour studies
RFS2001-01 for seismic risk

[12] Nuclear safety authority guidelines (ASN), *Guides de l'Autorité de sûreté nucléaire*
ASN/guide/2/01: Taking seismic risk into consideration for nuclear facility civil works design - *Prise en compte du risque sismique à la conception des ouvrages de génie civil d'installations nucléaires de base à l'exception des stockages à long terme des déchets radioactifs*
ASN/guide/7/01: *Guide relatif à l'application de l'Arrêté du 31/12/99. Thème: Incendie*

[13] CEB bulletin n°233 - Design guide - « Design of fastenings in concrete », 1997

[14] CEB bulletin n°187 - « Concrete structures under impact and impulsive loads », 1988

2.3 Standards for materials and devices

[100] NF EN 10080 – Steels for the reinforcement of concrete – Weldable reinforcing steel – General

[101] NF EN 10028-2 – Flat products made of steels for pressure purposes - Part 2 : non-alloy and alloy steels with specified elevated temperature properties

[102] NF EN 10025 – Hot rolled products of non-alloy structural steels

[103] NF EN 10210 – Hot finished structural hollow sections of non-alloy and fine grain steels

[104] NF EN 10219 – Cold formed welded structural hollow sections of non-alloy and fine grain steels

[105] NF EN 10138 – Prestessing steels

[106] prEN 15129 2007– Antiseismic devices *(note: the definitive standard EN 15129 shall be taken into account for detailed design at its final issue)*

[107] EN 1337 Structural bearings

[108] ETAG 001 : Guideline for European Technical Approval of Metal Anchors for Use in Concrete

[109] CISMA : *Recommandations à l'usage des professionnels de la construction pour le dimensionnement de fixations par chevilles métalliques pour le béton - Amendement décembre 2004*

2.4 2.4 ITER Documents

[200] ITER_D_3DBL8C - Supplier Deviation Request #572 – AE PA 6.2.P2.EU.02 – PBS 61/62/63/65 - Proposal to refine the Requirements for Stability after an Earthquake Event followed by a Fire Event followed by an Earthquake Event

3 Actions and combinations of actions

3.1 Definition of actions

The characteristics values of the design actions are specified in [3]. Classification of actions and notations are as follows:

3.1.1 Permanent actions

Notation	Description
G_0	Self-weight of construction works
G_e	Fittings, ancillaries and fixed equipment
G_{sr}	Reaction of the soil to the permanent actions (including effects of soil settlements)
G_s	Permanent value of lateral thrust of soil
G_w	Permanent value of pressure due to water table
G_p	Permanent value of overpressure or depression
G_T	Permanent value of temperature into buildings
G_{sh}	Shrinkage
G_c	Creep
$P_{sup,0}$	Upper characteristic value of the prestressing action calculated at the postulated end of life of the structures
$P_{f,inf}$	Lower characteristic value of the prestressing action calculated at the initial time

3.1.2 Variable actions

Notation	Description
Q_c	Actions applied during construction
Q_e	Variable loads due to equipment
Q_{VDEII}	Loads from the Tokamak machine in case of VDE II
$Q_{\mathbf{mfield}}$*	Action of the magnetic field on buildings structures
Q_D	Dynamic loads due to equipment
Q_{car}	Variable loads due to carriages
Q_o	Normal variable loads due to occupancy and light equipment
Q_p	Normal overpressure or depression during operation or shut-down
Q_{test}	Test variable action
Q_s	Variation of actions due to lateral thrust of soil from their permanent values
Q_{WT10}	Actions due to ground water table level with a return period of 10 years
Q_{WT30}	Actions due to ground water table level with a return period of 30 years
Q_{WT50}	Actions due to ground water table level with a return period of 50 years
T	Internal or external variation of temperature
W	Wind
S	Snow

*Q_{mfield} – See section 7.1.3 of [3] for an explanation of the treatment of these loads

Actions due to the equipment are further divided in:

- Quasi-permanent values $Q_{qp} = \psi_2\ Q_k$: it applies to equipment (or parts of equipment) which is nearly permanently in place (e.g. duration of occurrence $\geq$ 50% $\times$ life time), except during relatively short periods of time (e.g. for maintenance).
- Frequent values $Q_f = \psi_1\ Q_k$: it applies to load values due to equipment which are reached rather frequently (duration of occurrence $\geq$ 1% $\times$ life time).
- Rare values $Q_r = \psi_0\ Q_k$: Load values which are less frequent than above are classified as rare.

3.1.3 Seismic actions

Notation	**Description** (as defined in section 5.2 of [3])
E_{OB}	Operating basis earthquake action (i.e. SL-1)
E_{SMHV}	Maximum historical earthquake action (i.e. SMHV)
E_D	Design earthquake action (i.e. SL-2)
E_{DE}	Design Extension Earthquake (i.e. SL-3)

The live load case from equipment in seismic conditions is defined as follows:

$$Q_{L.seism} = Q_{qp} = \psi_{2,seism}\ Q_k$$

Where $\psi_{2,seism}$ combination coefficients shall vary with the seismic action considered (SL-1, SL-2 and SL-3) as follows:

- $\psi_{2,SL-1}$ = 0.7 for SL-1 conditions
- $\psi_{2,SL-2}$ = 0.5 for SL-2 conditions
- $\psi_{2,SL-3}$ = 0.3 for SL-3 conditions

3.1.4 Accidental actions

Notation	**Description**
A_p	Accidental pressure due to Loca and Lova
A_{Ti}	Thermal effect due to Loca and Lova
$A_{VDEIII/VDEIV}$	Action from the Tokamak machine in case of VDE III / VDE IV
A_L	Accidental action due to load drop
A_f	Accidental internal flooding
A_{eexp}	Accidental action due to external explosion
A_{iexp}	Accidental action due to internal explosion
A_{APC}	Accidental action due to airplane crash
A_e	Other accidental action due to equipments
A_w	Accidental level of water table
A_s	Exceptional value of snow
A_T	Extreme climatic temperatures

3.2 Calculation methods

This section defines, for some design actions, acceptable calculation methods to simulate their effects on structures and verifications to be performed for the design.

3.2.1 Design Earthquake (E_D)

Principles for the design of buildings to seismic action are given in the guideline ASN/GUIDE/2/01 [12].

Seismic analysis shall be performed using representative methods, with an adequate degree of reliance for such structures. Acceptable methods are given in appendix A.

3.2.2 Loads drops (A_L)

Appendix B gives an acceptable procedure for the analysis of the resistance of structural members under load drops. Alternative methods (for instance explicit non linear dynamic calculations) may be used, with accurate justifications.

3.2.3 External explosion (A_{eexpl})

To calculate the actions on structures, it shall be considered that the wave may come from any horizontal direction, and reflexions and focalisations shall be taken into account. The effects of reflections and focalisations may generally be taken into account by the way of a global pondering coefficient to be applied on the incident pressure wave. Unless specific justifications, the coefficient shall not be less than 2 for vertical wall in front of wave, and 1.5 for roofs.

For the analysis of the effects of this action (A_{eexp}), suitable constitutive models including non linear behaviours shall be used. An acceptable procedure to determine a static equivalent pressure is given in Appendix D.

3.2.4 Airplane crash (A_{APC})

The verification shall include at least the stability of the building and the resistance of the impacted roofs, walls and floors.

The effect of vibrations induced by the impact shall also be analysed.

For the analysis of effects, the impact shall be considered in the centre of slabs and in the borders, to take in account more important risks of shear failure.

For the design against the engine hard impact, protection against perforation of slabs shall be considered. The verification may be done according to the procedure given in Appendix C for perforation, with a minimum concrete thickness of 0.4m and a minimum reinforcement ratio of 100 kg/m^3.

The analysis under airplane global soft impact may be done with the 2 following methods:

- non linear dynamic simplified model, according to the method given in appendix B,
- equivalent static forces, according to the method given in appendix D (in this case, the static force shall be spread uniformly over the same area as the impact area defined for each plane).

Alternative methods (for instance explicit non linear dynamic calculations) may be used, with accurate justifications

3.2.5 Internal explosion (A_{iexpl})

For the analysis of the effects of this action (A_{iexp}), suitable constitutive models including non linear behaviours shall be used. An acceptable procedure to determine a static equivalent pressure is given in Appendix D.

3.3 Definition of limit states

Limit states are states beyond which the structure no longer satisfies the design performance requirements (see EC0, 1.5.2.12). Limit states are classified into:

- static equilibrium limit states (EQU),
- ultimate limit states (ULS),
- serviceability limit states (SLS).

Limit states are further classified according to Table 1.

EQU	Static equilibrium limit state (all situations)
ULS.f	Fundamental ULS (persistent or transient situations)
ULS.a	Accidental ULS (accidental situations)
ULS.e	Seismic ULS (seismic situation)
SLS.c	Characteristic SLS
SLS.f	Frequent SLS
SLS.qp	Quasi-permanent SLS

Table 1 : List of limit states

Static equilibrium limit states correspond to the loss of equilibrium of the structure or any part of it, considered as a rigid body. When considering a limit state of static equilibrium, it shall be verified that:

$$E_{d,dst} < E_{d,stb}$$

where $E_{d,dst}$ and $E_{d,stb}$ are respectively the design effects of destabilising and stabilising actions (see EC0, 6.4.2).

Ultimate limit states are those associated with collapse or other forms of structural failure. When considering an ultimate limit state, it shall be verified that:

$$E_d < R_d$$

where E_d is the design value of the effect of an action (e.g. internal force or moment) and R_d is the corresponding design resistance, associating all structural properties with the respective design value (see EC0, 6.4.2).

At the ultimate limit states, three types of situations are considered:

- persistent and transient design situation (fundamental), with normal partial safety factors applied to the actions (fundamental combination → ULS.f),
- accidental design situation, with reduced partial safety factors applied to the actions (accidental combination → ULS.a),
- seismic design situation (seismic combination → ULS.e).

Serviceability limit states correspond to states beyond which specified service requirements are no longer met. They are in no case related to safety requirements. Serviceability limit states which may require consideration include:

- deformations or deflections which affect the effective use of the structure (including the malfunction of machines or services) or cause damage to finishes or non-structural elements,
- cracking of the concrete which is likely to affect durability or leak tightness adversely, damaging of concrete in the presence of excessive compression which is likely to lead to loss of durability (see EC0, 3.4).

The verifications required at the different limit states are associated to load combinations described in next section.

3.4 Combinations of actions

The main combinations of actions relate to the following situations:

- Combinations relative to construction (1)
- Combinations relative to normal operation (2)
- Combinations relative to test conditions and maintenance (3)
- Combinations relative to Design Earthquake (4)
- Combinations relative to LOCA and LOVA events (5)
- Combinations relative to loads drops (6)
- Combinations relative to airplane crash (7)
- Combinations relative to external explosion (8)
- Combinations relative to internal explosion (9)
- Combinations relative to internal flooding (10)
- Combinations relative to exceptional snow (11)
- Combinations relative to accidental level of table water (12)
- Combinations relative to accidental actions due to equipment (13)
- Combinations relative to extreme climatic temperatures (14)

Combinations of actions are defined in matrix 1 for normal operation and maintenance situations and in matrix 2 for accidental situations, with specific variable actions combinations given in matrix 3.

Combinations of actions related to construction (1) shall be defined taking into consideration the construction phases, temporary loads, as given in load drawings, and a 5 years return period of climatic conditions appropriate to the phases of construction considered.

Any variable or accidental action which has a favourable effect on a given effect of actions shall be taken as 0 in combinations, additional to those considered in the matrices.

In the matrices, G collectively represents all permanent actions, as applicable for the structure or part of the structure considered. It includes notably G_0, G_e, G_{sr}, G_s, G_w, G_p, G_T, G_{sh}, G_c as defined in § 3.1.1.

In the matrices, Q collectively represents variable actions, as applicable for the structure or part of the structure considered. It includes notably Q_e, Q_D, Q_{car}, Q_0, Q_p, Q_{test}, Q_s, Q_w, as defined in § 3.1.2. Actions or effects of actions that can not exist simultaneously due to physical or functional reasons shall not be considered together in combinations of actions. In other cases, variable actions shall be considered simultaneously. In matrix 1, the characteristic value Q_k or the representative values (Q_{qp}, Q_f or Q_{comb}) are used. Only Q_{qp} appears in matrix 2.

In the fundamental load combination, when the basic action is a variable one, the load factor can be reduced from 1.5 to 1.35, if the variable action, Q, is well known in practice and is limited in intensity. For example, this is the case for the weight of a fluid in a tank or a fluid pressure if the maximal level is limited. Also, this is the case of specific parts of equipment the weight of which is well determined and which are located for maintenance in specific areas. The loads associated to this specific equipment are taken into account in combinations 3.1 relative to maintenance with a load factor equal to 1.35. Other equipment loads are considered in the same combination with a load factor equal to 1.5.

In the matrices, (W; S) means W and/or S. In general, W and S follow the same combination rules than variable actions.

In the matrices, T represents the variations of temperature, as applicable for the structure or part of the structure considered.

In accidental combinations, accompanying variable actions will be taken at their quasi-permanent values, except for accidental fire, where frequent values will be taken into account for the main variable action.

Q_{test} is not considered in accidental situations.

The action of ground water table level with a return period of 10 years, Q_{WT10}, is considered as quasi-permanent and shall be combined with the Design Earthquake.

MATRIX 1: NORMAL OPERATION

N°	Situation	Limit State	PERMANENT ACTIONS			VARIABLE ACTIONS				**Verification criteria** for RC structures	**Verification criteria** for steel structures
			$G_{k,}$ Unfav	$G_{k,}$ Fav	P	Q	T	W ; S	Other actions		
2.1	Normal operation	EQU	1.1	0.9	1	1.5 Q_k	0.9	0.9(W+S)		§ 4.5 and § 4.6	§ 5.3
2.2		EQU	1.1	0.9	1	1.5 Q_r	1.5	0.9(W+S)			
2.3		EQU	1.1	0.9	1	1.5 Q_r	0.9	1.5 W + 0.9 S 0.9 W + 1.5 S			
2.4		ULS.f	1.35	1	1	1.5 Q_k	0.9	0.9(W+S)			
2.5		ULS.f	1.35	1	1	1.5 Q_r	1.5	0.9(W+S)			
2.6		ULS.f	1.35	1	1	1.5 Q_r	0.9	1.5 W + 0.9 S 0.9 W + 1.5 S			
2.7		SLS.c	1	1	1	Q_k	0.6	0.6(W+S)			
2.8		SLS.c	1	1	1	Q_r	1	0.6(W+S)			
2.9		SLS.c	1	1	1	Q_r	0.6	(W+S)			
2.10		SLS.f	1	1	1	Q_f	0.2				
2.11		SLS.f	1	1	1	Q_{qp}	0.5				
2.12		SLS.f	1	1	1	Q_{qp}	0.2	0.2(W+S)			
2.13		SLS.qp	1	1	1	Q_{qp}	0.2				
3.1	Maintenance or test	ULS.f	1.35	1	1	1.35 or 1.5 Q_k		0.2(W+S)			
3.2		ULS.f	1.35	1	1	1.5 Q_r	0.5	0.2(W+S)	1.5 Q_{test}		
3.3		SLS.c	1	1	1	Q_r	1	0.6(W+S)	Q_{test}		
3.4		SLS. f	1	1	1	Q_r	0.2				

MATRIX 2 : SEISMIC AND ACCIDENTAL SITUATIONS

N°	Situation	Limit State	PERMANENT ACTIONS			VARIABLE ACTIONS			ACCIDENTAL ACTIONS	**Verification criteria** for concrete structures	**Verification criteria** for steel structures
			$G_{k,}$ Unfav	$G_{k,}$ Fav	P	Q_{qp}	T_{qp}	Other actions			
4.1	Design earthquake	ULS.e	1	1	1	1	1		E_d	§ 4.5, § 4.6.1 & § 4.7	§ 5.3
4.2	Design Extension Earthquake**	ULS.e	1	1	1	1	1		E_{DE}	§ 4.5, § 4.6.1 & § 4.7	
5.1	Accidental overpressure	ULS.a	1	1	1	1	1		A_P	§ 4.5 and § 4.6.1	
5.2		ULS.a	1	1	1	1	1*	A_{Ti}			
6	Accidental internal temperature	ULS.a	1	1	1	1	1*		A_{Ti}	§ 4.5 and § 4.6.1	
7	Loads drops	ULS a	1	1	1	1	0.5		A_L	§ 4.5 and § 4.8.2	
8	Air plane crash	ULS.a	1	1	1	1	0.5		A_{apc}	§ 4.5 and § 4.8.3	
9	External explosion	ULS.a	1	1	1	1	1		A_{eexp}	§ 4.5 and § 4.8.4	
10	Internal explosion	ULS.a	1	1	1	1	1		A_{iexp}	§ 4.5 and § 4.8.4	
11	Internal flooding	ULS.a	1	1	1	1	1		A_f	§ 4.5 and § 4.8.5	
12	Exceptional snow	ULS.a	1	1	1	1	1		A_s	§ 4.5	
13	Accidental level of water table	ULS.a	1	1	1	1	1		A_w	§ 4.5	
14	Other accidental action due to equipments	ULS.a	1	1	1	1	1		A_e	§ 4.5	
15	Extreme climatic temperatures	ULS.a	1	1	1	1	1*		A_T	§ 4.5	

* Quasi-permanent temperature T_{qp} is applied to elements where the accidental temperature is not applied.
** These combinations are only applicable to specific elements of these structures, as defined in Section 5 of [2]

MATRIX 3.1 : COMBINATION SITUATIONS*

In Normal combination situations (Category I and II) the design is required to demonstrate at least 20% margin when considering the support of the Cryostat.

In Accidental combination situations (Category III and IV) the design is required to demonstrate at least 50% margin when considering the support of the Cryostat.

		Category of Combination	Combination Number	Actions to be considered coincidentally	Buildings that must consider the combinations
Combinations of actions to be	NORMAL	Cat. I	Combination 1	MFD I + MD I	11,14,74,19
		Cat. II	Combination 2	MD II + VV ICE II	11,14,74,19
			Combination 3	VDE II + VV ICE II	11,14,74,19
			Combination 4	MFD I + MD II	11,14,74,19
			Combination 5	MFD I + VDE II	11,14,74,19
			Combination 6	MFD II + MD I	11,14,74,19
			Combination 7	VV ICE II + MD I or II	11,14,74,19
			Combination 8	Cr ICE II + MFD I or II	11,14,74,19
			Combination 9	SL-1 + MD 1	11,14,74,19
			Combination 10	SL-1 + MFD I or II	11,14,74,19
			Combination 11	SL-1 + baking	11,14,74,19
	ACCIDENTAL	Cat. III	Combination 12	MD II + VV ICE III	11,14,74,19
			Combination 13	VDE II + VV ICE III	11,14,74,19
			Combination 14	MD III + VV ICE II or III	11,14,74,19
			Combination 15	VDE III + VV ICE II or III	11,14,74,19
			Combination 16	VDE III + MFD I	11,14,74,19
			Combination 17	MFD II + Cr LOVA III	11,14,74,19
			Combination 18	VV ICE III + MD III	11,14,74,19
			Combination 19	VV ICE II + MD III	11,14,74
			Combination 20	VV LOVA III + MD III	11,14,74
			Combination 21	Cr LOVA III + MFD I or II	11,14,74,19
			Combination 22	SL-1 + MFD II + MD II	11,14,74,19
			Combination 23	SL-1 + MFD II + VDE II	11,14,74,19
			Combination 24	SMHV + Cr ICE III	11
			Combination 25	SMHV + Helium leak in galleries	11,14,74,19
			Combination 26	SMHV + Cr ICE III + MFD I or II	11,14,74,19

*These combinations are to be considered in conjunction with the appropriate permanent or quasi-permanent actions. For Category I and II combinations, the appropriate variable actions factors from Matrix 1 of this report should be applied. For Category III and IV combinations, the appropriate factors shall be taken from Matrix 2 of this report.

MATRIX 3.2 : COMBINATION SITUATIONS (continued)*

In Accidental combination situations (Category III and IV) the design is required to demonstrate at least 50% margin when considering the support of the Cryostat and in Beyond Design Basis combination situations (Category V) the design is required to demonstrate at least 10% margin when considering the support of the Cryostat.

In Design Extension situations (Category V involving SL-3), no margin is requested on the loads.

		Category of Combination	Combination Number	Actions to be considered coincidentally	Buildings that must consider the combinations
Combinations of actions to be considered	ACCIDENTAL	Cat. IV	Combination 27	MD IV + VV ICE III or IV	11,14,74,19
			Combination 28	VDE IV + VV ICE III or IV	11,14,74,19
			Combination 29	MD III + MFD I	11,14,74,19
			Combination 30	VV ICE IV + MD I or II or III	11,14,74,19
			Combination 31	Cr ICE IV + MFD I or II	11,14,74,19
			Combination 32	SL-1 + MD III	11,14,74,19
			Combination 33	SL-2 + Cr ICE III	11,14,74,19
			Combination 34	SL-2 followed by internal fire followed by SMHV	11,14 & 74- see [200] for specific rooms to consider 12,19,21,23,24,13, 42-43,44-45,61-00-ST(SR only),61-00-PB,22,18,20,K2,K3
			Combination 35	SL-2 + Helium leak in Galleries	11
			Combination 36	SL-2 + Baking	11,14,74,19
			Combination 37	LOCA PC (X8) III + VV ICE II	11,14,74
			Combination 38	Internal explosion + Internal fire	11,14,74,21,23
			Combination 39	Internal fire + internal flooding	11,14,74,21,23,24, 71,13,15,37,42-43,44-45,75,61-00-ST,61-00-PB,46-47,22,18,20,K2,K3
			Combination 40	LOCA + Internal flooding	11,14,74
			Combination 41	Aircraft impact + external fire	11,14,74,21,23
			Combination 42	Water table + external flooding	12,19,21,23,24,71, 13,15,37,42-43,44-45,75,61-00-ST, 22
			Combination 43	External explosion + External fire	11,14,74,21,23,24, 71,13,15,37,42-43,44-45,75,61-00-ST,61-00-PB,46-47, 22,18,20,K2,K3
			Combination 44	Accidental temperature + External fire	11,14,74,21,23,24, 71,13,15,37,42-43,44-45,75,61-00-ST,61-00-PB,46-47, 22,18,20,K2,K3
			Combination 45	Internal explosion + Drop load	11,14,74,21,23
			Combination 46	External fire + External explosion	11,14,74,21,23,24, 71

		Combination 47	Internal explosion + Internal flooding	**11,14,74,21,23**
		Combination 48	Internal fire + Internal explosion	**11,14,74,21,23,24, 13,37,42-43,44-45,75,61-00-ST,61-00-PB,46-47, 22,18,20,K2,K3**
		Combination 49	Damaged equipment (pipewhip) + Internal flooding	**11,14,74,21,23**
BEYOND DESIGN BASIS	**Cat. V**	**Combination 50**	MFD II + VDE IV	11,14,74,19
		Combination 51	Cr LOVA III + MFD II + VDE II	11,14,74,19
		Combination 52	VDE IV + VV ICE IV	11,14,74,19
		Combination 53	Cr ICE IV + MFD II + VDE II	11,14,74,19
		Combination 54	VDE IV + MFD I	11,14,74,19
		Combination 55	SL-2 + Cr ICE IV + HIG III + MFD II + VDE II	11,14,74,19
Design Extension	**Cat V**	**Combination 56**	SL-3 + LOCA#	11**,14**,74**,19**
		Combination 57	SL-3 + Helium Leak in Galleries#	11**,14**,74**,19**
		Combination 58	SL-3 + LOCA# + Helium Leak in Galleries#	11**,14**,74**,19**

*These combinations are to be considered in conjunction with the appropriate permanent or quasi-permanent actions. For Category IV and V combinations, the appropriate factors shall be taken from Matrix 2 of this report. In CAT V, the justification of the resistance of the basemat B2 (2nd confinement barrier) must be also provided where the confinement requirement shall be maintained for the combinations given in Matrix 3.2 (above). The design criteria define in the section 4.6.1 of this document shall be considered in the framework of this justification.

These loads (LOCA and HIG) are special cases defined for combination with SL-3 in the Design Extension Combinations

** These combinations are only applicable to specific elements of these structures, as defined in Section 5 of [2]

Combinations with significant thermal effects:
Several of the above combinations include load cases with significant thermal effects. In certain circumstances, these are to be combined with other load cases that occur over a short duration and only when the machine is in operation (e.g. VDE's). In such cases, it may not necessary to consider the worst load effects of the thermal effects (which take many hours to develop) in conjunction with the short duration dynamic effects of a VDE. The non-concomitance of loading conditions (such as the case highlighted above) should be considered on a case by case basis, and then approval sought from IO.

4 General rules for concrete structures

4.1 References and general assumptions

The design of the concrete structure is made in accordance with EN 1992-1-1: Eurocode 2 "Design of Concrete Structure", Part 1 "General Rules and Rules for Buildings", with the specific additional rules defined hereafter which complete or modify the corresponding specifications of EN 1992-1-1.

Whenever no alternative choice is specified below, the requirements of the French National Annex NF EN 1992-1-1_NA apply.

Partial factors for concrete and steel in design seismic situations shall be those of accidental situations.

4.2 Material properties

The characteristic values of material properties given below shall be used unless otherwise specified.

4.2.1 Concrete

Design rules are based solely on the characteristic 28 day strength, f_{ck}, of a cylinder. Classes of concrete to be used are given in Table 2.

Concrete classes	$f_{ck,\ cylinder}$
C30/37	30 MPa
C40/50	40 MPa
C60/75	60 MPa

Table 2 : Compressive strength of different classes of concrete

4.2.2 Reinforcing steel

The reinforcing steel bars are defined by their grade (specified characteristic yield stress) and their cross-sectional area. The reinforcing steel bars shall comply with Euronorm EN 10080. Reinforcing steel of class B or C in EN 1992-1-1:2004, Table C.1 shall be used

Only ribbed bars shall be used for structural purpose, with the following characteristic value for the basic design: f_{yk} = 500 MPa.

Only high ductility rebars with $\varepsilon_{uk} \geq 5$ % shall be used.

Special reinforcements like stainless steel or non metallic reinforcement are not covered by this design code. In case of use of special reinforcements, specific design rules shall be specified.

4.2.3 Prestressing steel

Steel for prestressing tendons are defined by their grade which defines the tensile strength f_{pk} and the 0.1% proof stress $f_{p0.1k}$ and their cross-sectional area.
It shall comply with Euronorm EN 10138.

4.3 Durability

The exposure class shall be:

- for external walls and slabs : XC4;
- for internal structures : XC1 (nota : if they may be in permanent contact with humid air, they shall be classified as XC3);
- for underground structures in contact with humid soil : XC2 + XA1.

Structural class shall be determined accordingly, in taking into account for this determination a design life time of 100 years, but shall not be lower than S5.

4.4 Design data

4.4.1 Concrete

- ❑ Design tolerance for cast in situ concrete: Δc_{dev} = 10 mm
- ❑ Volumetric mass: ρ = 2500 kg/m³
- ❑ Poisson's ratio: ν = 0.2 (0 for cracked concrete)
- ❑ Young's modulus: For prestressed concrete:
  - Young's modulus for short term actions (transient pressure, dynamic effects, seismic…) is given in EN1992-1-1, 3.1.3,
  - For long term effects: $E_{cm,d} = \dfrac{E_{cm}}{1+\varphi_d(t_f,t_0)}$, where $\varphi_d(t_f,t_0)$ is the dry creep coefficient, as defined in Annex B of EN1992-2, with t_f the design life time and t_0 = 1 year,
  - For thermal effects lasting less than 24 hours: $E_{cm,th} = \dfrac{E_{cm}+2E_{cm,d}}{3}$,
  - For the calculation of $P_{f,inf}$ at the end of the life time, to estimate deformations due to creep, it may be assumed that the Young's modulus of concrete E_c is 20% lower than the design value, unless actual concrete properties can be better assessed.

For reinforced concrete structures, above formula are applicable, but approximate values given in Table 3 may also be used (values calculated for φ_d = 2).

Concrete class	C30/37	C40/50	C60/75
f_{ck}	30 MPa	40 MPa	60 MPa
For the analysis of dynamic or short term actions	33 000 MPa	35 000 MPa	39 000 MPa
For the analysis of transient and accidental thermal actions	18 000 MPa	20 000 MPa	22 000 MPa
For the analysis of long term actions and permanent thermal actions	11 000 MPa	12 000 MPa	13 000 MPa

Table 3 : Approximate values of Young's modulus of different classes of concrete

- Shrinkage: Shrinkage strains are calculated:
    - according to EN 1992-1-1, 3.1.4 for concrete up to grade C50/60 included and for non massive members for which the notional size h_o is less than or equal to 1.20m,
    - according to EN1992-2, Appendix B, either for concrete of grade above C50/60 or for massive slabs or walls, the notional size h_o of which is above 1.20m.

The mean relative humidity of the ambient air RH is defined in the systems requirements documents (SRD) for buildings.

- Thermal properties :
    - thermal expansion coefficient: $\alpha_C = 10 \times 10^{-6}$ /°C
    - heat capacity: C = 1000 J/kg °C
    - conductivity: λ = 2.0 W/m °C

4.4.2 Reinforcing steel

- Young's modulus: E_S = 200 GPa
- Thermal expansion coefficient: $\alpha_S = 10 \times 10^{-6}$/°C

For the analysis of accidental situations, steel properties may be assumed at constant values independent from temperature between 0 and 200 °C.

4.4.3 Prestressing steel

Young's modulus: E_p = 190 GPa for strands
E_p = 200 GPa for bars

4.4.4 Thermal and shrinkage effects

For the application of EN1992-1-1, 5.4, for the calculation of thermal and shrinkage effects, the stiffness reduction due to concrete cracking has to be taken into account. The stiffness of the cracked zones of concrete elements with 30 MPa $\leq f_{c28} \leq$ 60 MPa and a total longitudinal (for bending) reinforcement ratio $\rho_l \leq 0.01$ may be reduced by the coefficient below:

- 0.7 for SLS combinations,
- 0.5 for ULS combinations.

However, different values may be adopted with accurate justifications or non linear analyses may be performed taking into account a representative model of cracking.

4.5 Verification rules

Design values of properties and partial factors are defined in EN 1992-1-1 and EN 1998-1 and their National Annexes, unless defined in the present document.

Verification rules and criteria are described and identified below, either at ULS or SLS, with an identification rule number (e.g. "ULS.R2"). For each structure and each combination of action, a group of criteria based on a combination of rules shall be verified. These groups of criteria are defined in § 4.5.3. It is important to note that several criteria or groups of criteria may be applied to a given structural element, according to the various roles it plays.

4.5.1 Ultimate Limit States

4.5.1.1 ULS.R1: general verifications at ULS

Applicable to ULS.f, ULS.e and ULS.a.
EN1992-1-1, 6.1 to 6.6 apply, in particular:

- The design of members with shear reinforcement is based on a truss model. EN 1992-1-1, 6.2 applies.
- Shear at the interface between concrete casts is verified according to EN1992-1-1, 6.2.5 (1) to (5).
- For punching, EN1992-1-1, 6.4 applies. A particular attention is drawn to figure 6.12 which gives an appropriate verification model for punching. The shear resistance shall be checked along defined control perimeters.

4.5.1.2 ULS.R2: Bending with or without axial force

Design strain of reinforcement shall be limited to 10‰ in the followings cases:
- the concrete member supports the anchorage of a SIC component;
- the concrete member belongs to the second confinement barrier in the Tokamak building, or the slabs and walls of the Hot Cells rooms.

4.5.1.3 ULS.R3: Bending in accidental conditions

Limitations in design material properties:

▪ Concrete : $\varepsilon_{cu} = 2‰$ $\varepsilon_{cu2} = 3.5‰$

▪ Steel : $\varepsilon_{ud} = 0.9\ \varepsilon_{uk} = 4.5\%$

4.5.1.4 ULS.R4: Shear

The design of members with shear reinforcement is based on a truss model. EN 1992-1-1, 6.2.3 is modified as follows:

✓ Notations are as follows (see EN 1992-1-1, 6.2.3, Figure 6.5) :

- α is the angle between the shear reinforcement and the beam axis, perpendicular to the shear force (measured positive as shown in Figure 6.5),
- θ is the angle between the concrete compression strut and the beam axis perpendicular to the shear force,
- F_{td} is the design value of the tensile force in the longitudinal reinforcement,
- F_{cd} is the design value of the concrete compression force in the direction of the longitudinal member axis,
- b_w is the lower value of width between tension and compression chords,
- z is the inner lever arm, for a member with constant depth, corresponding to the maximum bending moment in the element under consideration. In the shear analysis of reinforced concrete without axial force, the approximate value $z = 0.9d$ may normally be used.

✓ The inclination of compressive struts shall verify the following equation:
$1 \leq \cot\theta \leq \cot\theta_0$

The maximum permitted value for $\cot\theta$ follows the expression:

- if a mean compressive stress σ_{cp} is applied ($\sigma_{cp} > 0$):

$$\cot \theta_0 = 1.2 + 0.2 \frac{\sigma cp}{fctm}$$

- if a mean tensile stress σ_{cp} is applied ($\sigma_{cp} < 0$):

$$\cot \theta_0 = 1.2 + 0.9 \frac{\sigma cp}{fctm} \geq 1$$

✓ When the value cot θ_0 is chosen for cot θ, the shear resistance shall be taken as the lesser of:

$$V_{Rd} = \frac{Asw}{s} \cdot z \cdot f_{ywd} \cdot (\cot \theta_0 + \cot \alpha) \sin \alpha + V_{fd}$$

and

$$V_{Rd\,\max} = \alpha_{cw} \cdot v_1 \cdot b_w z \cdot f_{cd} \cdot \frac{\cot \theta_0 + \cot \alpha}{1 + \cot^2 \theta_0}$$

where:

- A_{sw} is the cross-sectional area of the shear reinforcement,
- s is the spacing of the stirrups,
- f_{ywd} is the design yield strength of the shear reinforcement,
- v_1 is a strength reduction factor for concrete cracked in shear; v_1 take the following values (EN 1992-1-1, 6.2.3 (3)):
  - for $f_{ck} \leq 60$ MPa: $v_1 = 0.6$,
  - for $f_{ck} \geq 60$ MPa: $v_1 = 0.9 - f_{ck}/200 > 0.5$.
- α_{cw} is a factor taking into account the state of stresses in the compression chord. The factor α_{cw} is defined as follows, with σ_{cp} defined in EN 1992-1-1, 6.2.3 (3):
  - If $0 < \sigma_{cp} < 0.25 f_{cd}$: $\alpha_{cw} = 1 + \frac{\sigma_{cp}}{f_{cd}}$
  - If $0.25 f_{cd} < \sigma_{cp} < 0.5 f_{cd}$: $\alpha_{cw} = 1.25$
  - If $0.5 f_{cd} < \sigma_{cp} < f_{cd}$: $\alpha_{cw} = 2.5 \left(1 - \frac{\sigma_{cp}}{f_{cd}}\right)$
- V_{fd} is the contribution of concrete to shear resistance. It is taken as :
  - If a mean compressive stress σ_{cp} is applied ($\sigma_{cp} \geq 0$) :

$$V_{fd} = 0.068 \cdot b_w z \cdot \left(1 - \frac{\cot \theta_0}{4}\right) f_{cd}$$

  - If a mean tensile stress σ_{cp} is applied ($\sigma_{cp} < 0$) :

$$V_{fd} = 0.068 \cdot b_w z \cdot \left(1 - \frac{0.36}{\cot \theta_0}\right) f_{cd}$$

✓ When values other than cot θ_0 are chosen for cot θ, the shear resistance shall be taken as the lesser of (EN 1992-1-1, 6.2.3 (4)):

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot (\cot \theta + \cot \alpha) \sin \alpha$$

and

$$V_{Rd,max} = \alpha_{cw} \cdot \nu_1 \cdot b_w z \cdot f_{cd} \cdot \frac{\cot\theta + \cot\alpha}{1 + \cot^2\theta}$$

The contribution of concrete to shear resistance V_{fd} is neglected.

To ensure a ductile failure the following expression shall also be satisfied:

$$\frac{A_{sw,prov} f_{ywd}}{b_w s} \leq \frac{\alpha_{cw} \cdot \nu_1 \cdot f_{cd}}{2} \cdot \frac{\sin\alpha}{1 - \cos\alpha}$$

4.5.1.5 ULS.R5 : Punching in accidental conditions

EN1992-1-1, 6.4 applies. A particular attention is given to figure 6.12 which gives an appropriate verification model for punching. The shear resistance shall be checked along defined control perimeters. For calculation of v_{Rdc}, the value of v_{min} shall be equal to $0.053\ k^{3/2} f_{ck}^{1/2} / \gamma_C$.

When punching is due to hard impact, appendix C is applicable, and clauses C.3 and C.4 give acceptable criteria.

4.5.1.6 ULS.R6: Local verification for impact of general aviation plane

Limitations in design material properties:

- Concrete : $\sigma_c = f_{ck}$ $\varepsilon_{cu2} = 5‰$
- Steel : $\varepsilon_{ud} = 0.9\ \varepsilon_{uk} = 4.5\%$

These limitations substitute to those of EN 1992-1-1, Sect. 6, for the local verification. However, different values of limit strains may be used (see appendix B) to account for uncertainties due to simplified analysis.

For concrete structures that protect SIC components, in order to prevent secondary missiles against SIC, the maximal steel tensile strain in bending should be limited to : $\varepsilon_s = 2.0\ \%$.

Appendices B and D define two alternative acceptable procedures for the analysis of stability and resistance. Appendix C gives an acceptable method to design the members against perforation due to hard impacts (engines).

For the punching verification (appendix B) the stirrup rupture is avoided and the elongation in the stirrups is limited to the admissible strain ε_{ud}.

For the evaluation of ultimate displacements, the calculations are performed with the following admissible rotation θ_{lim} in the plastic hinge:

$$\theta_{\lim} = Min\left\{0.025; \frac{0.005}{x/d}\right\}$$

with d the effective depth of the cross-section of the wall and x the neutral axis depth.

4.5.1.7 ULS.R7: Local verification for load drops

Limitations in design material properties:

- Concrete : $\sigma_c = f_{ck}$ $\varepsilon_{cu2} = 5‰$
- Steel : $\varepsilon_{ud} = 0.9\ \varepsilon_{uk} = 4.5\%$

These limitations substitute to those of EN 1992-1-1, Sect. 6, for the local verification. However, different values of limit strains may be used (see appendix B) to account for uncertainties due to simplified analysis.

For concrete structures that protect SIC components, in order to prevent secondary missiles against SIC, the maximal steel tensile strain in bending should be limited to : ε_s = 2.0 %.

Appendix B gives an acceptable procedure for the analysis of structural members resistance under load drops.

4.5.2 Serviceability Limit States

The serviceability requirements given in EN 1992-1-1, Sect. 7, in terms of stress limitation, crack control and deflection control, intend to avoid a reduction of the durability, under quasi permanent situations for reinforced concrete structures and frequent situations for prestressed concrete. These requirements are extended to both situations for reinforced concrete in order to provide a certain level of tightness. Requirements in characteristic situations are also introduced.

For stress analysis in sections submitted to long term effects, the ratio E_s / $E_{cm,d}$ shall be taken equal to 15 in quasi-permanent and frequent situations.

4.5.2.1 SLS.R1: limitation of stresses

The following limitations apply to SLS.qp and SLS.f, for elements where crack widths have to be limited (see § 4.5.3).

- the mean compressive stress in the concrete is limited to 0.45 f_{ck},
- the maximal compressive stress in the concrete is limited to 0.6 f_{ck},
- the tensile stress in the longitudinal rebars is limited to:

$$\sigma_y = Min\left\{\frac{2}{3} f_{yk} ; Max\left(0.5 f_{yk} ; 110\sqrt{\eta f_{ctm}}\right)\right\} \text{ in MPa,}$$

with η equal to 1.6 for high bond bars.

4.5.2.2 SLS.R2: limitation of stresses

The following limitations apply to SLS.c.

- the maximal compressive stress in the concrete is limited to 0.6 f_{ck},
- the tensile stress in the longitudinal rebars is limited to 0.8 f_{yk}

4.5.2.3 SLS.R3: Minimum reinforcement areas

The minimum areas of reinforcement required in tensile zones of walls and slabs of confinement barrier may be calculated from the following formula:

$$A_s = k_c\ k\ f_{ct,eff}\ A_{ct} / \sigma_s$$

where:

- A_{ct} is the concrete area within the tensile zone; the tensile zone considered is the part of the cross section where tensile behaviour was determined just before formation of the first crack,
- k_c is equal to 1 for pure tension and equal to 0.4 for bending without axial compressive force,
- k is equal to 1 for a wall thickness lower than 300 mm and equal to 0.65 for a thickness higher than 800 mm,

$f_{ct,eff}$ is the mean value of the tensile strength of concrete, effective at the time when the cracks may first be expected to occur:
$f_{ct,eff}$ = f_{ctm} or lower, i.e. $f_{ctm}(t)$ if cracking is expected earlier than 28 days,
σ_s is equal to f_{yk}.
Intermediate values may be interpolated, according to EN 1992-1-1, 7.3.2.

4.5.2.4 SLS.R4: Minimum reinforcement in peripheral walls

At the first level of buildings above raft and above soil level for embedded buildings, a minimum reinforcement complying with rule SLS.R3 shall be provided in the peripheral walls in the horizontal direction with k_c = 1.

4.5.2.5 SLS.R5: Limitation of stress and strain

The following limitations apply to confinement barriers in Design Extension situations Cat V :

- Concrete $f_{cd} = f_{ck} / 1.3$
- Steel $f_{yd} = 1.0\ f_{yk}$ and $\varepsilon_{yd} = 0.25\%$

For other structural elements, criteria are the same as in accidental / seismic design.

4.5.3 Groups of criteria

Groups of criteria are defined in Table 4 below, together with their main cases of application.

	ULS							SLS				Application
Group of criteria	ULS.R1	ULS.R2	ULS.R3	ULS.R4	ULS.R5	ULS.R6	ULS.R7	SLS.R1	SLS.R2	SLS.R3	SLS.R4	
C1	X	X		X					X			All structures
C2	X	X		X				X	X			Structural elements where crack limitation is strictly required in normal situation
C3	X		X	X	X							Accidental situations. See § 4.8
C4	X			X		X						General aviation plane impact. See § 4.8.3.
C5	X	X		X				X	X	X		Confinement barriers. See § 4.6.1.
C6	X	X		X					X		X	Peripheral walls of all buildings
C7	X			X			X					Load drops

Table 4 : Groups of design criteria

4.6 Additional rules for certain specific structures

4.6.1 Confinement barriers

For confinement barriers, the following criteria apply:

- Under SLS.c combinations, the rule SLS.R1 applies.
- Under ULS.a combinations for LOCA / LOVA or Design Earthquake, the rule SLS.R2 applies. In addition, through walls cracks have to be limited with the following additional rule. When the whole section is under tension, the rule SLS.R1 applies.
- Under Design Extension Earthquake seismic action (SL3 level), for structures identified in [2] and attached design extension loads combinations, the rule SLS.R5 applies to confinement barriers. In addition, through slab cracks have to be limited with the following additional rule: when the whole section is in tension, the rule SLS.R1 applies.

In addition, specific care has to be taken during construction stage to avoid cracking due to shrinkage or thermal effects during concrete hardening.

4.6.2 Buried ducts and tunnels

For concrete buried ducts and tunnels which may contain radioactive fluids, SLS.R1 applies in characteristic serviceability limit states combination to limit the maximum design crack width.

4.6.3 Columns supporting the anti-seismic bearings

For the concrete columns supporting the anti-seismic bearings, the following criteria apply:

- Under SLS.c combinations, the rule SLS.R1 applies.
- Under ULS.a combinations for Design Earthquake, the rule SLS.R2 applies.

4.7 Additional rules for earthquake resistance

4.7.1 Primary and secondary structures

For the verification of structural elements in seismic situations, when the structure consists in frames with columns and beams, it may be considered that some of these elements are secondary structural elements (in the sense of EN 1998-1), which only transfer vertical forces and do not participate to the resistance to seismic horizontal forces. Such elements may be neglected in the structural analysis related to the horizontal seismic action and may be designed so as to resist the displacements imposed by the main structure. They shall comply with requirements of EN 1998-1, 5.7.

Those concrete elements which resist the horizontal seismic action are primary elements.

4.7.2 Additional requirements for linear elements (columns and beams)

No brittle failure (shear or bond failure) shall occur before the ultimate capacity in bending is reached. To that aim, primary linear elements (columns, beams and beam-column joints) shall be designed to ductility class M, according to EN 1998-1, including corresponding detailing.

The resistance of the parts of the element submitted to compression is high enough to ensure that the ultimate strength of the element submitted to bending with or without axial force is governed by the reinforcement and not by the concrete.

In the cases where masonry or concrete infill may have local effects on columns, EN 1998-1, 5.9 applies.

4.7.3 Retaining walls of buildings

The design of retaining structures or structural elements shall take into account the dynamic soil pressure and the dynamic water pressure below the water table.

They shall comply with requirements of EN 1997 and EN 1998-5, Sect. 7.

It shall also take into account the effects of the dynamic forces (vertical and horizontal) and moments applied to the foundation of an adjacent building when it is located above the retaining element.

4.7.4 Non structural elements

Non structural elements are elements not taken into account in the structure resistance (for instance partition walls). The requirements for non structural elements are not in scope of the present specification.

When earthquake stability is required, the design seismic action applied to these elements may be evaluated from floor spectra calculated at the interface with those structural elements which support the non structural elements considered. For small non structural elements, in the absence of specific prescriptions, the effects of the design seismic action may be evaluated in applying EN 1998-1, 4.4.5.2, with a behaviour factor of the element and an importance factor equal to 1.

Masonry partitioning inside the buildings shall be avoided. If used, they shall be designed so as not to increase the stiffness of the primary structure (notably in the cases of frames). When earthquake stability is required, they shall, in the absence of specific requirements, comply with requirements of EN 1998-1, 9.2 and 9.5 concerning confined or reinforced masonries with limited ductility. Such partitioning shall not be considered as participating in the resistance to lateral forces. Other partitioning shall at least comply with requirements related to unreinforced masonries and comply with requirements of EN 1998-1, 9.5.

4.8 Additional rules for accidental situations

4.8.1 Rules in case of fire

In the fire situations, the design (including structural arrangement, protection of structural elements, construction rules…) shall comply with requirements of EN 1992-1-2 and EN 1993-1-2 and their National annexes.

4.8.2 Additional rules for loads drops

Group of criteria C7 applies.

4.8.3 Additional rules for air plane crash resistance

Group of criteria C4 applies.

4.8.4 Additional rules for external or internal explosion

Group of criteria C3 applies. For the analysis of effects, see also § 3.2.3.

4.8.5 Additional rules for internal flooding

Rooms where flooding is expected must be leak proof, and withstand water load.

Group of criteria C5 applies, see also § 4.6.1.

4.8.6 Additional rules for LOCA

See § 4.6.1.

4.9 Detailing

Rules of EN 1992-1-1 apply, with the modifications below.

4.9.1 Reinforced concrete walls, slabs and basemats

The area of reinforcement shall not be less than 0.00125 A_c at each face of the wall and in each direction. Alternatively, in the case of non base isolated structures, this minimum area shall not be less than 0.0025 A_c in the horizontal direction when the dimension of the building in the same direction is higher or equal to 25 m. In addition, the minimum area 0.0025 A_c applies to the second confinement barrier in the Tokamak building and to the slabs and walls of the Hot Cells rooms.

In both directions, bar spacing must not exceed 250 mm.

Tying reinforcement shall be placed at the intersections between walls, or between walls and slabs (or basemat). The area of this reinforcement shall not be less than 0.01 A_{ci}, where A_{ci} is the concrete area of the intersection, when A_{ci} is lower than 0.25 m^2. This area shall not be less than 0.005 A_{ci} when A_{ci} is higher than 1 m^2. It shall be linearly interpolated for intermediate values of A_{ci}.

The number of transverse reinforcing bars in walls and slabs shall not be less than 4 per m^2.

4.9.2 Walls and slabs submitted to accidental impacts

The present requirements apply to walls and slabs submitted to aircraft impact or significant load drops. Transverse reinforcing bars (links) shall be provided at each node of the net formed by the longitudinal main reinforcement at each face. The number of transverse reinforcing bars shall not be less than 16 per m². The spacing of stirrups shall not be greater than 250 mm in each direction. The transverse reinforcement ratio shall not be less than 0.0015.

4.9.3 Beams

The minimum reinforcement according to EN 1992-1-1, 9.2.1.1 shall be considered for bottom and top longitudinal reinforcement separately, including the beam-column joint.

The cross sectional areas of the tension reinforcement and of the compression reinforcement shall not be greater than 0.04 A_c other than at overlaps.

Beams shall also comply with requirements of § 4.7.2 for seismic resistance.

4.9.4 Columns

The minimum amount of longitudinal tensile reinforcement shall not be less than:

$$\text{Max}\ \{0.10\ N_{sd} / f_{yk}\ ;\ 0.005\ A_c\ \}$$

with N_{sd} the design axial compression force and A_c the cross- section of the concrete.

The area of reinforcement shall not exceed 0.04 A_c except in laps zones.

Even at the overlaps, the area of reinforcement shall not exceed the upper limit 0.08 A_c.

Spacing between transverse reinforcement shall not exceed 250 mm.

Columns shall also comply with requirements of § 4.7.2 for seismic resistance.

4.9.5 Anchorage and junction of reinforcing bars

Anchorage lengths and lap lengths shall be calculated according to the requirements of EN 1992-1-1, Sect. 8.

In addition, the minimum lengths $l_{b,min}$ and $l_{0,min}$ shall not be less than 100 d_b / $f_{ctk\ 0.05}$ in case of straight bars, and 60 % of this length in case of hooked bars or straight bars with transverse welded bars (only in case of machine welded bars), where d_b is the bar diameter and $f_{ctk\ 0.05}$ the lower characteristic concrete tension resistance (5 % fractile).

Transverse reinforcement calculated according to EN 1992-1-1 is required to balance the transverse tension forces perpendicular to the first layers (on both faces of the walls and slabs) of reinforcement in the lap zones of these layers. However, in case of staggered laps, this tension forces may be decreased by 50% in the calculation.

Confinement transverse reinforcement in beams, columns and walls shall comply with EN 1998-1, 5.6.1 (2)P (made of closed stirrups with 135° hooks and 10 d_{bw} straight length extension).

Mechanical couplers may be used, provided that the ductility of junction is justified by testing.

4.9.6 Joints between buildings

The thickness of joints between buildings shall be compatible with seismic differential displacements as obtained by structural analysis. In addition, the free joint thickness shall not be less than:

- 50 mm for buried levels,
- 100 mm for levels above soil,
- 200 mm for base isolated structures.

If a crane runway crosses a joint, the runway detailed design shall take into account the seismic differential displacements.

5 General rules for steel structures

5.1 References and general assumptions

The design of structural steelworks is made according to NF EN 1993: Eurocode 3 "Design of Steel Structures", and particularly the following parts:

- Part 1-1: General Rules and Rules for Buildings,
- Part 1-3: Cold formed thin gauge members and sheeting,
- Part 1-5: Plated structural elements,
- Part 1-6: Strength and stability of shell structures,
- Part 1-7: Strength and stability of planar plated structures transversely loaded,
- Part 1-8: Design of joints,
- Part 1-9: Fatigue strength of steel structures,
- Part 3: Towers, Masts and Chimneys,
- Part 6: Crane supporting structures.

Specific additional rules are defined hereafter which complete or modify the corresponding specifications of NF EN 1993. Whenever no alternative choice is specified below, the requirements of the French National Annexes apply.

5.2 Material properties

For hot rolled products, the allowed steel grades are those defined as S235, S275 and S355 according to NF EN 10025. For hollow sections, the allowed grades are those defined as S235H, S275H and S355H according to NF EN 10210 and NF EN 10219 standards.

Ductility of bolts shall be compatible with EN 1998-1 requirements.

High elasticity limit is allowed for anchorages, only if minimum ductility requirements for bolts are met.

The volumetric mass of steel is ρ = 7850 kg/m^3.

5.3 Verification rules

5.3.1 Ultimate Limit State

ULS strength verifications of members are carried out according to EN 1993-1-1, Sect. 6, with partial factors defined in NF EN 1993-1-1/NA:

Type of verification	Partial factor γ_M
Resistance of cross-sections	γ_{M0} = 1.00
Resistance of members to instability	γ_{M1} = 1.00
Resistance of cross-sections in tension to fracture	γ_{M2} = 1.25

Table 5 : Partial factors for steel structures

5.3.2 Serviceability Limit State

SLS verifications are carried out according to EN 1993-1-1, Sect. 7 and NF EN 1993-1-1/NA, with the following additional requirements.

a) Deflection of beams except crane runways

The allowable vertical deflections are given in Table 6, unless more severe criteria are required for specific purpose (like supporting special equipments).

Type of structural member	Allowable vertical deflection (*)	
	w_{max}	w_3
Roof elements	L / 200	L / 250
Floor elements	L / 250	L / 350
Floor elements under moving loads		L / 500
Elements supporting columns	L / 400	L / 500
Elements supporting glazed frames		L / 500

Table 6 : Allowable vertical deflections for steel structures

(*) Where:
w_c is the precamber of the unloaded structural member,
w_1 is the initial deflection under permanent loads,
w_2 is the additional long term deflection under permanent loads,
w_3 is the additional deflection under variable loads,
w_{tot} is the total deflection : $w_{tot} = w_1 + w_2 + w_3$
w_{max} is the maximal deflection including precamber : $w_{max} = w_{tot} - w_c$
L is the span length. For cantilever beams, L is equal to twice the cantilever length.

The allowable horizontal deflections are given in the following table, unless more severe criteria are required for specific purpose (like supporting special equipments).

Type of structural member	Allowable horizontal deflection
Elements supporting cladding (girt or column)	L / 200
Elements supporting cladding with glazed frames	L / 300
Main roof bracing	L / 500
Floor elements under moving loads	L / 500
Portal frame (height H) without crane (1)	H / 150
Single level building without crane (2)	H / 250
Single level building with crane – deflection at rail top combinations including wind effect combinations not including wind effect	 H / 200 H / 400

Table 7 : Allowable horizontal deflections for steel structures

(1) criterion for building without severe deformation requirement

(2) criterion for building with deformation requirement (for example due to brittle walls, comfort, serviceability, …)

b) Deflection of crane supporting beams

SLS verifications are carried out according to EN 1993-6, Sect. 7. The main deflection requirements are summarised hereafter. More severe criteria may be required for specific purpose (for example for crane of Tokamak hall).

- vertical deflection: $\delta_z \leq L / 600$
- horizontal deflection: $\delta_y \leq L / 600$

5.3.3 Design of joints

Connections shall be designed according to NF EN 1993-1-8, with partial factors defined in NF EN 1993-1-8/NA.

Type of verification	Partial factor γ_M
Resistance of joints	
bolts, rivets, welds, axis, diametral pressure	$\gamma_{M2} = 1.25$
sliding resistance	$\gamma_{M3} = 1.1$; $\gamma_{M3,ser} = 1.25$
prestress of high strength bolts	$\gamma_{M7} = 1.1$
other coefficients	$\gamma_{M4} = \gamma_{M5} = \gamma_{M6,ser} = 1.0$

Table 8 : Partial factors for steel joints

The following specific rules add to the previous.

Preloaded bolts with tightening torque:
- The design value of the friction coefficient shall be considered as 0.4, for sand- or shot-blasted surfaces to get a surface treatment grade Sa 2.5.
- The use of a higher value of such coefficient shall be justified by testing.

5.3.4 Fatigue design

The fatigue calculations shall be carried out according to EN 1993-1-9. Concerned elements are:

- Members supporting lifting appliances or rolling loads,
- Members subjected to repeated stress cycles from vibrating machinery,
- Members subjected to wind-induced vibrations.

5.4 Detailing

5.4.1 General

The structural global analysis shall be an elastic linear analysis. However, for specific studies (like in particular accidental situations), non linear analysis may be used.

Profile sections shall be class 2 minimum.

The circulation of any horizontal action shall be carefully studied. Particularly, in front of each significant active mass, specific bracings shall be set (considering the 2 horizontal directions), in order to carry the corresponding seismic actions to the main bracing elements. This applies notably to each beam or purlin.

Non-dimensional slenderness $\overline{\lambda}$ of elements supporting compression forces shall not exceed 2.2.

The minimum thicknesses to be used for open cross section profiles are:

- Structures inside buildings: 4 mm,

- Structures outside buildings: 5 mm.

For hollow sections, these thicknesses may be reduced to:

- Structures inside buildings: 3 mm,
- Structures outside buildings: 4 mm.

5.4.2 Earthquake resistance

In seismic analysis of buildings housing SIC systems, behaviour factors shall not be used (q = 1). Nevertheless, the structure shall be designed in order to provide a robust behaviour during earthquakes. This objective is met with applying ductility requirements according to EN 1998:

- the design of steel structures shall provide a ductility level equivalent to Ductility Class Medium (DCM),
- capacity design principle shall apply to non dissipative areas of structure, notably for connections of dissipative members.

5.4.3 Built up girders

The welded connections between the flanges and the web of « I » shapes have to be designed and calculated with symmetric and continuous welds.

5.4.4 Truss beams

The buckling restraint of the top chord of truss beams shall be provided by the wind roof bracing; only the nodes of such bracing can be considered as « restraining points ». The restraint of such forces by the roof sheeting is not allowed.

The buckling restraint of the bottom chord of truss beams can be achieved by lateral bracings connected to purlins or « scissors »; such scissors shall be connected to a node of the roof bracing system. In that case, lateral bracings shall be located only on one side of the truss beam in order not to create an intermediate support for the purlins.

The stabilising force for buckling restraint shall be evaluated according to EN 1993-1-1, 5.3.3.

The lacings shall be set symmetrically on both sides of the medium plane of the beam.

The loads shall be preferentially applied on the nodes of the beam. Otherwise, the chord of the truss beam has to be designed to support the corresponding bending moments.

5.4.5 Purlins

If necessary, the lateral forces shall be restrained by sag rods. The restraint of such forces by the roof sheeting is not allowed. However, the roof sheeting with an adequate connection with the purlins may be used as torsional buckling restraint for purlins.

5.4.6 Floors

Gratings shall not be used to:
- act as lateral torsional buckling restraint for floor beams,
- carry horizontal lateral forces.

Plate floors shall not be used as wind bracing, unless they are specifically designed for this purpose, enough stiffened, and not taken down.

The minimum thickness of the checked plates shall be 4/6.

5.4.7 Secondary wall structures

The secondary wall structures shall not be used for purpose of stabilisation of the main structure.

The internal skin of double skin wall sheeting can act as a restraint for the lateral torsional buckling of secondary gable columns and wall struts under sagging moment (outside flange in compression).

If necessary, the restraint of the internal flange is done by specific parts (sag bars or equivalent).

5.4.8 Connections

All the bolts used in a same structure will be of the same quality, except in the following case. Bolted preloaded and not preloaded connections may be mixed in the same structure, if the two different types of connections correspond to different structural functions. In that case, two bolts qualities may be used: a specific quality for preloaded connections (10.9 for instance) and another for non preloaded ones (8.8 for instance).

For the whole bracing system (including diagonals and chords), the connections shall comply with EN 1998-1, 6.5.5. Notably, the allowed types of connection are:
- welded connection,
- preloaded bolted connection (categories B, C or E according to EN 1993-1-8, 3.4.1), with friction surface class A or B,
- non preloaded bolted connection with fitted bolts.

Are forbidden:
- semi-rigid connections,
- the use of bolts of strictly following diameters,
- the design of one bolt connections,
- the use of intermittent fillet welds for structures exposed to weather conditions; only symmetrical and continuous welds are accepted for flange/web connection in case of I shape sections,
- direct tension - indicating washers.

Are imposed:
- horizontal stiffeners in columns at the junction with beam flanges,
- stiffeners on web under concentrated load or bearing load.

5.4.9 Crane runways

The crane runways shall be designed according to EN 1993-6, on the basis of simply supported beams.

At the supports, the simply supported design is achieved:
- either by a vertical thick endplate (end stiffener) extended downwards under the bottom flange of the main beam,
- or by using a bearing knuckle.

The beams incorporate stiffeners to prevent torsional deformations and web buckling.

The lateral displacement of the upper flange of the crane runway shall be prevented by an appropriate restraint. The welded connection between the upper flange and the web shall be a full penetrated butt weld with a double bevel preparation (K). Intermittent fillet welds are forbidden.

If a crane runway crosses a joint, the runway detailed design shall take into account the seismic differential displacements.

5.4.10 Anchoring

The shear force at the bottom of columns shall be transmitted to the concrete using a shear abutment.

Steel shims between the top of the concrete and the bottom of the base plate, arranged to compensate the differences of level, shall not exceed an overall depth of 30 mm.

5.4.11 Metal anchors in concrete

Metal anchors may be only used for anchorage of secondary structural elements, for example like studs supporting cladding or door frames.

The performance of these post-installed fasteners shall be demonstrated by a European Technical Approval (ETA). The anchorages shall be designed in accordance with Annex C of the ETAG 001 [108], completed with CISMA guide [109] for earthquake resistance. The choice of the anchorages shall be made assuming that the concrete is cracked. The metal anchors shall be of corrosion resistant material.

If chemical anchors are used, fire resistance and radiation effects have to be analysed.

5.5 Seism design of buildings classified SC2

Buildings classified SC2 shall be designed for seism SL-2 with the following behaviour requirements:

- stability of structures,
- no detrimental interaction with adjacent buildings.

They may be designed with non-linear calculation methods, in accordance with the requirements of ASN/GUIDE/2/01, Sect. 2.5.3.2 [12].

The design shall provide a ductility level equivalent to Ductility Class Medium (DCM), according to EC8 classification.

6 General rules for base isolation and isolating devices

6.1 References and general assumptions

The base isolation of the Tokamak complex is made of elastomeric bearings. The design of base isolation and elastomeric bearings shall comply with the following standards:

- NF EN 1998 – Eurocode 8, particularly Part 1, Sect. 10: Base isolation
- NF EN 1337 – Structural bearings, particularly Part 1: General design rules, Part 3: Elastomeric bearings, Part 10 : Inspection and maintenance
- prEN 15129:2007 – Anti-seismic devices *(note: the definitive standard EN 15129 shall be taken into account for detailed design at its final issue)*

These standards are completed or modified by the specific additional rules defined below.

6.2 Base isolation general design

The general design of base isolation shall comply with EN 1998-1, Sect.10.

The arbitrary accidental additional eccentricity defined in NF EN 1998-1 (10.9.1.2(P) and 4.3.2) is not applicable. This arbitrary eccentricity is neglected: $e_{ai} = 0$.

The bearings layout under basemat has to closely follow the distribution of vertical loads due to the structure, with the following objectives:

- the permanent load on bearings has a spatial distribution as uniform as possible, with variations between extreme loads and average load less than ± 20 % under a quasi-permanent load combination [(1)],
- the eccentricity between the bearings stiffness centre and the building gravity centre has to be as low as possible, in order to reduce torsional effects,
- there is no significant stresses in building structure due to distribution of bearing loads.

(1) Nota: this first criterion aims to ensure a homogeneous behaviour of bearings, according to EN 1998-1 § 10.5.2(2). However, it is acceptable not to meet this requirement for some bearings, with the following conditions:

- *number of bearings for which the criterion is not met < 10 % of total number,*
- *the horizontal stiffness of bearings is not significantly affected by the variation of vertical load (note: according to prEN 15129:2007, the dependence of horizontal stiffness with vertical load is significant only for loads greater than 1/3 of buckling load),*
- *all other design criteria are met.*

6.3 Structural analysis

Appendix A gives specifications for seismic analysis.

6.4 Isolation devices

6.4.1 Mechanical properties

The main material properties whose depend the main mechanical characteristics of elastomeric bearings are:

- G_s static shear modulus of elastomer
- G_d dynamic shear modulus of elastomer
- ξ_{eff} effective damping ratio

The main mechanical characteristics needed for seismic analysis are:

- K_{hs}, K_{hd} static and dynamic horizontal stiffness
- K_{vs}, K_{vd} static and dynamic vertical stiffness
- ξ_h, ξ_v horizontal and vertical effective damping ratio

The mechanical characteristics to be used for the global calculations are given in Table 9.

	Parameter	Symbol	Value	Unit
Static distorsion	Mini shear modulus	$G_{s\ min}$	0.9	MPa
	Maxi shear modulus	$G_{s\ max}$	1.4	MPa
Static compression	Stiffness	K_{vs}	4900	MN/m
Dynamic distorsion	Mini shear modulus	$G_{d\ min}$	0.9	MPa
	Maxi shear modulus	$G_{d\ max}$	1.5	MPa
	Damping ratio	ξ_h	5	%
Dynamic compression	Mini stiffness	$K_{vd\ min}$	4900	MN/m
	Maxi stiffness	$K_{vd\ max}$	7000	MN/m
	Damping ratio	ξ_v	5	%

Table 9 : Characteristics of elastomeric bearings

6.4.2 Design criteria

The design criteria are defined in NF EN 1337-3 for non seismic situations and in prEN 15129:2007 for seismic situations.

The design displacement of isolators includes static displacement (especially due to thermal effects and shrinkage) and seismic displacement. The partial coefficient γ_x defined in NF EN 1998-1, 10.3(2)P and applicable to seismic displacement is equal to 1.00.

The partial coefficient for resistance criteria is γ_m = 1.15 (although the recommended value is 1.00 according to NF EN 1337-3, 5.3.3 and prEN 15129:2007, 8.2.3.4.2).

Isolators have to be fixed on structures at both sides: on supporting substructure and on super-structure. The typical fixing method recommended by prEN 15129, 8.2.3.2 is an anchorage on concrete structure by bolted plates. The anchoring method may be replaced by recess or dowel methods (to avoid sliding) only if there is no risk of uplift.

In addition, in order to provide design margins, the normal compressive stress on the less loaded bearing in seismic situation shall not be less than 1 MPa if isolators are not anchored, and 0 MPa if isolators are anchored (i.e. the isolators are not in tension).

The design of the bearings shall take into account the variability of mechanical parameters around the mean values defined above.

The design of structures and isolators has to enable inspection of isolators and possible replacement.

6.4.3 Testing, inspection and maintenance

The requirements are defined in ITER SDCB Part 2.

7 Verification rules of anchorages in concrete

7.1 Loads applied to anchorages

The solicitations applied to the fixture generate tensile and shear stresses in the anchor rods (connectors).

These stresses are determined by assuming that this torque is applied at a point off-centre relative to the centre of gravity of the fixture (medium/fixture installation tolerance).

The stresses in the anchorages are calculated assuming an (elastic) planar distribution of the deformations while checking that:

- The anchor plate is sufficiently rigid and does not distort,
- The stiffness of all the anchorages (connectors) is identical.

7.2 Tensile forces in anchors

For each limit state, the failure modes to be checked for an anchorage or a group of anchorages relate to:

- steel failure,
- anchorage-to-concrete bond failure,
- concrete cone failure,
- concrete edge failure (spalling).

Formulae used to check the above failure modes depend on the type of anchorage (connector, simple rod, stud...), the depth and spacing thereof, the distance from the anchorage or group of anchorages to a free edge and, where applicable, the state of stress of the medium (concrete in tension or not).

These checks are carried out in accordance with the CEB Design Guide: “Design of Fastening in concrete” [13] or any other validated method; reference shall be made to Eurocode 2, Eurocode 3 and Eurocode 4 for the cases not covered by the CEB Design Guide.

1) Steel failure

§ 15.1.2.2 of the CEB Design Guide applies.

2) Bond failure

✓ Simple rod: embedded anchorage length

Eurocode 2 is applied (§ 8.4.2 and § 3.1.6) to determine the available bond stress:

$$N_{bd} = \pi\ \Phi\ \ell\ f_{bd}$$

where
- Φ is the diameter of the anchorage,
- ℓ is the length of the anchorage,
- f_{bd} $= 0.36\ (f_{ck})^{1/2}\ /\gamma_{Mc}$ for smooth steel,
  $= 2.25\ f_{ctk,0,05}\ /\gamma_{Mc}$ for high-bond steel,
- f_{ck} is the characteristic resistance to compression of concrete,
- $f_{ctk,0,05}$ is the 5% fractile of the tensile resistance,
- γ_{Mc} is the partial factor defined in § 7.6.

Anchor efficiency may be improved using anchor rod with end plate. In that case, the anchor force is transmitted in concrete by compressive stresses at plate interface, and the anchor length may generally be reduced.

✓ Stud

§ 15.1.2.3 of the “CEB Design Guide” applies.

3) Concrete cone failure

§ 15.1.2.4 of the ”CEB Design Guide” applies.

4) Concrete edge failure

§ 15.1.2.6 of the “CEB Design Guide” applies.

7.3 Shear forces in anchors

For each limit state, the failure modes to be checked for an anchorage or a group of anchorages relate to:

- steel failure,
- concrete cone failure,
- concrete edge failure (spalling).

The formulae used to check the above points depend on the type of anchorage (connector, simple rod, stud...), the depth and spacing thereof, the distance from the anchorage or group of anchorages to a free edge and, where applicable, the state of stress of the medium (concrete in tension or not).

These checks are carried out in accordance with the CEB Design Guide [13] or any other validated method; reference shall be made to Eurocode 2, Eurocode 3 and Eurocode 4 for the cases not covered by the CEB Design Guide.

1) Steel failure

§ 15.1.3.2 of the "CEB Design Guide" applies.

In the case of an anchorage welded to the fixture, the factor 0.6 is replaced by 0.75 so as to take the presence of the weld into account in the shear resistance.

For a group of anchorages, the strength of the group is derived from the cumulated strength of each anchorage, decreasedf by a factor 0.8 (§ 9.3.2.1 of the CEB guide).

2) Concrete cone failure

§ 15.1.3.4 of the "CEB Design Guide" applies.

3) Concrete edge failure (spalling)

15.1.3.5 of the "CEB Design Guide" applies.

4) Resistance of the concrete to the diametrical pressure of the anchor rods

The resistance of the concrete to the diametrical pressure of the anchor rods is to be checked according to EN 1994-1-10, 6.6.3.1.

When the fixture is subjected to high shear stresses, provision shall be made for cleats or abutments on the underside of the plate and possibly confinement of the concrete in the volume submitted to high stresses. The strength of the concrete under these shear stresses is to be verified in accordance with EN 1992-1-1, 6.7, with regard to local crushing of the concrete and the transverse tensile stresses generated.

7.4 Combined tension / shear

If the anchorages are submitted to combined tensile and shear forces, the following requirements (from the "CEB Design Guide", § 15.1.4), shall be verified:

$$N_{Sd} / N_{Rd} \leq 1$$

$$V_{Sd} / V_{Rd} \leq 1$$

$$(N_{Sd} / N_{Rd})^{3/2} + (V_{Sd} / V_{Rd})^{3/2} \leq 1$$

7.5 Welds

Stresses in welds between connectors and plates shall be verified according to EN 1993-1-8.

7.6 Material partial factors

- Steel

If there are no particular specifications associated with the type of connector used, the following partial factors for steel which conforms to EN 1992-1-1, 2.4.2.4 and the CEB Design Guide (§ 3.4):

- ✓ ULS

  γ_{Ms} = 1.15 under fundamental situations (ELU.f)
  = 1.0 under accidental and seismic situations (ELU.a and ELU.e)

- ✓ SLS

  γ_{Ms} = 1.0

- ❑ Concrete

The following partial factors apply to concrete which complies with requirements of EN 1992-1-1 (§ 2.4.2.4) and the CEB Design Guide (§ 3.2.3.1 and § .3.4).
When the concrete is in tension, the partial factor used for the concrete under compression is increased by 1.2:

γ_{Mc} = $1.2 \times \gamma_c$ when the concrete is in tension,
= γ_c otherwise.

It is assumed that the concrete is in tension when the longitudinal or axial stresses in the volume where the anchorages are located are tensile stresses; these stresses are calculated assuming that the cross-sections are uncracked and taking in account effects of shrinkage and other distortions which the concrete is subjected to.

- ✓ ULS

γ_c = 1.5 under fundamental situations (ELU.f)
= 1.2 under accidental and seismic situations (ELU.a and ELU.e)

- ✓ SLS

γ_c = 1.0

APPENDIX A. Seismic analysis

This appendix gives guidelines for the seismic analysis of the structure to determine design values of physical quantities needed for the design of:
- Civil works: soil reactions, internal forces, absolute or relative displacements…
- Equipment: accelerations, relative displacements between anchorages…

A.1. Ground motion

A.1.1. Reference spectra

Reference spectra are defined in [3].

A.1.2. Time histories consistent with the spectrum used

For certain analysis, notably non linear analysis, time histories may be used. The selected input expressed in terms of acceleration shall comply with [12]. References of accelerograms corresponding to Cadarache earthquakes spectra are given in [3]. Modelisation and analysis shall then comply with [12].

A.2. Soil profile characteristics

A.2.1. Homogeneous equivalent soil

A homogeneous half space is represented by an elastic, homogeneous isotropic material, defined by its mechanical characteristics (dynamic Young's or shear modulus, dynamic Poisson's ratio, density, material damping).
Real layered soils generally have lower radiative damping than homogeneous ones. This difference is addressed in § A.4 when computing the modal damping.

A.2.2. Layered soils

If the soil properties vary with depth, this variation shall be taken into account in the analysis.
If a building lies on a layer of infill above rock, this shall be taken into account in both static and dynamic analyses.
When appropriate, the soil is defined as horizontally layered with bedrock; each layer has constant properties.

A.2.3. Variability of soil properties

Due to uncertainties in the definition of the soil properties of equivalent homogeneous soil as well as in the definition of layers, a variation of the shear modulus related to the best estimated value (G_{ave}) is required for each layer. The variation range is limited either by [G_{min} = 2/3 G_{ave} and G_{max} = 3/2 G_{ave}] or by other values which remain to be justified.

As the soil properties (G and material damping) are strain dependent, the degradation curves are also necessary.

The non-linear properties of the soil layers are approximated by equivalent linear properties consisting of the equivalent linear shear modulus and damping ratio for the soil which are compatible with the induced strain amplitudes in the soil medium (method proposed by Seed and Idris).

A.3. Modelling

A.3.1. Material dynamic properties

The following material damping values are admissible at ULS,e:
- Reinforced concrete: 7 %
- Pre-stressed concrete: 5 %
- Bolted Steel: 7 %
- Welded steel: 4%

A.3.2. Structure

The structure is defined by:
- its geometry,
- its material properties.

A.3.3. Masses and inertia

Masses and inertia to be introduced in the model shall take into account the mass of all structural and non structural elements, the mass of equipment in their normal operating conditions and the masses associated to variable loads, at their quasi-permanent values associated to the seismic situation. In the absence of other indication, the mass associated to live loads Q_0 shall be taken at 50% of their maximal characteristic value, not less than 100 kg/m^2. However, for local design of structural members, the whole live load has to be taken into account.

In SL-3 conditions, the mass associated to live loads to be introduced in the dynamic model shall be taken at 25% of their maximal characteristic value, not less than 100 kg/m².

The modelisation of mass includes:
- mass of structural members, water and other liquids, and of equipment present during normal operating conditions,
- masses corresponding to average loads in normal operating conditions.

Equipment is defined by its mass located at the centre of gravity and its supports.

A.3.4. Stiffness

Concrete members are generally modelled as uncracked sections for the dynamic analysis. In that case, the Poisson ratio is equal to 0.2.

However, when cracking due to flexure can not be neglected (notably if is has an influence on the members design or the floor response spectra), the cracked bending inertia shall be taken into account. In the absence of more detailed analysis, the inertia variability of the cracked elements can be evaluated according to the simplified approach given in appendix 1 of ASN/Guide/2/01 [12], by the way of an equivalent Young modulus with the following variability range :

- for walls and slabs : 20 000 - 33 000 MPa,
- for beams and columns, if the bracing system is a frame system : 10 000 - 20 000 MPa.

When it is needed according to § A.3.5, the stiffness of the masonry panels is evaluated from the existing standard applicable to the type of masonry blocks considered. It is acceptable to take into account the cracking of the masonry panels, with cracked properties being taken half their values in uncracked conditions.

The equipment deformability is included in the structural model as far as they induce significant effects on its behaviour.

A.3.5. Structural models

The models for the dynamic analysis are detailed 3D models using finite elements.

The models shall allow for a good representation of the seismic behaviour of the structure, notably with a representative modelling of the distribution of masses and of the bracing system. It is possible to combine models to reach this general objective.

The modelisation shall include finite elements such as elements of beams, plates, shells and volumes. The number of nodes and elements shall be adequate for a good representativeness of modes and internal forces under seismic action, including local effects.

When secondary structural elements or non structural elements may influence the dynamic behaviour of the structure because of their stiffness and of their rigid liaisons to the structure, this stiffness shall be taken into account in the model.

Distribution of masses (including lump masses) in the model shall be chosen so as to obtain modal shapes and modal effective masses representative of the global and local behaviours of the structure under the seismic design situation:

- lumping of masses shall not induce unrealistic local deformations;
- enough masses shall be introduced to represent adequately the local deformation of flexible elements (walls, floors…);

When derived by impedance functions as given in § A.3.6, the soil-structure interaction (SSI) is represented by systems of stiffness matrix or alternatively by systems of springs located at each node of the interface, so as to reproduce adequately the global behaviour of the soil and of the foundations, as detailed in § A.3.6 The damping effect is taken into account according to § A.3.6 and A.4.2.

The SSI can also be taken into account by a finite element model including both the soil and the structures.

As the SSI is taken into account, masses and inertia associated to the foundation elements shall be taken into account to adequately represent their kinetic energy.

For the modelling of deep foundations, requirements of EN1998-part 5 may be used.

The effects of hydrodynamic mass and damping shall be considered in the analysis of structures and tanks, for horizontal and vertical components of earthquake.

For the modelling of specific constructions or of specific effects (such as fluid-structure interaction), EN 1998 may be used whenever it covers the said constructions or effects.

Whenever types or constructions or effects are not covered either by the present text or by EN 1998, recognised technical manuals or references may be used, subject to the Owner approval. In particular, adequate methods shall be proposed for the analysis of underground structures such that tunnels.

A.3.6. Modelisation of soil-structure interaction

1. General

The soil is represented in the structural model by stiffness matrices (or, in a more simplified way, by sets of springs), located at each node at the interface between the soil and the structure. These matrix are determined to represent the global elastic behaviour of the soil and, whenever applicable, the interaction between structures. One of the methods below may be used:

a) from frequency dependant impedance functions, when the foundation may be considered as sufficiently rigid as regards the soil behaviour; additional conditions are given below for the use of this method;

b) from a modelisation of the soil with appropriate discrete elements taking into account the wave dispersion in the different soils layers.

Whichever method is used, the different layers of soil shall be taken into account with their mechanical properties.

The effect of embedment shall be taken into account.

2. Impedance functions

Impedance functions are obtained using qualified analytical or numerical methods. For this purpose, the basemat can be taken as rigid and flat.

The stiffness coefficients are calculated from the impedance functions for the fundamental frequencies, i.e. those frequencies corresponding to the modes with the higher modal effective mass in each direction. When necessary, the dynamic analysis is iterated so that there is a good agreement between the frequencies used for the stiffness determination and those finally obtained from the dynamic analysis taking into account the SSI.

It is admitted to calculate the impedance functions with the assumption of a homogeneous soil when the variations of the shear modulus between the different layers are progressive and moderate over a depth equal to at least 1.5 times the largest basemat dimension. In that case, equivalent soil modules are obtained from a static analysis, so that, for each direction and each force (or moment) under consideration, the static displacement (or rotation) of the homogeneous soil is equivalent to that of the stratified soil, taking into account the different mechanical properties of the different layers. When the different equivalent modules calculated by this method differ by less than 20%, it is acceptable to use a single mean value.

3. Distribution of global stiffness on a widespread basemat

When the interface between the soil and the structure is discretised in the frame of a detailed finite elements modelisation, stiffness matrices are associated at each interface node of the model. These matrices may also couple degrees of freedom of different interface nodes.

When the stiffness and the damping of the soil have been determined from the impedance functions, the stiffness matrix at the interface nodes shall represent globally the same dynamic behaviour of the soil than that obtained through the impedance functions.

Notably, in the case of spring’s representation, the global SSI springs have then to be distributed over basemat nodes proportionally to the surface. This distribution is performed in such a way that global forces and displacements at the basemat-soil interface are consistent with the global stiffness in each of the 6 DOFs.

Uplift from the foundation soil shall be taken into account by appropriate methods in non linear analysis and requirements above apply. However, it may be assumed that accelerations in the building calculated by a linear analysis are an acceptable approximation, provided that the uplift remains lower than 30% of the contact surface during the movement.

4. Soil damping

Soil damping is taken into account by the means of a modal damping ratio, calculated by mode, proportionally to the strain energy.

The soil damping is determined taking into account:

- the material damping of the soil which is assumed to be equal 5%, whatever the considered frequency ;
- the SSI radiative damping, which can be calculated for each seismic component and each mode from the impedance functions at the corresponding frequency.

A.4. Dynamic analysis

A.4.1. General

Seismic design is done in the elastic domain for all SIC structures and for non-SIC structures that house SIC systems. No behaviour factor is taken into account (q = 1).

The modal analysis used in § A.4.2 is the reference method.

The simplified methods given in EN 1998-1, 4.4.3.2 are not acceptable in the frame of the present document (however, they may be used in preliminary studies to obtain a rough estimation of the seismic forces).

Frequency domain analysis is a suitable method for SSI on layered sites, as there is no specific restriction in the thickness and properties of the soil layers.

Time history analysis shall also be used for the derivation of floor spectra (see § A.5) or for non linear analyses.

Frequency domain analysis and time history analysis are not developed in the present document. Their use shall comply with relevant methodology according to the state-of-the-art in these fields. Modelisation for material non linear analysis shall comply with requirements of ASN/Guide/2/01 [12] and EN 1998-1.

A.4.2. Modal analysis

1. General

In this method, the maximum response of each mode is evaluated from the structural model set up according to § A.3, using the site specific spectrum defined in § 3.2.1.

When the soil reactions obtained from the modal analysis for the design seismic combination at ULS-e are tensile forces at the interface between soil and structure, the basemat is uplifted from the soil. In that case, the soil reactions shall be re-evaluated in cancelling these tensile forces. However, it is acceptable not to proceed with this modification when the compressed interface area is higher than 90% of the total interface area.

When the compressed interface area is lower than 70% of the total interface area, a non linear time history analysis shall be performed, or alternatively, an equivalent static analysis taking into account the uplift. If the result of this non linear analysis shows that the compressed area is greater than that obtained in the modal analysis, it is acceptable either to keep the results of the modal analysis or to decrease the effects of the modal analysis so as to obtain the same uplift than in the non linear analysis. In the cases where the soil strains are far beyond the elastic range, specific studies shall be performed.

2. Number of modes

It shall be retained a sufficient number of modes to correctly represent the deformation of the structure under the seismic action. In principle, this number is obtained when one of the conditions below is fulfilled:

- the frequency of the last mode taken into account is higher than the cut-off frequency (33 Hz),
- the sum of the effective masses of the modes retained is higher than 90% of the total mass of the structural model.

The response of all modes contributing significantly to the global response shall be taken into account.

In the case of a detailed structural model using finite elements, it is acceptable to retain modes summering only 70% of the total mass.

In the two above situations where 90% of the total mass is not obtained, a rigid body mode shall be added, with the frequency of the last mode taken into account.

3. Modal damping

Modal damping has to reflect the contribution of soil material damping, radiative (geometric) SSI damping and structures damping.

For each mode 'i', the following weighted sum has to be used, in which each subsystem damping is weighted by its deformation energy.

The weighted sum is the following :

$$\eta_i = \lambda . \left\{ \sum_k \left(\eta_{i_k} \cdot \frac{E_{i_k}}{E_{i_T}} \right) \right\} + \eta_{soil} \cdot \frac{E_{i_{soil}}}{E_{i_T}} + \eta_{struct1} \cdot \frac{E_{i_{struct1}}}{E_{i_T}} + \eta_{struct2} \cdot \frac{E_{i_{struct2}}}{E_{i_T}} \ldots$$

where :

η_i is the modal damping,

η_{i_k} is the geometric damping coefficient associated with the k degree of freedom of the global soil spring at the frequency of mode 'i',

$\eta_{soil}, \eta_{struct1} \ldots$ are the material damping coefficients for the soil and the structures, defined in § A.3,

$\frac{E_{i_k}}{E_{i_T}}$ is the fraction of the total strain energy associated with soil spring DOF 'k',

$\frac{E_{i_{soil}}}{E_{i_T}}, \frac{E_{i_{struct1}}}{E_{i_T}}$ are the fractions of the total strain energy in the soil and in the structures,

The radiative damping is multiplied by λ to account for the simplification of the soil profiles:

- λ = 1/2 if impedances functions are calculated with the assumption of an homogeneous soil,,
- λ = 1 if the variation of soil properties in the different layers is explicitly taken into account in the impedance functions calculation method.

In addition, the resulting value of η is limited to 0.30.

4. Combination of modes

The mechanical effects such that displacements of nodes, relative displacements between nodes, internal forces and moments, support reactions, etc (with the exception of the absolute acceleration at each level of the structure), are calculated for each mode in a given seismic direction and shall be combined using the complete quadratic method (CQC). Let x be one of these quantities, $\bar{x}$ its combined value and x_i its n modal values $\bar{x}$ are obtained by the following formulae:

$$\bar{x} = \sqrt{\left(\sum_{i=1}^{n}\sum_{j=1}^{n} C_{ij} x_i x_j\right)}$$

where C_{ij} is given by:

$$C_{ij} = \frac{8\left(\eta_i \eta_j\right)^{0.5}\left(\eta_i + r\,\eta_j\right) r^{1.5}}{\left(1-r^2\right)^2 + 4\eta_i \eta_j\, r\left(1+r^2\right) + 4\left(\eta_i^{\,2} + \eta_j^{\,2}\right) r^2}$$

r being the ratio between the two pulsations: $r = \omega_j/\omega_i$, η_i and η_j the damping ratio of the modes considered.

This combination shall not be used with modes with frequencies higher than the cut-off frequency. These modes shall be incorporated in a rigid body mode and this rigid body considered in the CQC.

5. Combination of directions

A design effect of the seismic action shall be obtained in combining the values of the same effect obtained in each direction of excitation, either by the Newmark or the SRSS rule:

- Newmark rule : if responses Rx, Ry, Rz result from the analysis in the three directions of excitation, then the most penalising combination among the three following is to be considered:

$$+ |Rx| \pm 0.4\,|Ry| \pm 0.4|Rz|$$
$$\pm 0.4\,|Rx| + |Ry| \pm 0.4|Rz|$$
$$\pm 0.4\,|Rx| \pm 0.4\,|Ry| + |Rz|$$

- According to SRSS rule : $R = \sqrt{Rx^2 + Ry^2 + Rz^2}$

A.5. Dynamic analysis of isolated structures (Tokamak complex)

EN 1998-1, 10.9 defines allowable structural analysis methods. The present paragraph gives additional requirements, in addition to § 6.

Seismic design is done in the elastic domain for both substructure and superstructure (q = 1).

The simplified methods of NF EN 1998-1, 10.9.3 or 10.9.4 are acceptable only for preliminary design purpose, and with the following additional requirements:

- due to the small number of shear walls, the building cannot be considered as rigid in the calculation of vertical load distribution on isolators, under overturning moment effect: the influence of building flexibility has to be taken into account (deformation of basemat is not necessary plane), therefore the load distribution on bearings has to be calculated using a 3D finite element analysis
- the influence of the soil flexibility has to be assessed, especially on vertical and rocking behaviour.

For detailed design purpose, only multi-modal analysis is allowed (or time-history if necessary), performed with a global finite element model representing the superstructure, the isolators, the infrastructure, and the soil-structure interaction.

The variability of both soil stiffness and isolation stiffness has to be taken into account. These 2 variable parameters may be combined as follows:

- 1st assumption: minimal stiffness of both soil and isolation,
- 2nd assumption: maximal stiffness of both soil and isolation.

In addition, it shall be justified that these two assumptions envelop the most adverse effects on the structures and isolation system.

A.6. Calculation of local solicitation distribution in the structure

The distribution of local solicitation in the structure is calculated using a finite element analysis. Two acceptable alternative methods may be used:

- either solicitations are derived directly from dynamic seismic analysis,
- or they are calculated according to an equivalent static analysis.

If the solicitations are derived directly from dynamic seismic analysis, special care is drawn to the following points:

- the eventual non linear effects which have to be taken into account (such as foundation uplift) are not compatible with modal analysis,
- for elements whose design depends on several solicitation components (for instance combined flexure in beam or shell elements), the simultaneity of the different components values and signs has to be established according to validated method.

In case of equivalent static analysis, the equivalent static load cases are derived from pseudo-accelerations calculated by the dynamic analysis, for each direction of excitation, and the combination of directions is carried out according to Newmark rule. Equivalent static load cases have to be validated by comparison of global resultant forces and moments at each building level with those resulting from dynamic analysis.

A.7. Floor response spectra generation

The floor response spectra (FRS) are determined by calculation of maximum responses of oscillators submitted to floor accelerograms derived from a time-history calculation of the building's response. They are computed for 2, 5, 7, 10 and 20% damping ratios.

The FRS computation includes the following phases:

1. Time-history calculation

For each of 3 directions of excitation, 3 different time-histories are to be used; the 3 results from the 3 time-histories shall then be averaged.

Two procedures can be used to combine the 3 directions of excitation:
a. The calculation is made for each set of 3 artificial time-histories acting simultaneously in the 3 directions;
b. One calculation is made for each of 3 directions of excitation, for the 3 time histories. Afterwards, results are combined using Newmark or SRSS rule (see c) below).

2. Calculation of the Floor Response Spectrum for each earthquake direction and each degree of freedom of translation at each selected structural node of the floor.

3. Combination of directions

a. In case of simultaneous excitation in the 3 directions, the response already includes the combination of directions;
b. In case of 3 independent calculations for the 3 earthquake directions, Newmark or SRSS rule as given in § A.4.2 shall be used to combine the FRS resulting in one direction from the 3 excitations.

4. Envelope of nodes

The envelope of FRS obtained at relevant structural nodes of the given floor level or area is computed.

5. Envelope of soil conditions

In order to cover all possible soil conditions in the range, the envelope is computed. Peaks which are shifted by the change of soil conditions have to be linked together so that the corresponding valleys are filled.

6. Broadening

In order to account for uncertainties in the input data and the modelling assumptions and techniques, the spectra are broadened by ± 15 % around the FRS peaks corresponding to Eigen frequencies of the soil-structure system according to NRC Regulatory guide RG 1.122.

7. Determination, as far as it is needed, of the envelope of both horizontal directions.

Attention is drawn to the fact that the model used or the method of analysis used in the time domain shall take into account local amplifications in the vicinity of the equipment concerned by the floor spectra. This is particularly the case when a modal decomposition is used, where the rules given in A.4.2 (2) may be not sufficient.

A.8. Sensitivity studies

Sensitivity studies may be necessary on the following aspects:
- structure soil structure interaction between adjacent buildings (coupling effects between buildings),
- effect of embedment,
- modelling assumptions of the structures (cracked concrete ...),

- influence of basemat uplift on floor spectra, when the compressed interface area is lower than 70% of the total soil / structure interface area.

APPENDIX B. Verification of concrete structures under impact and impulsive loads

This appendix describes an acceptable simplified method which can be used to assess the bearing capacity of concrete structures submitted to impact and impulsive loads, such as aircraft impacts and load drops. The method is derived from CEB guide [14].

B.1. Principles of the modelisation and calculation's method

B.1.1. Principles of the modelisation

The simplified model is composed of three masses which are connected by elasto-plastic springs. These masses represent:

- the local mass M_3 of the punching cone under the effect of the impact,
- the local mass M_2 of the slab which is yielded in bending,
- the mass M_1 which represents the surrounding structure.

The masses are connected by the following springs:

- an elasto-plastic spring R_3 represents the punching phenomenon with the contribution of the tensile strength of concrete, the stirrups which are elongated until rupture and the bending reinforcement which develops large deformations,
- an elasto-plastic spring R_2 represents the elasto-plastic bending of the slab,
- an elastic spring R_1 represents the surrounding structure.

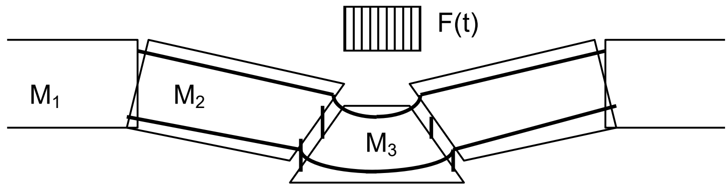


Figure B1: ***Structural parts for the simplified model***

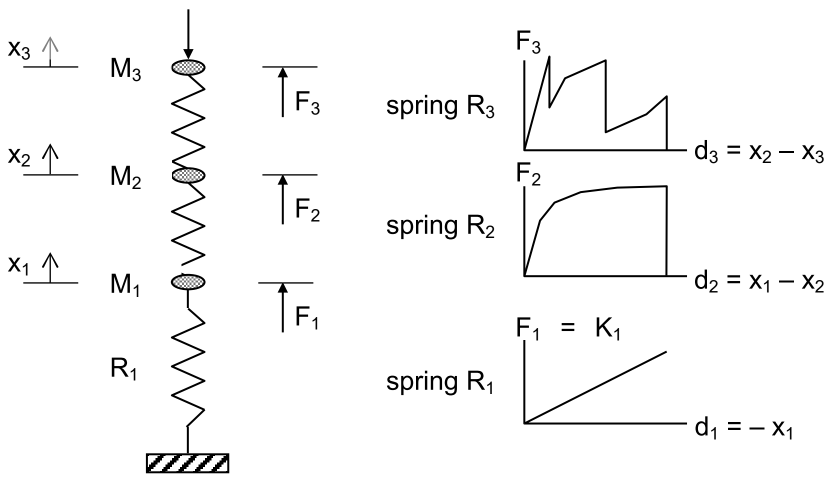


Figure B2: ***Simplified model***

This simplified model is submitted to the time-history load F(t) representing the impact. A time-history analysis is performed in order to determine the maximum displacements.

The structural damping ratio is taken equal to 2% for the spring which represents the punching phenomenon and equal to 7 % for the spring which represents the bending of the slab and of the structure (structural elements). However, for the non linear springs R_2 and R_3, precautions have to be taken for damping modelling. Actually, a viscous damping model could overestimate the effective damping effect in non linear range. In the absence of specific justifications, damping in springs R_2 and R_3 has to be neglected. For these elasto-plastic springs, the major part of the damping consists in the hysteretic damping due to yielding effect.

Soil-structure interaction may be associated to the spring representing the structure. In that case, the corresponding clauses of Appendix A apply and damping associated to this particular spring shall be limited to the following values:

- horizontal and torsional vibrations: 15% of critical damping
- vertical vibrations: 30% of critical damping

B.1.2. Geometric modelling

The impact area is rectangular or circular. For each wall or slab panel, various impact locations have to be studied, at least one impact at panel centre and one impact near supports.

The punching cone geometry is defined by the width a and the length b (> a) of the rectangular impact area, or the radius a of the circular impact area, by the slab thickness h and by the angle α of the punching cone. An angle α = 45° may be used.

The wall or slab panel geometry is defined by the length L_x, the width L_y and the thickness h of the panel, and its reinforcement. In case of circular impact, an equivalent circular slab geometry is defined with a radius r of the slab and by the shell thickness. The radius r may be defined according to the location of the bearings.

The third mass and the third spring represent the mass and the displacements of the surrounding structure.

B.2. Model for rectangular impact

B.2.1. Mass modelling

B.2.1.1. Punching cone mass

The punching cone mass M_3 is:

$$M_3 = \rho \left(a\,b\,h + (a+b)\frac{h^2}{tg\alpha} + \frac{\pi}{3}\frac{h^3}{tg^2\alpha} \right)$$

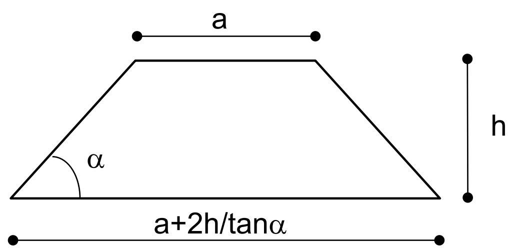


Figure B3: ***Punching cone***

B.2.1.2. Wall or slab panel mass

The dynamic equivalent mass of the panel is evaluated under the assumption that the simplified model of the slab with the total mass M_2 + M_3 has a self frequency equal to f_2, self frequency of the panel according to its boundary conditions. If K_{2t0} is the elastic stiffness of the slab (see § B.2.2.2), the panel equivalent mass is:

$$M_2 = \frac{K_{2t0}}{(2\pi \cdot f_2)^2} - M_3$$

In addition, the total mass M_2 + M_3 is limited to the following maximal value:

- 50 % of total panel mass ($M_{panel} = L_x L_y h \rho$) in case of one way simply supported panel,
- 25 % of M_{panel} in other cases (embedded panel, two ways simply supported panel).

B.2.1.3. Surrounding structure mass

The surrounding structure mass is evaluated assuming that the simplified model with the total mass M_1 + M_2 + M_3 has a frequency equal to f_1, with f_1 an estimation of the total structure frequency in the direction of impact. If K_1 is the elastic stiffness of the structure (see § B.2.2.3), the surrounding structure mass is:

$$M_1 = \frac{K_1}{(2\pi \cdot f_1)^2} - M_2 - M_3$$

B.2.2. Stiffness modelling

B.2.2.1. Concrete punching cone elasto-plastic spring

The elasto-plastic spring R_3 represents the punching phenomenon. The force F_3 is the sum of the three corresponding contributions:

$$F_3(d) = F_{3.1}(d) + F_{3.2}(d) + F_{3.3}(d)$$

- $F_{3.1}$ is the contribution of the tensile strength of the concrete along the expected cone boundaries
- $F_{3.2}$ is the contribution of the stirrups which are elongated until rupture,
- $F_{3.3}$ is the contribution of the bending reinforcement which develops large deformations.

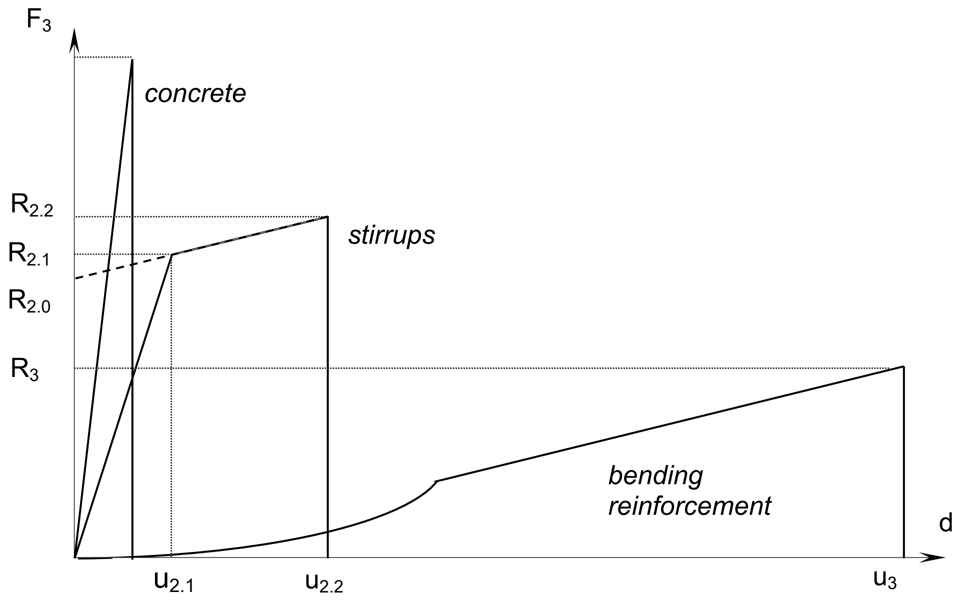


Figure B4: ***Constitutive law for punching cone elasto-plastic spring.***

$F_{3.1}(d)$ is a linear function until concrete tensile break defined by:

- displacement $u_1 = 0.33\, h \dfrac{f_{ctk,0.05}}{E_{cm}}$
- force $R_1 = \Sigma \times f_{ctk,0.05}$
- where Σ is the projected area of concrete punching cone : $\Sigma = 2\,(a+b)\dfrac{h}{\mathrm{tg}\,\alpha} + \pi \dfrac{h^2}{\mathrm{tg}^2\alpha}$

$F_{3.2}(d)$ is a bi-linear function defined by:

- a linear comportment until yielding point of stirrups, defined by the displacement $u_{2.1} = 0.33\, h \dfrac{f_{yk}}{E_s}$ and the force $R_{2.1} = \Sigma\, A_t\, f_{yk}$, where A_t is the stirrups area density (the stirrups are supposed to be perpendicular to the panel plane),
- a plasticity comportment until stirrups tensile break defined by the displacement $u_{2.2} = 0.9\, h\, \varepsilon_{ud}$ and the force $R_{2.2} = \Sigma\, A_t\, f_{tk}$, where $\varepsilon_{ud} = 0.5\, \varepsilon_{uk} = 2.5$ %,

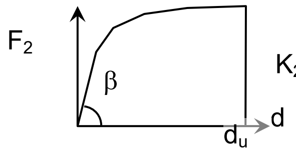
$F_{3.3}(d)$ is determined by the bending reinforcement membrane catenary comportment:

- the membrane span length is $\ell = a + \dfrac{2\,h}{\mathrm{tg}\,\alpha}$,
- the membrane length is $L = b + \dfrac{2\,h}{\mathrm{tg}\,\alpha}$,
- $F_{3.3}(d) = 2\,\sin\left(\mathrm{arctg}\dfrac{4\,d}{\ell}\right) L\; A_i\; \sigma(\varepsilon)$, where d is the membrane deflection and
- $\varepsilon(d) = \dfrac{1}{2}\left[\sqrt{1+\left(\dfrac{4\,d}{\ell}\right)^2} + \dfrac{\ell}{4\,d}\ln\left(\dfrac{4\,d}{\ell} + \sqrt{1+\left(\dfrac{4\,d}{\ell}\right)^2}\right)\right] - 1$

The contribution of stirrups ($F_{3.2}$) and/or bending reinforcement ($F_{3.3}$) may be neglected if $F_{3.1}$ is sufficient to equilibrate the impact force.

B.2.2.2. Concrete panel elasto-plastic spring

The spring R_2 represents the global bending comportment law of the impacted panel. The non linear law $F_2(d)$ is determined by a structural analysis of the rectangular panel under progressive distributed load on the impact area. The model has to take into account the reinforcement distribution in the panel, and the non linear comportment laws of concrete (cracking and yielding) and reinforcement (yielding).



K_{2t0} = tangent stiffness = tg β

Figure B5: ***Constitutive law for concrete panel in bending.***

The displacement d is defined as average displacement under the impacted area. The ultimate displacement d_u is defined by the reach of one of the criteria ULS.R6 (see § 4.5.1.6):

- maximal concrete compressive strain : ε_{cu2} = 5 ‰,
- maximal steel tensile strain : $\varepsilon_s = 0.5\ \varepsilon_{uk}$ = 25 ‰,
- maximal rotation of plastic hinges : $\theta_{\lim} = Min\left\{0.025; \frac{0.005}{x/d}\right\}$.

The structural non linear analysis shall use either a non linear finite element model, or other validated analytical or empirical approach.

B.2.2.3. Surrounding structure elastic spring

The spring R_1 is elastic linear. Its stiffness K_1 may be determined using the finite element model of the building, calculating the average displacement d of panel borders under a unit load F applied on the impact area : $K_1 = F / d$.

B.3. Model for circular impact

The general methodology is defined in § B.1 and B.2. Specific formulae are developed hereafter to adapt the model for circular impact cases.

B.3.1. Mass modelling

B.3.1.1. Punching cone mass

The punching cone mass M_3 is:

$$M_3 = \pi \cdot \rho \cdot h \cdot \left[a^2 + \frac{ah}{\tan\alpha} + \frac{1}{3}\left(\frac{h}{\tan\alpha}\right)^2 \right]$$

B.3.1.2. Circular slab mass

The lower natural frequency of a circular slab clamped in the periphery and with an uniform mass is:

$$f_d = \frac{10.22}{2\,\pi\,r^2}\sqrt{\frac{D}{\rho\,h}}\text{, where D = E I}$$

The dynamic equivalent mass of the circular slab is evaluated under the assumption that the simplified model of the slab with the total mass $M_2 + M_3$ has a frequency equal to f_d. If K_d is the elastic stiffness of the slab, the slab mass is:

$$M_2 = \frac{K_d}{(2\pi \cdot f_d)^2} - M_3$$

B.3.1.3. Surrounding structure mass

The surrounding structure mass is evaluated assuming that the simplified model with the total mass $M_1 + M_2 + M_3$ has a frequency equal to f_1, with f_1 an estimation of the total structure frequency in the direction of impact. If K_1 is the elastic stiffness of the structure (see § B.3.2.3), the surrounding structure mass is:

$$M_1 = \frac{K_1}{(2\pi \cdot f_1)^2} - M_2 - M_3$$

B.3.2. Stiffness modelling

B.3.2.1. Punching cone elasto-plastic spring

The global law of R_3 spring is defined according to the following formulae.
$F_{3.1}(d)$ is defined by:

- the displacement $u_1 = 0.33\, h \frac{f_{ctk,0,05}}{E_{cm}}$
- the force $R_1 = \Sigma \times f_{ctk,0,05}$
- where Σ is the projected area of concrete punching cone : $\Sigma = \frac{\pi\, h}{tg\,\alpha}\left(2\,a + \frac{h}{tg\alpha}\right)$

$F_{3.1}(d)$ is defined by:

- a linear comportment until yielding point of stirrups, defined by the displacement $u_{2.1} = 0.33\, h \frac{f_{yk}}{E_s}$ and the force $R_{2.1} = \Sigma\, A_t\, f_{yk}$, where A_t is the stirrups area density (the stirrups are supposed to be perpendicular to the panel plane),
- a plasticity comportment until stirrups tensile break defined by the displacement $u_{2.2} = 0.9\, h\, \varepsilon_{ud}$ and the force $R_{2.2} = \Sigma\, A_t\, f_{tk}$, where $\varepsilon_{ud} = 0.5\, \varepsilon_{uk} = 2.5$ %,

$F_{3.3}(d)$ is determined by the bending reinforcement membrane catenary comportment, and may be assumed to be a linear function until reinforcement break, defined by:

- the displacement $u_3 = \ell \sqrt{\frac{3\,\varepsilon_{ud}}{2}}$
- the force $R_3 = 4\,\pi A_l f_{yk} u_3$
- where A_l is the amount of lower reinforcement in each direction (area per unit length) and ℓ is the membrane radius: $\ell = a + \frac{d}{tg\,\alpha}$

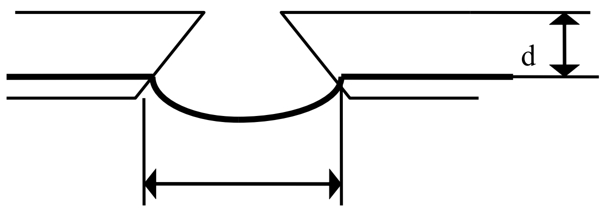


Figure B6: ***Deformation of the bending reinforcement.***

B.3.2.2. Concrete panel elasto-plastic spring

The general method described in § B.2.2.2 is applicable. However, a simplified bilinear elasto-plastic law defined hereafter may be used, with the following conditions:

- the radius of the circular slab is limited to 10 times the slab thickness,
- bending rebars are approximately isotropic at each side of the shell,
- stirrups are perpendicular to the mean fibre of the shell,
- the impact is in the centre of a plate to allow for yielding by flexure.

If these assumptions are not fulfilled, an appropriate model has to be developed.

If the last condition is not fulfilled, the model may be adapted to fit the real behaviour of the structure. For instance, if the impact occurs near a support line, the model may be modified in removing the second spring in the model, as the stiffness in direct shear is very high. In that case, the mass associated to the flexure of the slab shall be merged with the mass associated to the structure.

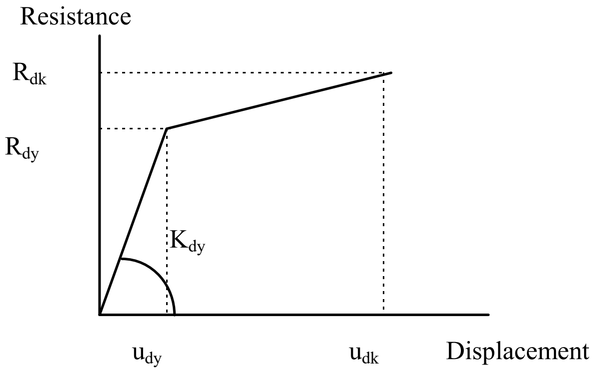


Figure B7: ***Constitutive simplified law of the circular slab.***

According to the amount of reinforcement and to the concrete cross section, the elastic bending moment M_{dy} and the cracked rigidity D (D = $EI_{cracked}$) are evaluated taking into account the eventually normal force associated with the bending moment.

The ultimate plastic bending moment M_{dk} is evaluated following the design criteria concerning the allowable strains in the concrete and in the reinforcement.

The elastic displacement of the circular slab clamped onto the periphery and loaded by a unit force concentrated on a circular area is:

$$\frac{1}{K_{dy}} = w = \frac{a^2}{64\pi \cdot D} \cdot \left[\left(4 - 3\gamma^2\right) + 4 \cdot \gamma^2 \ln\gamma\right]$$

with γ = a / r ; the Poisson's ratio ν is taken equal to 0.

The elastic resistance R_{dy} of the slab is calculated according to the formula of a circular slab clamped on the periphery and loaded by a unit force concentrated on a circular area:

$$R_{dy} = \frac{16\pi \cdot M_{dy}}{(1+\upsilon) \cdot \left[\gamma^2 - 4\ln\gamma\right]}$$

The maximal elastic displacement of the slab is:

$$u_{dy} = \frac{a^2 \cdot M_{dy}}{4 \cdot D} \cdot \frac{\left[\left(4 - 3\gamma^2\right) + 4 \cdot \gamma^2 \ln\gamma\right]}{(1+\upsilon) \cdot \left(\gamma^2 - 4\ln\gamma\right)}$$

The plastic resistance R_{dk} of the slab is calculated according to the formula from the yield-line theory of a circular slab clamped onto the periphery and loaded by a unit force concentrated on a circular area:

$$R_{dk} = \frac{2\pi \cdot \left(M^{+}_{dk} + M^{-}_{dk}\right)}{\left(1 - \frac{2a}{3r}\right)}$$

M^{+}_{dk} is the ultimate bending moment which creates tension in the lower fibre,
M^{-}_{dk} is the ultimate bending moment which creates tension in the upper fibre.

The maximal plastic displacement of the slab is evaluated according to the allowed rotation θ_{lim} of the plastic hinge: $u_{dk} = r \times \theta_{lim}$

B.3.2.3. Surrounding structure elastic spring

The paragraph B.2.2.3 applies.

B.4. Use of results

The 3 DOF model allows to calculate the forces and displacements in each element (punching cone, concrete panel and surrounding structure), and to verify directly the absence of break in punching cone and concrete panel.

Then, the resulting forces have to be used for other design aspects:

B.4.1. Shear design of concrete panel

For the shear design of concrete panel, the 1 DOF model of the concrete panel cannot give reliable results, because the shear response is influenced by other self vibration modes than the first one. Therefore, the shear force cannot be derived directly from the global force in the simplified model. A further analysis is needed to give accurate values for shear design.

An acceptable conservative method is to consider that for shear design purpose, the global force transmitted to the concrete panel is equal to the maximum value of impact elastic response spectrum (see appendix D). The shear force distribution may be derived by a static linear calculation.

B.4.2. Design of surrounding structure

The surrounding structure (actually the whole building) has to be designed to resist the transmitted forces due to aircraft impact. The corresponding global force is derived from the simplified model (force in R_1 spring). This global force has to be taken into account in the finite element analysis of the building in order to determine the distribution of forces in the whole structure.

An acceptable conservative method to estimate the global force transmitted to the surrounding structure is to consider that the global force is equal to the maximum value of impact elastic response spectrum (see appendix D).

B.5. General methodology for load drops analysis

Analysis of effects in case of load drop has to be done according to the two following criteria:

- the structure perforation under load impact shall be avoided. The criteria defined in appendix C may be applied for slabs (although the impact speed is generally less than 20 m/s which is the validity limit of the formula),
- the structure bending capacity shall be checked by an energetic method whose principles are defined hereafter.

The principle of energetic method consists to verify that the kinetic energy transmitted to the slab during impact (E_t) can be equilibrated by elasto-plastic deformation of the slab, with a safety factor $\gamma = 2$ (due to uncertainties).

The transmitted energy E_t is derived from the missile kinetic energy is $E_0 = ½\, m_0\, v_0^2$ by the following formula (which takes into account the energy absorption during impact) :

$$E_t = \frac{E_0}{1 + k\frac{m}{m_0}}, \text{ where}$$

- m_0 is the missile mass and v_0 its speed,
- m is the mass of impacted element (circular equivalent slab, beam),
- k is a coefficient depending on the type of impacted element: k = 1/6 for an impact on a slab, k = 1/3 for an impact on a beam.

The global elasto-plastic behaviour law of the slab has to be determined, under a progressive load applied on the impact area until the reach of one of the break criteria. The methods given in § B.2.2.2 and B.3.2.2 are applicable. In addition, shear failure has to be prevented by checking that the maximal load of the global law generates allowable shear forces in the slab. The integration of the global law allows calculating the maximum strain energy (E_{ud}) which can be accumulated in the slab before break. The equilibrium criterion is $E_t < E_{ud} / \gamma$.

APPENDIX C. Penetration of concrete slabs by hard missiles

C.1. Notations

- M Missile mass (kg)
- D Missile diameter (m)
- V Missile speed (m/s)
- H Slab thickness (m)
- f_{ck} Characteristic value of concrete compressive strength (Pa)
- ρ Concrete density (pseudo-parameter used to obtain dimensionless variables) $\rho = 2500$ kg/m^3.

C.2. Validity range

- $0.5 < D/H < 1.5$
- $0.5 < M/\rho H^3 < 5.0$
- 30 MPa $\leq f_{ck} \leq$ 45 MPa

  Note: Concrete with a characteristic strength at 28 days of 25 MPa are accepted as having a sufficient strength at the time of impact; they are included in the domain of validity.
- 100 kg/m^3 < symmetrical reinforcement < 250 kg/m^3
- V > 20 m/s

C.3. Perforation formula

The minimum thickness H of the slab or wall is given by the formulae below:

$$H_{perforation} = 0.8\, f_{ck}^{-3/8}\, \rho^{-1/8}\, (M/D)^{1/2}\, V^{3/4}$$

C.4. Application conditions

The following applies to reinforced concrete and prestressed concrete.

– A 10 % increase of the thickness calculated as above shall be considered for all situations where perforation of reinforced concrete walls and slabs is unacceptable on a safety point of view:

$$H_{acceptable} = 1.1\, H_{perforation}$$

– In other situations, $H_{acceptable} = H_{perforation}$

APPENDIX D. Calculation of static forces equivalent to certain accidental loads (explosion and plane crash)

D.1. Analysis principle

This simplified procedure applies to concrete rectangular slabs which yield in bending prior any other rupture mode, under a transverse action spread over its surface or a large part of it.

The equivalent static force (F) - or pressure - is calculated as: $F = \alpha\ \beta\ F_{max}$
Where:

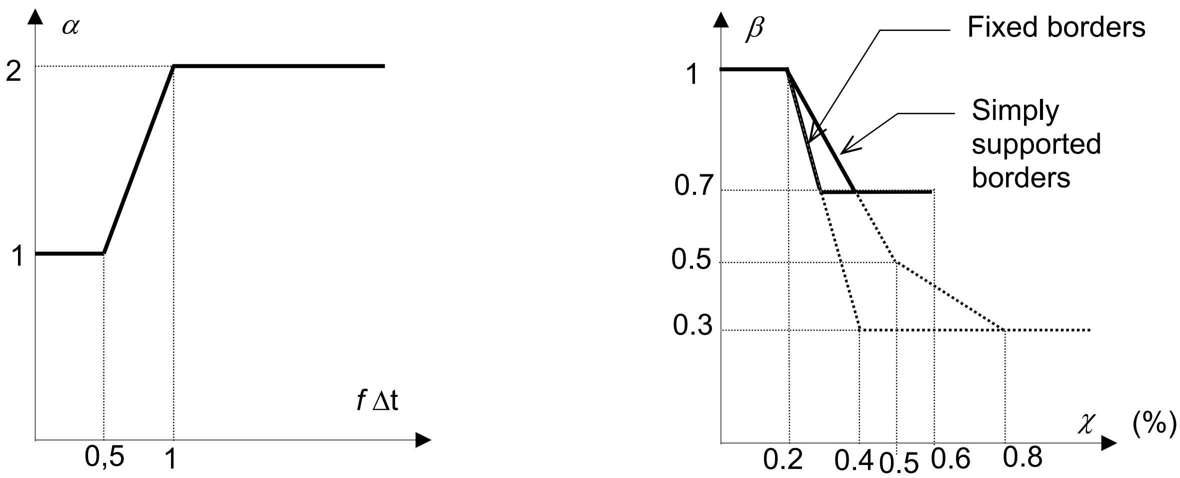
- F_{max} is the maximum value of the time-history loading curve
- α is the dynamic amplification factor
- β is a plasticity factor

D.2. Bending design

D.2.1. Dynamic amplification factor (α)

The fundamental natural frequency *f* of the rectangular slab in bending is first calculated, taking into account the most representative support conditions at borders.

The dynamic amplification factor is then calculated from the time history load. For external explosion with a triangular shape, the factor is given in the figure *D1 a)* below as a function of the product of *f* and the wave duration Δt.



a) Dynamic amplification factor *b) Plasticity factor*

Figure D1: ***Dynamic and plasticity factors for external explosion***

D.2.2. Plasticity factor (β)

β shall be determined by test and/or non linear analysis. In the absence of such results, for external explosion, β can be taken from figure *D1 b)*, where χ is the maximum curvature (m^{-1}) which can be estimated as (*h* is the thickness of the slab):

$$\chi = \frac{\max imum\ steel\ extension}{equivalent\ lever\ arm} = \frac{0.008}{0.9\ h}$$

β shall then not be taken lower than 0.7.

D.3. Shear design

For shear design purpose, α and β factors shall not be less than 1 (each of them), unless a specific dynamic calculation gives more accurate values.

D.4. Application to plane crash

Following the same procedure than above, the static forces equivalent to a plane crash impact include a dynamic amplification factor (figure D2) and a plasticity factor (figure D3), both assuming that the impact is in the centre of the slab or the wall. β shall then not be taken lower than 0.6.

However, to obtain the maximum shear effect, the same impact shall be considered near the supports of the slab or wall; in that case, the plasticity factor shall be taken equal to 1 and the dynamic amplification factor at its maximum value, as given by figure D2.

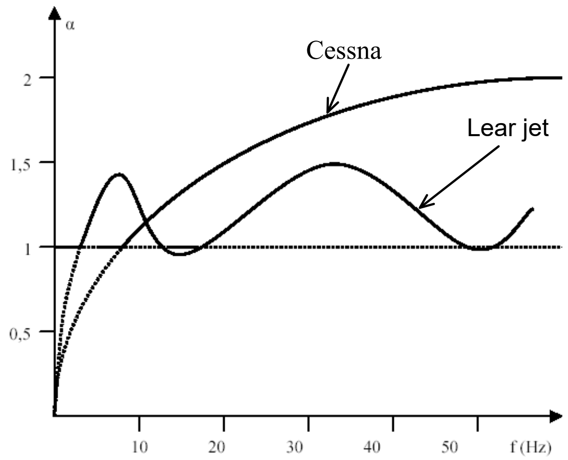


Figure D2: ***Dynamic amplification factors for plane crash.***

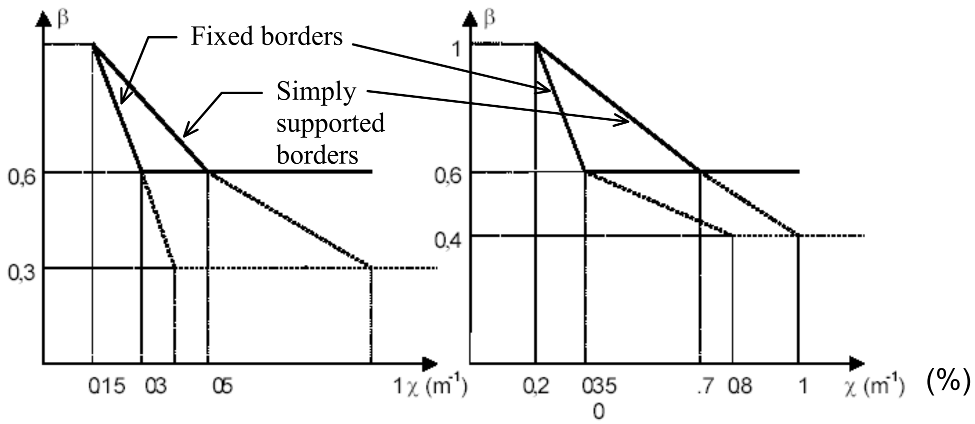


Figure D3: ***Plasticity factors for plane crash***