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Consideration of external events in the design of nuclear facilities other than nuclear power plants, with emphasis on earthquakes

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FOREWORD

The design of nuclear facilities other than nuclear power plants in relation to external events is not a well harmonized practice around the world. Traditionally, the design of these facilities has either been left to the provisions collected in national building codes and other industrial codes not specifically intended for nuclear facilities, or it has been the subject of complex analyses of the type usually performed for nuclear power plants.

The IAEA has recently started a programme of development of safety standards for such facilities. The need to define the appropriate safety requirements for nuclear installations prompted a generic review of siting and design approaches for these facilities in relation to external events. Therefore the assessment methods for siting and design were reviewed by the engineering community to provide the overall design of such facilities with the necessary reliability level.

This report aims to provide guidelines for the assessment of the safety of nuclear facilities other than nuclear power plants in relation to external events through the application of simplified methods and procedures for their siting and design. The approach adopted is both simplified and conservative compared with that used for power reactors. It seeks to provide a rational balance for a suitable combination of sustainable effort in site investigations and refinement in design procedures, compatible with the assigned safety objectives.

This publication is related to IAEA-TECDOC-348 "Earthquake Resistant Design of Nuclear Facilities with Limited Radioactive Inventory" (1985) which focused on the seismic design of nuclear facilities with limited radioactive inventory. After some 17 years, parts of IAEA-TECDOC-348 needed modification, as new operational data have become available from many facilities. In addition, sophisticated design methodologies are now more easily obtainable, and experts felt that the trade-off between sustainable investment in the facilities and design conservatism had to be redefined.

A large number of consultants from various States were involved in updating IAEA-TECDOC-348 and in reviewing draft sections of the present publication — their efforts are greatly appreciated.

The contributions of M. Lebelle (France) and T. Fukuda (Japan) are acknowledged. The IAEA officers responsible for this publication were P. Contri and H. Tomura of the Division of Nuclear Installation Safety.

CONTENTS

1. INTRODUCTION

1.1. BACKGROUND

The safety of a nuclear installation requires in general that it be sited, designed, constructed and operated to protect individuals, society and the environment against an uncontrolled release of radioactive material. External events (i.e. relevant to natural and human induced hazard) play a major role in challenging the plant defence and therefore appropriate design provisions need to be taken to guarantee an adequate safety margin in case of such events.

In recent years the development of design criteria, design methodologies and assessment approaches for the implementation of the general safety principles received major emphasis for nuclear power plants, while other nuclear installations received less attention even if their radioactive inventory was comparable with NPPs, as is the case for some research reactors. For fuel reprocessing plants, the associated risk for radiological and chemical contamination may be rather high, and for many research reactors associated with universities and research centres, their location is often very close to very densely populated areas.

This report aims at collecting the most relevant experience in Member States for the design of nuclear installations other than NPPs as set forth in Section 1.3, providing a coherent framework where safety aspects, site investigations, design procedures and assessment methodologies are presented in an engineering, ready to use context.

Many IAEA publications address siting and design of nuclear installations other than NPPs in relation to external events, namely:

- Design of spent fuel repositories are discussed in IAEA Safety Series G-3.1 and G-4.1 [1, 2] and in technical publications such as Refs [3, 4].
- Design, Safety Assessment and Operation for Research Reactors are discussed in Safety Series No. 35 [5–8] and in a number of TECDOCs [9–12].
- Safety requirements and design procedures for fuel cycle facilities are discussed in [4, 13–15].

However, these publications do not specify the methodology to be used in the design process and therefore, for the objectives of this report, reference is made to the more rigorous approach applied for the power plants, particularly to the following requirements and Safety Guides:

- Requirements for NPP site evaluation [16].
- *Evaluation of seismic hazard is presented in Safety Guides [17].*
- *Evaluation of human induced hazard is presented in Safety Guide [18].*
- *Evaluation of flood induced hazard is presented in Safety Guide [19].*
- Evaluation of hazard induced by extreme events is presented in Safety Guide [20].
- Requirements for NPP design [21].
- Seismic design is presented in [22].
- Design provisions in relation to external events are presented in [23].
- Design provisions for foundations are presented in [24].
- Safety assessment is presented in [25].

The specific design of the nuclear facilities other than NPPs was first addressed by the IAEA in 1985 with the publication of IAEA-TECDOC-348, Earthquake Resistant Design of Nuclear Facilities with Limited Radioactive Inventory [12], mainly oriented to seismic siting and design. The report was widely applied in many States and often it was also considered as contractual document for some research reactor design.

That publication presented simplified siting and design methods aiming at minimizing sophisticated calculations and emphasized the importance of construction and detailing principles. It proposed a simplified design approach alternative to the complicated and sophisticated methodologies which are associated with and borrowed from nuclear power plant analysis and design.

Such broad use of [12] in many applications in the world recently suggested a review of the original report, mainly due to the following, additional, technical reasons:

- Evidence from available operating experience: through databases like INES [26], an analysis was carried out at IAEA on the 43 events recorded since 1991 in research reactors, reprocessing plants, fuel fabrication plants, laboratories and accelerators. Among them 10% showed high off-site radioactive contamination, with a sharing among the different causes that is shown in Figure 1.

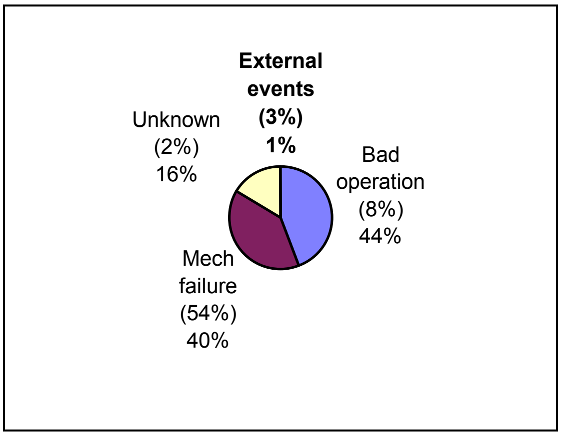

FIG. 1. Causes of INES events in NFOPs. (number in brackets are referred to NPPs, for comparison).

The analysis of available data shows the low incidence (in terms of number of events) of external events (3%), in the causes of nuclear events, compared with mechanical failures (40%) and bad operation (44%). However, the analysis of the consequences of such events clearly shows how external events (earthquakes, flood, extreme temperatures, lightning, etc.) are often associated to the most challenging degradation scenarios of the defence in depth barriers.

It needs to be recalled that the sample is very small and covered a short interval. Furthermore, the nuclear facilities other than power plants (NFOPs) are often in the most secure areas of the States for strategic reasons and some events probably have not been reported because of security.

- The emphasis on mechanical failures highlighted in the previous statistic shows also the importance for stringent criteria in component qualification.Feedback from applications in Member States: in most cases a combination of national codes and standards for conventional or industrial facilities is applied as a design basis, often very different for civil structures, mechanical components and equipment design, with the consequent impossibility to evaluate the safety margin of the facility.
  Moreover, little emphasis is usually given to the safety classification and accident analysis (safety analysis reports are in fact available for very few facilities) with the consequence of a generic difficulty in the safety assessment of the facilities.
- The availability of new and cheap design methodologies and computer codes. Modern design methodologies allow cheap and reliable design processes with limited investment of time and money and therefore require new definition of the optimal balance between design conservatism and capital investment mentioned above.
- The growing emphasis on requalification of existing facilities. In the case of research reactors, among 265 units in operation (2000), 50% are older than 40 years, with obvious implications in terms of ageing evaluation and requalification.

A first review of the publication [12] was then launched (1998) with the following targets:

- To update the structural typologies to be addressed, their particular design issues and relevant design limits.
- To update the proposed design methodologies in order to be consistent with current cost–benefit ratios and required accuracy level.
- To better specify the field of application of each procedure.
- To add a graduation in siting and design procedures, trying to provide an average engineering measurement of the introduced conservatism.
- To provide a minimum validation of the proposed assumptions.

Moreover, recently (1999 and 2000) an increased concern for such facilities was expressed in two IAEA Consultant meetings: the previous targets were confirmed and it was suggested to extend the scope of the publication to all external events (i.e. flood, aircraft crash, explosions, etc.) and to all facilities with similar structural design principles.

The following key ideas have driven the preparation of this report:

(1) The safety objectives defined for nuclear power plants need to also be valid for other nuclear installations and therefore reference is made to the basic IAEA publications for their presentation.

(2) A so-called ‘graded approach’ in the design of nuclear installations other than NPPs (NFOPs) is normal practice in Member States and therefore needs to be considered. Different values for the probability of exceedance of design basis events might be used in nuclear facilities according to their radioactive inventory (in general different than NPPs), to the potential for dispersion in the environment and to other characteristics of the radiological risk associated to external events.

The investment required for the construction and operation of a NPP is usually some order of magnitude higher than for other nuclear installations. Therefore in these latter facilities the extension of the site investigation campaigns, the use of expensive design procedures and the implementation of expensive QA systems in design, construction and maintenance needs to be reconsidered to provide a sustainable approach able to guarantee a safe operation.

A common approach for NFOP design is the acceptance of a higher conservatism in all the mentioned phases, able for example to exclude some sites or to compensate, in the design phase, for the discrepancies between simplified numerical models and the real behaviour of the structures in case of external events.

Such conservatism has to be carefully evaluated in order to guarantee that the final design incorporates the required safety margin.

This report is the final result of such efforts and it confirms the validity of the structure of the old publication [12], trying to update and extend its technical content, coming to a completely revised publication on the subject.

1.2. OBJECTIVE

The main objective of this report is to provide information on a rational basis for a safe design of nuclear facilities other than NPPs in relation to external events. Simplified methods and procedures are presented here to provide enough conservatism in siting and design processes in order to avoid the complicated and expensive procedures typical of the NPPs. Such an approach is in line with the more limited resources typically associated with NFOPs construction and operation.

The report aims also at providing a consistent basis for the assessment of the safety objectives assigned to the NFOPs, for the selection of appropriate design standards and codes consistent with the required safety margin and for the preparation of sound purchasing specifications for equipment and components.

However, the report is not intended to be an alternative to design codes: its emphasis is on safety issues related to the design and therefore it merges information for a safe design with suggestions for 'quick and easy' analytical methodologies.

The overall safety evaluation of the NFOPs needs to refer to the IAEA safety standards.

Although the publication was not originally developed with reference to the re-evaluation of existing facilities, most of its content can now be usefully applied to such assessment.

1.3. SCOPE

This publication is intended for use in siting and design of facilities other than NPPs in relation to external events. Qualification of components and equipment is not explicitly addressed.

In particular, the following nuclear facilities have been identified, as potential targets for this report:

(1) Surface facilities of uranium mines, mill factories and uranium tail repositories
(2) Fuel conversion plants
(3) Enrichment plants
(4) Fuel fabrication plants (including MOX fuel facilities)
(5) Research reactors
(6) Radioisotope laboratories
(7) Independent spent fuel storage facilities
(8) Fuel reprocessing plants
(9) Near surface repositories for low and intermediate level waste
(10) Deep waste disposal for permanent storage
(11) Heavy water distillation plants
(12) Accelerators.

However some of them have been excluded from the present report, according to the following set of criteria:

- Peculiar structural typologies or layouts, not comparable with the rest of the facilities: earth structures, disposal mounds (cask, barrels and earth structures, covered with soil materials, but without a bearing structure, like No. 9), underground structures and mines (like No. 1), etc.
- Structures with atypical layout and configuration, impossible to be categorized and rather unique in the world, like Nos. 11 and 12
- Facilities with peculiar safety objectives (very long term disposal, strong geological implications, decommissioning framework, etc.), like No. 10
- [Excluded by the Statute]

Therefore, only facilities of the type 2, 3, 4, 5, 6, 7, 8, meeting the above criteria are considered in the scope of the present report.

In any of the selected facilities, some special equipment is featured which cannot be designed according to the general principles stated in this report because of its peculiar functions or atypical configuration. The reactor core in research reactors, the centrifuges in the fuel enrichment plants are two typical examples.

This report defines the principles for classification and qualification of these critical components, but does not provide details for their design, installation and operation (e.g. qualification of welding, internal tolerances, etc.). Appropriate, specific design codes need to be used in these cases.

Moreover, due to the general objectives of this report, the present publication is focused on the structural design, which is usually the most expensive segment of the whole design process.

Other critical tasks of the design, like equipment capacity evaluation (with reference to all the acceptance criteria defined in Chapter 2, from stability to operability and interaction), geotechnical assessment, QA, monitoring, etc. are not completely addressed in this report. Either such tasks are not usually associated with high efforts or they cannot be simplified, even with conservative approaches, and therefore are not relevant for the objectives of this report. For guidelines and recommendations concerning the tasks not explicitly mentioned here, general conventional standards or nuclear safety standards are applicable, according to the risk classification of the facility of interest.

Concerning the applicability of this report to facilities with high radioactive inventory, it may also be applied to the design of critical and subcritical assemblies (hereinafter included under the term research reactor) to the extent that is appropriate for these facilities. Therefore, research reactors with power above several tens of megawatts, fast neutron spectrum research reactors, and small prototype power reactors may require additional measures and the use of techniques for power reactors may be more appropriate.

Further guidance on the appropriate application of the simplified approaches considered in the present report is provided in Section 3.

The resulting set of facilities addressed in this report is identified in the following as NFOP (nuclear facilities other than NPPs).

External events affecting a nuclear installation could be listed as in the following, according to the relevant safety standards for NPPs [23]:

(a) Human induced

- Aircraft crashes.
- Explosions (deflagrations and detonations) with or without fire, originated from off-site sources and on-site (but external to safety related buildings), like storage of hazardous materials, transformers, high energy rotating equipment.
- Release of hazardous gas (asphyxiant, toxic) from off-site and on-site storage.
- Release of corrosive gas and liquids from off-site and on-site storage.
- Fire generated from off-site sources (mainly for its potential for smoke and toxic gas production).
- Collision of ships and floating debris (ice, logs, etc.) with the water intakes.
- Electromagnetic interference from off-site (e.g. from communication centres, portable phone antennas) and on-site (e.g. from the activation of high voltage electric switch gears).
- Any combination of the above as a result of a common initiating event (e.g. explosion with release of hazardous gases, smoke and fire).

(b) Natural

- Earthquakes.
- Extreme meteorological conditions (temperature, snow, hail, frost, subsurface freezing, drought).
- Floods (from tides, tsunamis, seiches, storm surges, precipitation, waterspouts, dam forming and dam failures, snow melt, landslides into water bodies, channel changes, work in the channel).
- Landslides and avalanches.
- Cyclones (hurricanes, tornadoes and tropical typhoons).
- Abrasive dust and sand storms.
- Lightning.
- Volcanism.

This list is not exhaustive and other external events, not included in the list, may be identified and selected as design basis external events at the site.

However, particularly in case of natural events, some scenarios need to be treated as exclusion criteria for the site itself (e.g. local volcanism, local capable fault, aircraft crash, etc.) and therefore they are not discussed here. Such exclusion criteria are usually more restrictive than those for the NPPs, due to the different balance point between investment and risk.

However, in case when NFOPs are located at the same site of a NPP, they share the same hazard and therefore in these cases reference need to be made to the respective Safety Guide for NPPs.

Other scenarios are addressed preferably through site protection features (e.g. site drainage, protecting dams and levees, etc.) rather than with plant design measures and therefore they are mentioned here.

Furthermore, considered external human induced events are of accidental origin. Considerations related to protection of the plant from malevolent action by third parties are outside the scope of this report, but they could easily be accommodated into the approach for human induced events (such as explosions, aircraft crash, etc.).

In general, this report does not provide comprehensive guidance on design for any of the mentioned external events. In fact, in many cases the same provisions applied for NPPs need to be implemented for NFOPs (like in the case of extreme meteorological events, extreme winds, etc.) without particular effort and therefore they are not mentioned here. In some other cases, like for earthquakes or for aircraft crash, the design procedures usually require rather high investments and therefore they are in the main scope of this TECDOC, which proposes simplified approaches for their analysis.

In conclusion, in the following, main emphasis is given to design against earthquake, flood and aircraft crash which are usually associated with the most ‘expensive’ protecting features in siting and design, while other events are discussed only for their specific effects on NFOPs safety.

1.4. STRUCTURE

This TECDOC consists of nine sections and seven appendices.

Section 2 presents the safety objectives to be met during the design of the NFOPs considered under the scope of this report and the acceptance criteria usually considered for design. In addition, this Section shows the safety requirements developed to meet the objectives and to demonstrate the acceptability of the design. These are used to classify the facilities and to categorize their structures, systems and components to facilitate the straightforward application of the different methods included in the present report. Section 3 presents the basis for the design approach and component qualification, in line with the safety concepts defined at Section 2. Section 4 deals with the design basis definition for a plant: from the ground motion (earthquake) to other hazards. Based on availability of instrumental and/or historical data, procedures are suggested for the definition of a reliable design input. Section 5 contains guidance for site investigations, particularly of geological and geotechnical, including simplified methods for soil liquefaction analysis. Section 6 presents some simplified methodologies for the seismic design of building structures. Sections 7–8 deal with design of equipment and piping with special emphasis on structural detailing and anchoring. Section 9 presents some safety principles for the installation and operation of monitoring systems at the site, their quality assurance and data processing procedures.

Appendices present experience of some States in design and qualification of structures and components that can be used to solve specific design problems of NFOPs.

2. SAFETY OBJECTIVES AND CLASSIFICATION

2.1. INTRODUCTION AND EXAMPLES OF NFOPs

In Table I, the results of a very short analysis of the NFOPs in the scope of this publication is presented, as background information for the identification of the most common safety requirements for such facilities. The analysis is mainly based on INES data [26] and bibliographic results [27].

TABLE I. EXAMPLES OF NFOPS WITH ASSOCIATED MAIN SAFETY ISSUES (source: analysis of SARs and bibliographic references)

Facility	Main structural characteristics	External Event design	Safety issues	Notes
Fuel conversion plants	Many process buildings + support structures (pipe bridges, cooling towers, chemical tank farm, boiler, diesel fuel storage, electrical substation, settling basin)	Usually 100 year return period	Radiation Chemical hazard Fire, explosion Physical interaction	Design for decontamination and decommissioning Large quantities of radioactive waste
Fuel enrichment plants	Process bldg (centrifuges) + feed and withdrawal bldg, plant support facilities and utilities	Usually 100 year return period	Criticality Radiation Chemical hazard Fire, explosion Physical interaction	
Fuel fabrication plant	Process bldg + plant support facilities and utilities	Usually 100 year return period	Criticality Radiation Chemical hazard Fire, explosion Physical interaction In case of MOX, highest concern for the Pu[?] toxicity	Confinement in case of accident Full functionality during and after DBE
Fuel reprocessing plants	Process bldg + waste treatment + plant support facilities and utilities	Same return period for external events as in the NPPs	As above.	Large quantities of hazardous waste
Independent fuel storage facilities (for NPPs and	Storage pool + transportation systems + plant support facilities and utilities	Sometimes same hazard of NPPs, sometimes	Radiation Fire, explosion	Leaktightness is required to pools (wet solution) Storage of casks (dry

reprocessing plants)		100 years return period		solution)
Research reactors	Reactor bldg + plant support facilities and utilities Containment in some cases		Criticality Radiation Fire, explosion	

In addition, recent surveys [28] have been carried out on accidents at NFOPs, also following Tokai Mura accident, where the following points were highlighted.

- Since 1945, nearly 60 criticality accidents of varying degrees of severity have occurred worldwide, mostly at military related sites in the USA and the former Soviet Union. All but two of them – prior to Tokai Mura – took place before the early 1980s. The two most recent both took place at Russian military related facilities in 1997.
- Of those before the early 1980s, 36 (two thirds) occurred either in research reactors or in laboratories working on 'critical assemblies'. None of them resulted in any significant release of radioactive material to the environment, but there was a total of nine fatalities. One further person died in an incident in 1997. The causes and consequences of each accident varied according to specific operational circumstances at the various facilities.
- The others, one third (21), occurred in nuclear fuel cycle facilities. Seven people died, including one in 1997, and 40 more received significant exposure to radiation. Again, there was never any significant release of radioactive material to the environment. Twenty out of 21 occurred in US or Soviet facilities – the exception was a 1970 incident in the United Kingdom. Twenty out of 21 also involved liquid solutions of fissile materials, and none of them involved either failure of equipment and/or material or faulty calculations.
- The main cause appears in most cases to have been a failure to appreciate the number of possible accident scenarios, particularly bearing in mind the potential for human error.

Other statistics, focused on criticality, are available in Ref. [29] with many details oriented to prevent similar accidents in the future.

Concerning research reactors, data are available in Refs [30, 31].

As a result of these surveys on the main facilities in the scope of this TECDOC, aimed at the identification of the most common safety concerns for such facilities, the most recurrent accident scenarios in NFOPs were identified as:

- Criticality
- Fire and explosions
- Chemical hazard: release of radioactive effluents or prevention of safety related operator actions
- Physical interactions among components: drop loads, impairment of operator actions, particularly as a consequence of external events
- Internal misoperation with radioactive material spreading on-site and off-site.

These scenarios might be strongly affected by plant conditions that in the case of NFOPs could be very critical, particularly in relation to:

(1) ageing (some facilities are very old and designed for different purposes, including support to military oriented tasks),
(2) maintenance (standard of maintenance have proven to be poor in many cases for NFOPs)
(3) decommissioning procedures (often, extensive facility contamination and presence of many sources of chemical hazard can complicate the process)

According to this basic survey carried out on the most significant facilities, it comes out that the most common requirements for structural design of buildings, components, equipment and distribution systems can be summarized as follows:

- Confinement or isolation (ventilation systems): leaktightness
- Leaktightness of pools, through liners or prestressing
- Avoidance of drop loads
- Functionality of safety related items (e.g. heat removal, ventilation, fire control, etc.)
- Appropriate anchoring of items
- Avoidance of interaction problems: mechanical, chemical or through impairment of operator action
- Radiological shielding.

2.2. SAFETY OBJECTIVES

NFOPs need to meet the general safety objectives as stated for NPPs [21].

The **general safety objective** stated for all nuclear installations is to protect individuals, society and the environment from harm by establishing and maintaining in nuclear installation effective defences against radiological hazards. This general nuclear safety objective is supported by two complementary safety objectives dealing with radiation protection and technical aspects.

The **radiation protection objective** is to ensure that all operational states radiation exposure within the installation or due to any planned release of radioactive material from the installation is to kept below prescribed limits and as low as reasonably achievable, an to ensure mitigation of the radiological consequences of any accident.

The **technical safety objective** is to take all reasonably practicable measure to prevent accidents in nuclear installations and to mitigate their consequences if they occur: to ensure with a high level of confidence that for all possible accidents taken into account in the design of the installation, including those of very low probability, any radiological consequences would be minor and below prescribed limits; and to ensure that the likelihood of accidents with serious radiological consequences is extremely low.

To achieve the above mentioned safety objectives in case of an external event (EE), the design of the facility needs to prevent any damage to structures and equipment that could lead to significant exposures to facility personnel or members of the public because of uncontrolled release of radioactive material.

The consequences to workers and public are discussed in Ref. [12].

In the case of the public, the estimated average doses to the relevant critical groups of members of the public shall not exceed the following limits [32]:

(1) An effective dose of 1 mSv in a year;

(2) In special circumstances, an effective dose of up to 5 mSv in a single year provided that the average dose after five consecutive years does not exceed 1 mSv per year;

(3) An equivalent dose to the lens of the eye of 15 mSv in a year; and

(4) An equivalent dose to the skin of 50 mSv in a year.

In the case of workers, the consequences need to be expressed in terms of occupational exposure. The limits are the following:

(1) An effective dose of 20 mSv per year averaged over five consecutive years;

(2) An effective dose of 50 mSv in a single year;

(3) An equivalent dose to the lens of the eye of 150 mSv in a year; and

(4) An equivalent dose to the extremities (hands and feet) or the skin of 500 mSv.

2.3. DEFENCE IN DEPTH

The application of the defence in depth approach to nuclear power plants is well defined in engineering tradition (see for example [12, 33]), with a specified number of barriers, levels and features.

For NFOPs, the number of levels could be maintained, but the number and role of the barriers could be different from that of a power plant, depending very much on the accident scenarios and the reliability that may be associated with any defence level. There is no unified approach in Member States and therefore only a case by case approach supported by some probabilistic evaluation seems to be the only strategy to be followed.

However, some guidelines can be provided for a reasonable implementation of the approach to NFOPs:

- One of the basic ideas of the defence in depth approach is a proper balance between *prevention* of an accident and *mitigation* of its consequences. This balance needs to be applied also to NFOPs to avoid design strategies in which ‘low’ radioactive inventory facilities are designed only with reference to mitigation measures for potential accident scenarios.
  In other words, the low potential for radioactive contamination (i.e. a small ‘source term’) is not to be used to justify a low number of provisions on the ‘prevention’ side. The ALARA principle in fact aims at protecting also the workers at the site who deserve a prevention policy against radioactive contamination.
- The *reliability* of the levels of defence in depth dealing with design measures is implicitly guaranteed by the application of the ‘single failure’ criterion. In NFOPs single failure may be applied to systems (as it is done for NPPs) and also to items in some cases, but its consequences on the design need to be considered together with the selected QA approach for design, construction and operation, with maintenance procedures and with the probability associated to the initiating events.
  Particularly for *active systems*, redundancy, independence, fail-safe design and

appropriate auxiliary services can achieve the required reliability levels with reference to random failures in conjunction with appropriate maintenance procedures. The balance has to be defined for the specific facility. In any case, the operator cannot be considered as a redundancy due to the consideration of human errors.

Passive systems (typically structures, vessels, support devices, pipelines, etc.), are usually designed to perform their safety functions within the prescribed operating life only on the basis of appropriate QA, design standards, maintenance and surveillance procedures. Such assumption needs to be discussed in the SAR and reflected into appropriate procedures covering the plant lifetime.

- With special reference to some external events (typically earthquake, flood, aircraft impact, etc.), also common cause failures may have a high influence on the reliability of the defence in depth levels. Therefore, a special assessment needs to be carried out, independently from the application of single failure criterion, aiming at a proper balance of quality, segregation (physical separation) and diversity requirements to safety related items.

  A typical example concerns seismically classified items where a high level of quality (i.e. of qualification requirements) can guarantee an appropriate safety level where redundancy or diversity may lead to less effective solutions.
- In case of very rare events (e.g. aircraft crash) or combination of events (also internal and external scenarios), particularly when enough warning time is guaranteed (e.g.: heavy storms), or in a re-evaluation of an existing plant, many Member States accept a reduced level of defence which is limited to guarantee the shutdown of the nuclear reaction potentially leading to radiological accidents. Such limitation requires a critical interaction with operating procedures (warning, monitoring, operator actions, inspections) that has to be defined on a case by case basis.

Guidance for a proper application of these concepts can be found in Ref. [34].

2.4. SAFETY CLASSIFICATION OF THE FACILITIES

As a preliminary screening criterion for the NFOPs to be considered as facilities with associated radiological risk (in case of external events), it can be assumed that a conservatively estimated amount of the radioactive inventory is released following an accident (induced by an external event).

In case the screening shows that the radioactive inventory of the facility is ‘not significant’ (i.e. the criterion for the limit to the public is met), conventional standard for the design of the facility can be applied, according to the national codes for industrial facilities (of course a parallel evaluation needs to be carried out for other risks, like chemical, fire, etc.).

In case the limits for workers are exceeded, local engineering provisions need to be put in place for their prevention and mitigation. Conversely, if the release is ‘significant’, an evaluation of the risk associated to the facility needs to be carried out to define an appropriate safety margin to be associated with the design for EEs. Risk is defined as the product of the probability of the initiating event (external) with a measure of the consequence generated by a radioactive release.

The evaluation of the risk posed by a facility needs to analyse continuous emissions, accidental releases from operation, transportation or human induced events and from natural hazards. It can be health related or environmental related.

The targets of the risk are firstly the workers at the site, the people living near the site, the people affected by the transportation of contaminants through air, waterways or by contamination of agricultural products. Secondly, also the ecological systems need to be considered for their influence on the food chain.

Many procedures have been developed for risk analysis, with emphasis to risk comparison among different sources in the society and between voluntary and involuntary causes. Comprehensive information on the state of the art is collected in Ref. [35]. For the objectives of the present publication, only the radiological risk is addressed and only the risk induced by external events on the production facilities, without analysis of the effects associated to the waste which is supposed to be treated and transported according to specific national regulations.

Following the definition of risk given above, the first term, i.e. the probability of any EE at the site, needs to be evaluated according to its site specific hazard: detailed information is provided in this publication for the most relevant EEs in a graded form, with reference to [16].

The second term of the risk equation, i.e. the probability that EE generate a radiological consequence depends on characteristics both of the source (facility) and of the events, such as:

- Amount, type and status of radioactive inventory at the site (e.g.: solid, liquid, processed or just stored, etc.)
- Intrinsic reliability and hazard associated to the chemical and physical processes which take place in the facility (e.g. processes which implies transportation of hazardous substances might show a higher hazard than cases when fuel is not moved, like in operating NPPs)
- Installed thermal power of the facility
- Configuration of the facility for different kinds of productions
- Concentration of radioactive sources in the plant (e.g.: in the power reactor, most of the inventory is in the reactor and in the fuel pool, while in processing plants it is distributed in the layout)
- Facilities designed for experiments and research (such activities have an associated intrinsic unpredictability) or in any case subjected to frequent configuration and layout changes (such as activities associated to new product developments)
- Need for active safety systems to cope with mitigation of postulated accidents; amount of engineering features implemented for preventing and mitigating serious consequences from accidents
- Possibility of installation of warning systems able to detect in time the potential unfavourable development of an event (e.g. meteorological events vs. aircraft crashes)
- Characteristics of the process or of the engineering features which might show a ‘cliff edge effect’ (potential small deviations in plant parameters giving rise to severely abnormal plant behaviour) in case of an accident, without possibility to prevent the degeneration into radiological consequences
- Characteristics of the event (e.g. wind and explosion have a high potential for dispersion, while earthquake and aircraft crash have a minor contribution on the dispersion)

- Environmental characteristics of the site relevant to dispersion (e.g. windy area, coastal site, etc.)
- Easy implementation of emergency planning in relation to the event: access to the site, evacuation routes availability, time delay between accident and releases, etc.
- Potential for long term effect in case of contamination (long lived radionuclides, persistent effect in the environment)
- Number of people potentially affected by an accident at the facility
- Potential for off-site versus on-site radiological contamination.

A general evaluation of the risk associated with the NFOPs is difficult due to the high number of variables depending on the specific layout. In general a reasonable and reliable risk classification can be made only on a case by case basis, possibly after a detailed PSA analysis (usually not available at the design stage). Also generic probabilistic–deterministic procedures have been developed, mainly experience based, for hazard identification, accident modelling, effect propagation and response evaluation. Some indications on the main tasks are contained in [35, 36].

In the framework of this report, mainly oriented to an identified group of facilities and risks, the issues listed above could be interpreted as criteria for such risk classification, driving the final evaluation of the risk associated to the facility, ranging from a minimum risk (conventional building) to the highest values (nuclear power plants).

However, a full probabilistic approach in the classification of NFOPs in relation to the risk they pose to the environment is in general very difficult to be applied because of the large variety of designs, the wide range of power levels, the different modes of operation and purposes of utilization, the particularities of the site, and differences among operating organizations at the facility. Because of this, there is no consensus among the Member States both on the facility classification and on its effects in the design.

A reasonable and very simplified approach could account only for a reduced number of the criteria described above, such as:

- **Class 1 (high hazard)**: potential for significant off-site radiological contamination
- **Class 2 (moderate hazard)**: potential for significant on-site radiological contamination, with high criticality hazard
- **Class 3 (low hazard)**: potential for significant on-site radiological contamination
- **Class 4 (conventional hazard)**: ‘industrial risk’, conventional industrial buildings.

Following this approach, for some facilities the application of the mentioned criteria for facility classification shows a strong correlation between risk associated with the facility and its installed power or radioactive inventory only, instead of all the criteria described above. This correlation might simplify the classification process at the beginning of the design. Of course such assumptions need to then be assessed in the safety assessment phase and justified in the SAR.

According to this criterion, for example, research reactors with powers of several tens of megawatts, fast neutron spectrum reactors or small prototype power reactors can be classified as nuclear power plants and they are beyond the scope of this TECDOC in the sense that their design needs to follow the prescriptions for NPPs (i.e. the relevant safety standards).

Fuel reprocessing plants are usually classified in the highest class of risk, but the engineering features are very different from NPPs and therefore they are included in current discussion.

Most of the remaining facilities can be classified in Class 2 or below following the criteria described above and therefore their design could follow the guidance in this report.

The classification of the facility has influence on the following tasks through a decisional process which is not unified among the Member States:

- Selection of the annual exceedance probability for any EE at the site: higher values can be used for lower class facilities (see Section 4).
- Need for a site specific hazard evaluation: the higher the class of the facility, the more specific it has to be (see Section 4).
- The extension of site investigation campaigns: the higher the class of the facility, the more accurate they have to be (see Section 5).
- Definition of the safety margins in the design or, in general, of the reliability levels that safety functions need to comply with (see below and Sections 6, 7).
- Number of safety features: number and quality of defence in depth barriers and levels, emergency systems, emergency procedures, etc. (in this case, also engineering tradition, construction practice, economic consideration have strong influence). In particular the safety classification of systems and items (including their acceptance criteria in case of an external event) is a very important task affected by the facility classification and assessed by the Safety Analysis Report (SAR).
- Adoption of different QA level in siting, design, construction and operation.

With the support of these concepts, NFOPs need to be sited, designed, constructed and operated so as not to exceed the recommended limits in all operational states and accident conditions.

In many cases, it is convenient to develop some conservative assumptions in order to avoid, even for the highest classes of facilities, expensive investigation campaigns, hazard studies and design processes. However such conservatism is not to be confused with the requirements in terms of safety discussed above and associated to the facility classes: conservatism implies reduced efforts in the design stage, while a different classification in terms of safety is related to a different amount of safety margin. The analysis of the interaction between the two aspects, namely the safety classification of a facility and the conservatism required by a simplified design and siting procedure, is the main target of this report and it is discussed in the following chapters, for every task.

For all nuclear installations, including NFOPs, a safety analysis report is prepared which includes safety analysis of the radiological consequences initiated from external events. These analyses need to include assumptions concerning the safety functions associated with items important to safety which have to remain operable to keep the consequences within the above mentioned radiological limits.

In particular, for research reactors, annex to Ref. [5] presents selected safety functions associated with items important to safety (e.g., buildings and structures, reactor core and internals, fuel matrix and cladding, reactivity control system including the reactor shutdown system, reactor coolant primary circuit, emergency core cooling system, ventilation systems) which may be challenged by the occurrence of an earthquake.

2.5. CLASSIFICATION OF ITEMS IN RELATION TO EXTERNAL EVENTS

Following the general criteria defined above for the classification of the facilities according to the risk they pose to environment and population, the safety analysis of the plant needs to assess number and characteristics of the safety systems (including the barriers to prevent radiological accidents) through analysis of the consequences of any postulated initiating event (including external events) with application of the 'single failure' criterion to any safety group (see [21] for definitions). Special limitations to redundancy and diversity may be considered for passive systems, as specified in [21].

The safety analysis needs to be mainly based on the application of the defence in depth, aimed at maintaining the effectiveness of the physical barriers placed between radioactive materials and workers, the public and the environment.

Special care needs to be used in the application of the 'common cause failure' concept in the safety classification as external events may affect many items at the same time (the case of the earthquake is typical).

The design of systems and components in relation to external events (EE) requires a classification with the following aims:

(1) the identification of items and systems important to safety. Items needed to perform all actions required for the postulated initiating events have to be identified to ensure that the radiological limits are not exceeded, even in case of degeneration into a plant accident. Items which would lead to radiation exposure to workers or public in case of failure need to be added to such category. These items have to be protected in case of an EE.

(2) the identification of the safety functions assigned to them, and therefore their acceptance criteria (e.g. integrity, stability, operability, etc.). The design has to meet such acceptance criteria.

For a rational basis in the design, such information might be organized through an EE classification process of all plant items in order to identify the items who need to be considered in case of any EE and the relevant requirements.

To this aim, three classes of structures, systems and components (SSCs) can be identified:

- **External Event Class 1 (EEC1)**: safety systems pertaining to EE safety groups [21] or safety systems which, during and after an EE, interact with items in the safety group of the EEs
- **External Event Class 2 (EEC2)**: safety systems which are not in the EE safety groups and which, during and after an EE, do not interact with EEC1 items.
- **External Event Class 3 (EEC3)**: items not important to safety which could impair proper functions of EEC1 and EEC2 items or the operator action.

Acceptance criteria need to then be stated for both the operational state of the facilities and the accident conditions considered in the design of the facility.

Such criteria vary among Member States: they may include considerations such as those listed below.

(a) Radiological criteria, such as:

- ALARA levels
- Dose limits (or targets) for facility staff and workers at the facility site and the general public;
- Release limits to the environment; and
- Risk criteria (where applicable).

(b) Performance criteria, including

- Limits to damage of the physical barriers
- Limits to damage of safety significant structures, systems and components
- Frequency limits for certain anticipated operational occurrences and for particular accident conditions, including frequency limits for significant damage of physical barrier (where applicable).

These criteria need to define relevant behaviour limits for any component and structure in any EE class above according to the safety function which is associated with them in case of a sequence where an external event is the PIE.

It turns out that the EE classification is not related to the kind of behaviour limit (acceptance criteria) associated with the component/structure, but it is related only to the amount of safety margin required (Fig. 2). The more relevant a component is for safety; the highest safety margin has to be considered in its design. Therefore, the classification aims at the identification of the safety margin to be applied to any safety function (horizontal arrow in Figure 4), with grading from the highest associated radiological risk to the lowest. In this framework, the classification does not mix the behaviour limit (elasticity, integrity, etc.) with the required amount of safety margin (vertical arrow in Figure 2).

In probabilistic terminology, the deterministic concept of 'safety margin' needs to be related with the probability of exceedance of the design limit. However, in this report only reference to the safety margin needs to be made, due to the main objectives stated in Section 1.

This safety margin needs to be intended in a general sense as it might include design safety margins, but also qualification requirements, requirements for redundancy and diversity, reliability evaluations, number of safety features, QA prescriptions etc., as described above.

Moreover, the EE classification should not have any influence in the application of a specific methodology for the numerical simulation, as it was in the past [12]: numerical simulation techniques need to be validated in terms of their simplified assumptions and therefore in terms of their conservatism.

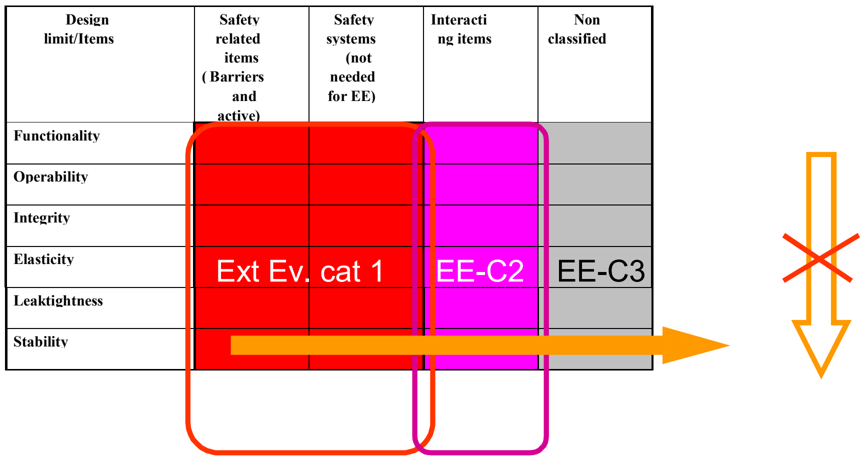

FIG. 2. Interaction between item classification and acceptability limits.

This approach is more in line with the availability of sophisticated calculation techniques, which can be applied to the design of NFOPs without high additional costs to the process.

In relation to the component design, an interaction could be stated between the facility classification and the item classification, as presented above. This interaction is discussed for example in Ref. [37] for the probabilistic methodology and it is practice in many Member States.

A possible model for such interaction in the design tasks relying on linear approaches, may follow the idea that the control of the safety margin is managed through reduction of the internal element forces only, in a sort of ‘single parameter control’. In this case a design class can be associated to a factor applied to the internal forces. Within each design class there is the potential for one or more acceptance criteria to be used. Table II shows a proposal for such interaction.

TABLE II. RELATIONSHIP BETWEEN FACILITY CLASSIFICATION AND ITEMS CLASSIFICATION IN CASE OF ‘SINGLE PARAMETER CONTROL’

DESIGN CLASS SELECTION				
Hazard Class / EE Class	High 1	Moderate 2	Low 3	Conventional 4
EE Class 1	Design Class 1	Design Class 2	Design Class 3	Design Class 4
EE Class 2	Design Class 2	Design Class 3	Design Class 3	Design Class 4
EE Class 3	Design Class 4	Design Class 4	Design Class 4	Design Class 4

3. GENERAL APPROACH TO THE DESIGN OF NFOP

3.1. GENERALITIES

The selection of the most suitable design methodology for the facilities selected in previous Chapters need to start from the identification of the common structural aspects. A selection of them is as follows:

- Most of NFOPs show a very distributed and irregular layout, mainly composed of flat buildings. Elevation can be high (about 20–30 m), but the lateral extension is almost always the dominant dimension.
- The most common construction technologies rely upon concrete frames, masonry, shear wall structures combined with steel frames for equipment
- Embedment is usually limited to 3–5 m, but in some cases the whole building is deeply embedded for more than 50% of their height
- Buildings are connected at many points by piping, conveyors, transfer tunnels, etc.
- Many equipment are connected in long chains: glove boxes, piping racks and bridges, etc.
- Many pools have liners
- Many cranes move heavy and hazardous loads.

These common aspects define also a sort of limit of applicability of the methodologies suggested in the following or, in many cases, they represent the critical areas where checks have to be carried out.

3.2. SAFETY MARGIN CONTROL IN THE DESIGN PROCESS

For comparison, the general approach to a safe design is shown in Figure 3 in the case of NPPs: the possible contributions to the overall safety margin in all the major phases of site evaluation, design, construction and operation are highlighted.

In the case of NFOPs, in order to meet the safety requirements discussed in Section 2 and to allow simplified and well consolidated design procedures, the requirements for the design procedures can be modified as in the following:

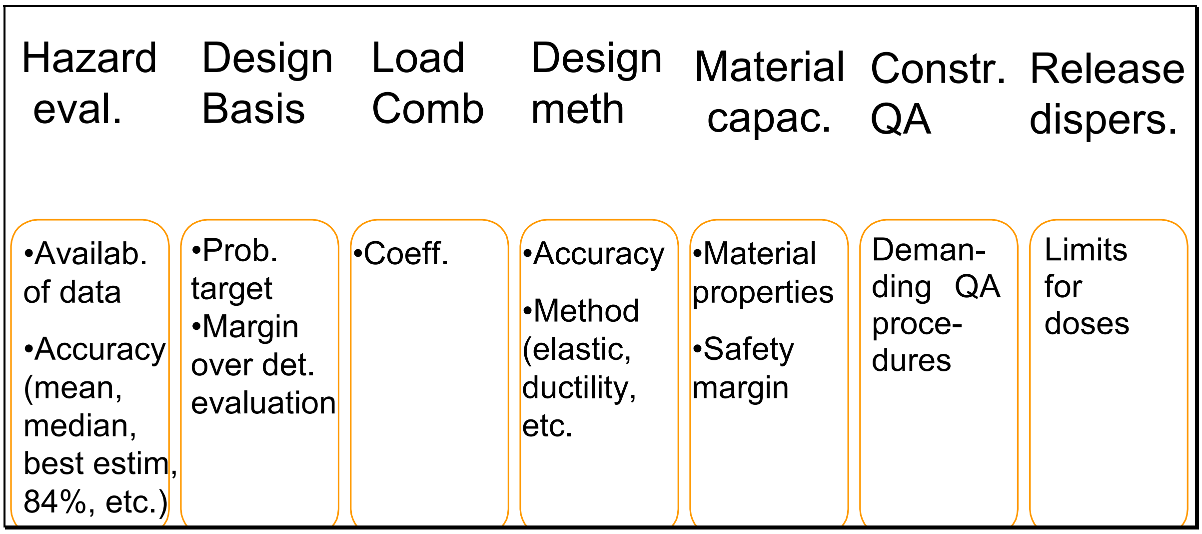

FIG. 3. Steps in the design process where safety margin might be located.

(1) A **grading** has to be defined between nuclear power plants safety approach and conventional buildings, according to their potential for radiological release and to the safety functions defined by the safety analysis. Site specific hazard evaluation procedures and site investigation campaigns, return period for design basis events, design class concept, as defined in Sections 2, 4 and 5, are affected by grading.

(2) The safety margin in the design needs to be **easily proven**, even in cases when different standards are applied. The 'single parameter' concept, as defined in Section 2, can be applied to this concern

(3) An adequate level of **conservatism** needs to be guaranteed to compensate for reduced site hazard analysis, site investigation campaigns and for simplified analysis methods, following the main objectives of this report. Larger areas for investigation campaigns, simplified, conservative, design basis and 'penalty' coefficients according to the applied analysis method can be applied to this concern

In this framework, conventional standards may be applied for structural design and qualification of components in NFOPs, with a big advantage for the management of design and qualification phases. The role of carrying the safety margin correspondent to the risk classification of facility and items is left either to the design basis loads, or to a reduction factor applied on the internal forces, both external to the 'black box' of the standard design.

In either case, the load reduction factor has to be selected according to the combination of facility and item classification, in a graded way, for example as suggested in Table I from 'no reduction' up to the building code and standards for conventional buildings.

It is noted that in general internal force reduction has the form of a factor, but in some cases it may also have a physical meaning, such as 'ductility' in the seismic design of structures. Ductility usually has a more general meaning, as its main part is related to the energy absorption in the specific structural configuration. However in this report structural detailing is not explicitly addressed, for simplicity, and therefore ductility is limited to that minimum amount that any structure can provide without specific provisions.

With this approach relying on conventional standards, other conservatism sources are implicitly included, such as: structural damping, soil structure interaction, structural detailing, etc.

It is common practice to apply different standards to the design of different items, mainly depending on the supplier. For example, building structures, mechanical components and electrical equipment, often follow national standards of different States.

In this case it is suggested that a detailed analysis is carried out on the compatibility of such standards to guarantee the proper management of the overall design safety margin, according to the general concepts described above.

Another conservative margin is usually applied to the hazard evaluation to compensate for simplified investigation campaigns or for simplified calculation methodologies. Such conservatism needs to be demonstrated by the designer and it never replaces the safety margin required by the classification discussed above, but is to be added on top of it.

The final combination of various effects is described in Fig. 4, where the mechanisms suggested to control safety margin and conservatism are listed for the most important siting and design tasks. It is noted that while facility and item classification are used to calibrate the safety margin, simplified design basis, reduced site investigation campaigns and simplified calculation methodologies may provide conservatism to the overall design.

In conclusion, in this design approach load combinations and material properties could be taken by standard design procedures and the whole siting/design process has only two major modifications compared to conventional building design:

- the hazard and the site properties are required to be site specific, even if evaluated with simplified and conservative procedures. The internal forces need to be corrected by a ‘pseudo-ductility’ coefficient (reduction factor to be applied to the internal forces) according to the interaction of facility and item classification, as shown in Table I
- the assessment of the acceptance criteria (and therefore the design limits associated to the material capacities) needs to be carried out explicitly in compliance with the safety objectives and documented in the SAR.

As a consequence, the site exclusion criteria may be revised (i.e.: in case of EE which require specific, complicated and expensive protection structures and systems, it might be convenient to exclude the site, when possible) to allow for simplified investigation campaigns and simplified design rules. Moreover, a requirement on the minimum level of site investigation needs to be defined and rules have to be defined to compensate for the limited investigation campaigns and to guarantee a certain conservatism. Section 4 deals with such aspects.

3.3. DESIGN PROCEDURES AND SAFETY ASSESSMENT

In case the above mentioned approach is followed for structural design, the safety assessment of the classified items is straightforward and implicit in the design.

However, there are cases in which the safety margin is not easy to be assessed and other approaches have to be put in place:

- in case of equipment and components, where the acceptability criterion is either ‘functionality’ or ‘operability’, the EE classification has to be reflected into requirements for direct qualification of the item by testing.
- in case of items in EEC3, it is difficult to assess equipment performance during an EE only by analysis. Therefore, a post-construction walkdown verification is suggested mainly focused on anchoring (mainly seismic), EE induced fire, flood, chemical release, drop loads etc., spatial interaction between items and building and items, impairing of operator access and action.

However, neither acceptance test nor walkdown can provide a measure of the safety margin (only fragility testing can do that) and therefore, in case such an evaluation is needed, special procedures can be followed, borrowed (and maybe simplified) from practice in NPPs, for example in Refs [22, 25].

Moreover, the application of single failure and common cause failure are intended to provide the design with a high level of reliability. A good balance among redundancy, diversity and quality requirements of items and systems needs to be guaranteed. A good reference for such analysis is provided in [34].

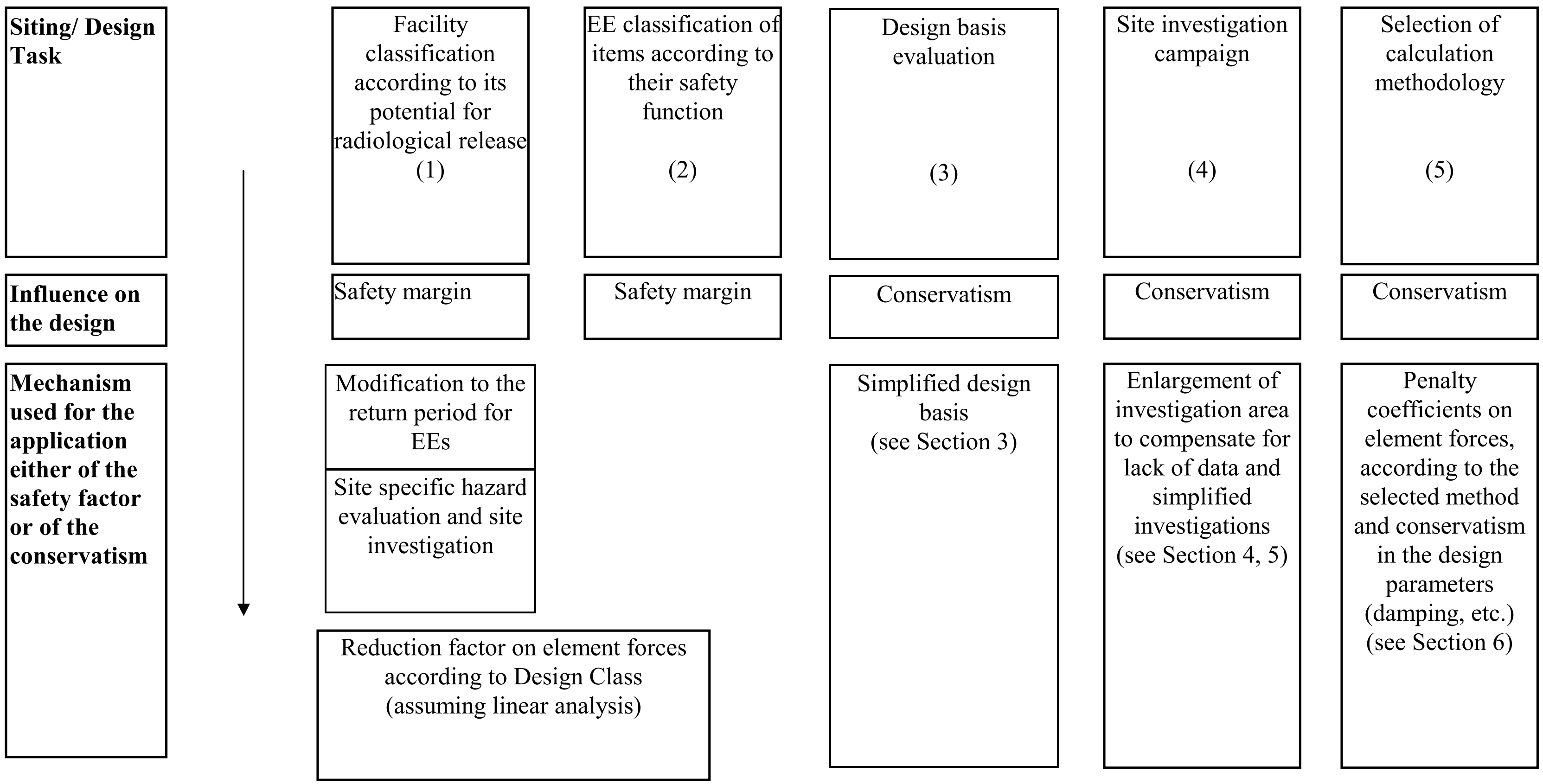

FIG. 4. Scheme of different sources of conservatism and safety factors.

3.4. AGEING EFFECTS ON DESIGN AND RE-EVALUATION

Traditionally, most of the NFOPs are not designed with reference to a 'design lifetime' like in the case of NPPs and a number of layout and plant modifications are normally expected during their operation. Therefore the ageing aspects need to receive special care, mainly through the following tasks:

- appropriate provisions during design: this needs to focus mainly on appropriate material selection and development of technical specifications for periodical inspections
- surveillance and testing to assess degradation of components and systems
- development of a preventive maintenance programme
- optimization of operating conditions
- management of repairing, replacing or refurbishing of components.

More details are available in [11], where strong emphasis is given also to the re-evaluation scenario.

These tasks need to be co-ordinated through an ageing management programme to be implemented during the plant lifetime.

In view of this, at the design stage appropriate material selection needs to be carried out and a detailed specification of the safety requirements to structures, systems and components need to be compiled as input for inspection, maintenance and ageing considerations.

In the re-evaluation of existing facilities the proposed approach is still valid in terms of facility and item classification. In case not enough safety margin is shown by the design assessment, the following two actions are envisaged, in a priority sequence:

(1) more refined procedures need to be applied eliminating excessive conservatism in the various areas defined above
(2) an 'easy fixes' programme needs to be implemented with the goal of a minimization of the investment with a maximization of the safety margin increase. Procedures for such assessment can be taken from the experience in the NPPs, such as [38].

According to general experience in Member States, these two actions provide a successful re-evaluation of most of the existing facilities.

3.5. THE SPECIAL CASE OF RESEARCH REACTORS

Research reactors show a wide variety of technological solutions, radioactive inventory, power level and siting environment. A limited selection of them was analysed at IAEA to support their EE classification and therefore to derive suitable design procedures for such type of structures.

Table III collects some data from worldwide installations, taken directly from the safety analysis reports.

Few comments to the Table are summarized in the following:

(1) There is a very broad range in nominal power, which is expected to be associated to a broad range of radiological inventory
(2) There are very big differences in layout, building design and safety system design even with the same reactor type. This fact shows the difference policy in the safety approach and design.
(3) There is a big difference in the design basis and in the standards applied for design: in some cases, even if sited in similar environment, the design basis does not include minimum requirements for external events and the design is carried out according to national standards for conventional industrial installations.

Therefore, according to the general principles set up in the previous section, and to the general background publications, in the case of research reactors the EEC1 and EEC2 systems may be associated to three different groups of items (in increasing order of importance) according to the installed power in the reactor, radioactive inventory and core design features, as described in the following:

(1) Only the reactor protection and reactor shutdown systems are required to be always operable.
Based on safety analysis, it needs to be shown that for all credible accidents, the radioactive inventory cannot be released from the principal barrier. This is the case of a research reactor of low power, with power level less than one megawatt, where therefore only the reactor shutdown system needs to be designed to withstand the EEs (they are often identified as Class C reactors: low power, low inventory and extremely low probability of breaking the fuel clad because of the reactor operation and utilization).
In order to ensure coolability of the core it may be required to ensure no disruption of the core, i.e., that reactor internals and other equipment or structures such as the crane bridge or the building roof need not to fail in a way that would obstruct the cooling of the core.
(2) The reactor protection and shutdown systems plus a system to remove decay heat from the core (shutdown core cooling system, whose function is executed in most of the research reactors by the emergency core cooling system) are required to be always operable.
This is the case of a research reactor of medium power, with power level less than two megawatts, where in addition to the reactor shutdown system, cooling of the reactor needs to be incorporated into the design and the capability of the cooling method to withstand the EEs needs to be demonstrated (they are often identified as Class B reactors: medium power, intermediate inventory and low probability of breaking the fuel clad during operation by loss of cooling or coolant).
(3) The reactor protection and shutdown system, a system to remove decay heat from the core plus a confinement system (building plus normal or emergency ventilation) to control the release of radioactive material to the environment are required to be always operable.
This is the case of a research reactor of high or medium power, where a confinement system (e.g.: containment building plus emergency ventilation) needs to be utilized to meet the requirements of previous Section (they are often identified as Class A reactors high power, high inventory and low probability of breaking the fuel clad during operation because of engineered safety features incorporated during the reactor design).

TABLE III. SITING AND DESIGN DATA FOR SOME RESEARCH REACTORS IN THE WORLD (SOURCE: SARS)

Location	type	power (MW)	typology	moderator	DBE (g)	Design conf.	Confinement	siting features
Ghana, Pakistan, China (4 reactors)	GHARR1	0.03	pool		0.2		Confinement	
USA	UCLA	0.1	pool		0.5			
Canada	SLOWPOKE	0.5	pool + tank		0			
Romania	TRIGA	0.5	pool		0.4			(VIII+0.5 MSK)
Mexico	TRIGA II	1	pool		0.3		confinement	
Slovenia	TRIGA II	1	pool		0.3			(IX MCS)
Indonesia	TRIGA	1	pool		0.6	Reactor containment building		
Malaysia	TRIGA II	1	pool		0.6		confinement	ACC, fire, explosion, earthquake, flood, storm
Norway	JEEP II	2	tank	HW	0	diameter 22 m, H=22 m	Containment (P test 0.15 kg/cm^2)	
Morocco	TRIGA III	3	pool		0.35		Confinement	
Greece	GRR-1	5	pool		0.18	Prestressed pool	Confinement	
Pakistan	PINSTECH	5	pool		0.2	0.1 g for aux bldg.	containment	fire, lightning, storms, earthquake
Chile	CNCR	5	pool		0.3	Liner 6 mm	Confinement	(SMA 0.6)
Australia, Denmark, Germany (6 reactors)	HIFAR	10	tank	HW	0.2	Steel structure on a concrete foundation, diameter 21 m, H=21 m	Confinement (P test 10 kPa)	
Hungary	VVR-SZM	20	pool		0.1	Reactor containment building	Confinement	
Netherlands (JRC)	HFR	45	vessel + pool		0.1		Containment	

(4) In addition to those requirements of the so-called ‘Class B’, it is necessary to incorporate an emergency ventilation system to mitigate the consequences if the radioactive fission products escape the principal barrier in spite of the requirements imposed by Class B. Based on safety analysis it needs to be shown that a confinement system is necessary to mitigate the consequences of some design basis accident. The operability of this confinement system therefore needs to be guaranteed.
For this class, it is much more difficult to assign a range of power levels. Clearly, however those reactors that are near the limit of applicability of this publication are in this class, usually above some megawattage (2–5 MW).

Table IV shows an example of the relationship between classification of research reactors and their structures, systems and components with regard to their design to EEs.

TABLE IV. CLASSIFICATION OF RESEARCH REACTOR FACILITIES, OF STRUCTURES, SYSTEMS AND COMPONENTS

CLASSIFICATION OF RESEARCH REACTORS FACILITIES			
FACILITY CLASS	A	B	C
RADIOACTIVE INVENTORY	HIGH	INTER-MEDIATE	LOW
POWER LEVEL[1]	HIGH	MEDIUM	LOW
CATEGORIZATION OF STRUCTURES, SYSTEMS AND COMPONENTS[2]			
SHUTDOWN AND PROTECTION SYSTEMS	EEC1	EEC1	EEC1
SHUTDOWN CORE COOLING SYSTEM	EEC1	EEC1	EEC3
CONFINEMENT SYSTEM (BUILDING AND EMERGENCY VENTILATION)	EEC1	EEC2–EEC3	EEC3
EMERGENCY POWER SUPPLY	EEC1	EEC2–EEC3	EEC3
WATER PURIFICATION SYSTEM	EEC3	EEC3	EEC3

[1] For the purpose of the present report, it is assumed that research reactors are grouped into three groups attending their power level: low, medium and high power research reactors. The first group (low power research reactors) includes subcritical and critical assemblies and research reactor up to 50 kW of nominal power and represents about 40 % of the currently operating research reactors; the second group (medium power research reactors) may include research reactors with powers higher than 50 kW and lower than 1.5 MW and represents about 20 % of the currently operating reactors; finally, research reactors with power higher than 1.5 MW may be included in the third group (high power research reactors).

[2] the assigned categories are related to the design and construction requirements, particularly referred to civil and mechanical structures, in accordance with the design level (or free-field acceleration) assigned to the DBE (see Table VI).

More in detail, in the case of the reactor building, according to [5], the external event scenarios require in general the following functions:

- Automatic shutdown signal in case a certain threshold is reached
- Shutdown functions and reactor protection systems
- Residual heat removal system
- Operator access and control (for independent manual scram and reactor monitoring)
- Radiation monitoring during and after the design basis event
- Long term monitoring of basic reactor parameters
- Functionality of all other systems required for defence in depth according to the specific acceptance criteria assigned to them (e.g. functionality of the emergency ventilation system, ECCS, etc.).

However, according to engineering practice, the functionality of the items not needed to bring the system into a safe shutdown might be excluded in many cases from the safety related list as a consequence of the safety analysis of the reactor itself; this approach is accepted in case it could be demonstrated that the radiological consequences of the postulated accident which they are designed for are below the regulatory limits.

Therefore, according to such approach, the emergency ventilation system, the ECCS and other safety systems might not be required in case of an accident for a reactor of limited power (usually 2–3 MW) and therefore, in case they are installed for additional protection, they might not be EE classified. These conclusions however have to be explicitly demonstrated in the SAR with an appropriate calculation of the radiation dose which would affect the site in case of a postulated accident.

As a consequence, in case of a design basis external event (DBEE), only the first part of the functions listed above could be required for small research reactors, asking for a classification of the relevant buildings according to the following design requirements:

- full service ability of control room to enable operator actions;
- full and safe access to the reactor building during and after a DBEE (including lighting, availability of stairs and necessary housekeeping);
- functionality of the safety systems mentioned above, with the redundancy requested by the 'design for reliability principle' [21];
- availability of power supply to the safety systems during and after a DBEE; and
- prevention of any interaction accident during and after a DBEE from loss of parts over safety related components, internal flood, fire and/or hazardous substances which could impair the required safety functions by components and operator.

These rules provide the basis for classification, and therefore for design, of RR structures and components.

4. DESIGN BASIS FOR EXTERNAL EVENTS

4.1. INTRODUCTION

One of the main targets of this publication is the evaluation by simplified methodologies of the design basis for the external events to be considered in the design of the facilities, trying to limit the high costs associated to the investigation campaigns preliminary to the traditional NPP design.

The evaluation of the site hazard for external events can in general follow the IAEA recommendations for siting and design of NPPs and research reactors [5, 16]. However, the site selection process may consider more restrictive exclusion criteria than for NPPs and research reactors, as a compromise with the investment requested for the facility design, construction and operation. In this sense, some events which are difficult or expensive to protect the facility against could be screened out, like aircraft crash (thick shielding and special equipment qualification would be required for facilities often without a containment), explosions (blast resisting structures are required), flooding (site protection engineering structures need to be built and maintained), etc. In fact in NFOPs usually the internal accident scenarios do not imply high demand to the structures, like in the case of NPPs where, for example, a containment is normally part of the design. Therefore a protection to external events would add heavy requirements to the design which might be incompatible with a rational approach.

In other cases an exclusion of some events from the design basis can be done according to specific engineering provisions implemented in NFOPs. An example is the consideration in the safety analysis of the typical spread building layout which might reduce the consequences of an aircraft crash through physical separation of safety related items.

When NFOPs are at the same site of a NPP, they share the same hazard, but the design basis might be different due to the considerations above.

Once a screening process on the events to be considered in the design is carried out, a detailed site hazard has to be evaluated. Facilities in Class 1, 2 or 3 always require site specific hazard evaluation, which could be simplified for the lowest classes, starting from the requirements in the IAEA 'S' series for Class 1.

Once the hazard has been evaluated, a grading on the return period is suggested for facilities in Classes 1, 2, 3. There is no agreement in Member States on this subject, but a reasonable proposal is shown in Table V, taken from [39], for the major hazard sources. Other useful references could be found in Refs [35, 36] where a broader review of probabilistic risk evaluation is described, with reference to the comparison with other sources of hazard for public and environment.

TABLE V. GRADING OF MEAN ANNUAL PROBABILITY OF EXCEEDANCE FOR DIFFERENT CLASSES OF FACILITIES (ACCORDING TO [39])

Event	Class 1(1)(2)	Class 2	Class 3	Class 4
Earthquake	1E-4	5E-4	1E-3	2E-3
Wind	1E-4	1E-3	2E-2	2E-2
Tornado	2E-6	2E-5		
Flood	1E-5	1E-4	5E-4	2E-3
Aircraft crash	1E-5	1E-4		

Note: (1) definition of classes is slightly different in [39] than in Chapter 2
(2) NPPs usually have an hazard level with an associated exceedance probability value smaller than Class 1.

The difference in the hazard sources is consistent with the criteria provided in Chapter 2 based on the evaluation of the impact of the different scenarios on the plant safety. Site investigation needs to cover in general geology, seismology, geotechnical engineering, hydrology, meteorology, marine environment, human development plans, industrial installations, communications, naval, train, road and air traffic, etc. However, when data are not available for the specific site (evaluations procedures on data reliability and requirements for minimum monitoring are available in the respective safety standards), the extension of the investigation campaigns might again be compensated by some conservatism in the definition of the design basis for facilities in Classes 2 and 3, as suggested in the following for different hazard types.

In any case it is suggested to carry out an estimation of the return period associated with the selected design basis, at least to allow a comparison with national standards for design of industrial facilities, and therefore to assess the compliance with the requirements of the facility classification.

4.2. DESIGN BASIS EARTHQUAKE

In this section some simplified, conservative rules for design basis evaluation are described for facilities in Class 2 or lower, in case few data are available at the site. Class 1 requires the application of [17] with reference to the exceedance probability defined in Table V.

Based on availability of instrumental or historical data, one of the following methods may be used to define design basis ground motions. In any case, the minimum design level acceleration needs to be 0.1 g.

4.2.1. Site specific design response spectra based on instrumental data

In case some instrumental data, such that the location of seismogenic sources and zones, attenuation relationships and possibly hazard maps within the region, are available for the region, then a site specific design response spectra (including site effects) can be generated by either using the envelope of response spectra (for 5 % damping) calculated from recorded data or using hazards maps that are constructed using such data.

The Safety Guide for NPP siting [17] needs to be used in this case, particularly in the evaluation of sufficiency and reliability of available data, with reference to the exceedance probability in Table V. Appropriate simplifications approved by the Regulatory Bodies might lead to a reduction of the embedded safety margin according to the facility classification.

4.2.2. Design response spectra based on historical seismicity

In the event that instrumental data are not available, the design basis ground motion can be evaluated on the basis of the maximum historical intensity in the region. For this evaluation, the following procedure might be applied for facilities in Class 2 or lower, provided the site region shows a reasonable uniformity from the seismotectonic point of view:

- A zone having a radius of minimum 100 kilometres from the site needs to be considered. Larger radius of up to 200 km needs to be considered when data is lacking and there is low seismicity.
- Using available publications and catalogues the maximum observed intensity in this area needs to be established and assumed at the site. The information needs to cover as much historical data as possible. In any case it needs to be extended to at least 100 years [17].

To each seismic design level index, a minimum value of design free-field acceleration for firm bearing strata (A_b) needs to be assigned as shown in Table VI. These assigned values need to be compatible with the seismic provisions of several national building design codes currently adopted by several States. However, intermediate design accelerations can be assigned based on detailed analyses of the data and other considerations.

The conservatism of such evaluation is embedded in the assumption of a large area of investigation, much larger than in the case of NPP siting [17]: the reduced investigation campaign is then compensated by an enlargement of the area of interest and therefore in the use of the highest seismicity values of the whole area for the site.

TABLE VI. ASSIGNED MINIMUM FREE-FIELD DESIGN ACCELERATIONS

Range of Maximum Historical Intensity, I_{max} [*1]	Seismic Design Intensity Level Index	Assigned Design Acceleration for Firm Bearing Strata A_b (g)
I_{max} < VIII	1	0.1
VIII ≤ I_{max} < IX	2	0.2
IX ≤ I_{max}	3	0.4 [*2]

[*1] Modified Mercalli Intensity Scale
[*2] For Seismic Design Level Index 3, intermediate design accelerations can be assigned between 0.2~0.4 based on detailed analyses of the data and other considerations

4.2.3. Design response spectral shapes

In the absence of site specific design response spectra constructed from instrumental or historical data, Figures 5 and 6 can be used as the normalized design response spectra representing different design intensity levels and soil types (discussed in the following chapter), to be used in conjunction with the pga values of Table VI. The numeric values are given in Table VII, from [54].

TABLE VII. SEISMIC DESIGN INTENSITY LEVELS

Design intensity level	**A**	**B**	**C**	**D**
1 & 2 (Soil 1,2,3)	0.05/1 0.05/1 0.05/1	0.1/3 0.2/2.5 0.5/2.3	0.2/3 0.6/2.5 1.1/2.3	2/0.3 3/0.55 4/0.81
3 (Soil 1,2,3)	0.05/1 0.05/1 0.05/1	0.1/3 0.24/2.5 0.5/2.3	0.4/3 0.9/2.5 1.6/2.3	2.1/0.4 3.5/0.7 4/0.8

These spectral shapes are given for a 5% damping ratio (ζ) and are valid as horizontal components. In order to get the spectral shape for other damping ratio zeta, the value of acceleration at points B and C have to be multiplied by the Dd coefficient defined as: $Dd=1.5/(1.0+10*\zeta)$ for $2\% < \zeta < 20\%$. In this formula, ζ needs to be set in unit, not in per cent.

For other period T, the acceleration values have to be linearly interpolated within the bilogarithmic domain.

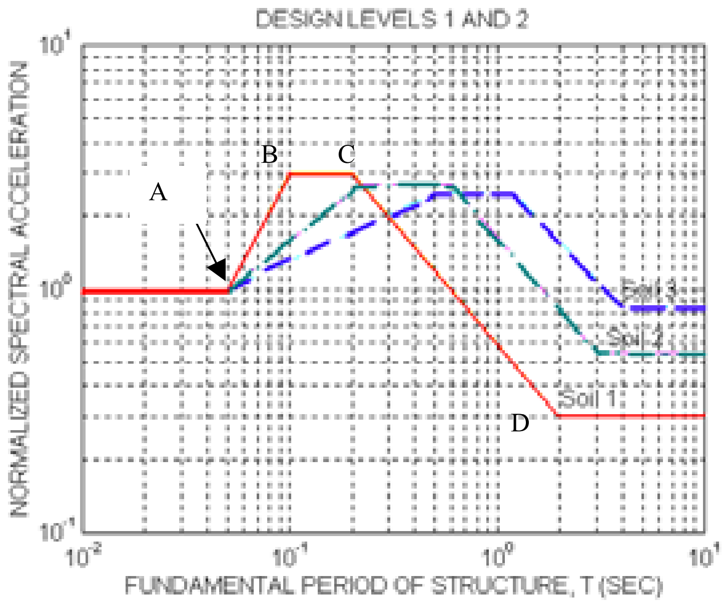

FIG. 5. Normalized design response spectra for seismic design intensity level. Indexes 1 and 2.

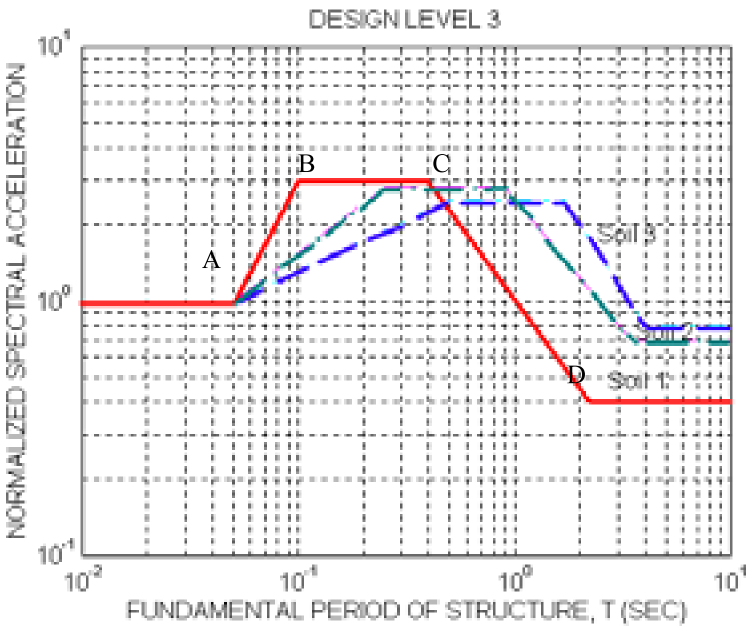

FIG. 6. Normalized design response spectra for design intensity level 3.

a(T)=a(Ti)*(a(Ti+1)/a(Ti)^(Log(T/Ti)/Log(Ti+1/Ti))

Where Ti and Ti+1 are the periods points A to D closest to T. Ti=< T=<Ti+1

For periods less than T_A, a(T)=1 for any period and damping ratio.

For periods more than T_D, a(T)=a(T_D)*(T_D/T)^2 (constant spectral displacement in this period range).

Both horizontal components of earthquake need to be taken equal to the above defined spectrum.-

The vertical component of accelerations needs to be at least 50 % of the horizontal component.

The earthquake is supposed to have an isotropic probability of occurrence with respect to the 3 space directions. This means that any combination of the 3 maximum components of the form alphaX*SX+ alphaY*SY+ alphaZ*SZ with alphaX^2+alphaY^2+alphaZ^2=1 is equally probable.

In light of recent observed data that exhibits larger than expected peak accelerations and long duration pulses, if and when a particular site, as identified by appropriate seismological and geological investigations, is (a) within 10 km of a fault and (b) capable of generating magnitude 7–7.5 earthquakes, then steps need to be taken to appropriately increase the design acceleration level for firm bearing strata, A_b. [40–42].

Except for Type 1 soil sites, a two step process needs to be carried out on the seismic input for the evaluation of the input at the foundation level: a convolution from the bedrock to the free field and a deconvolution from the free field to the foundation level. Simple models may be used based on mono-dimensional wave propagation and equivalent linear methods. However, large modifications of the ground motion at the bedrock needs to be carefully justified by parametric studies. The use of the free field motion at the foundation level is a conservative practice and is usually acceptable.

4.3. DESIGN BASIS FOR AIRCRAFT CRASH

4.3.1. General

The aircraft crash scenario (ACC) at a site of a NFOP needs to be generally excluded through probabilistic considerations and suitable siting procedures. However there are cases when such a scenario is included in the design, namely:

- when a NFOP is located at a site where the ACC cannot be ruled out, either for strategic reasons or because it has to share the site with other facilities;
- when some generic degree of protection is requested to the facility against external 'missiles' associated to strong wind or even to sabotage.

The effect of an aircraft crash on a NFOP needs to be analysed in order to identify the areas to be protected.

Evaluation of the effects of an aircraft crash needs in general to consider:

- global bending and shear effects on the affected structures ('overall missile effects');
- induced vibrations on structural members and safety related equipment ('global effects'), particularly when safety related items are located close to the external perimeter of the structures;
- localized effects including penetration, perforation, scabbing and spalling, by primary and secondary missiles ('local effects');
- the effects of fuel fires and possibly explosion on structural members as well as exposed safety related equipment (ventilation system, containment openings, air baffles).

In general, NFOPs do not show a distributed resistance to a crash, being built with steel and concrete frame structures; in fact only continuous concrete walls at the external boundary of the building can provide some degree of protection.

Therefore the analysis needs to consider that the location of the impact is potentially anywhere on the building (peripheral walls and roof) with any direction inwards into the building. In principle all exposed structural elements need to be checked against all mechanisms discussed above. Moreover, the definition of the impacting object is usually very difficult and needs to consider a wide variety of aeroplanes, helicopters, missiles, etc.

However, such an analysis of the failure mechanisms is very complex and expensive and it requires very high validation efforts. Therefore a simplified approach is described in the following for a quick assessment of the consequences of ACC hazard on NFOPs.

4.3.2. Basic data for screening

Basic data for aircraft crash protection implies an extensive knowledge of air traffic in the vicinity of the facility. The air traffic is conventionally grouped into three categories:

- commercial aircrafts more than 5.7 t weight
- general aircrafts less than 5.7 t weight (including helicopters)
- military aircrafts

As effects from impacts of military aircraft are very severe, it is suggested to avoid installing an NFOP near a military airfield. Basic studies need to be made in order to be able to collect the following parameters:

- presence of an airfield in the vicinity of the facility site.
- probability of crashes per flight from statistical data in the whole region surrounding the site
- number of flight per year
- mass and impact characteristics for the different possible aircraft. Impact characteristics is typically of 'rigid body' or 'hard shock' typically for military jets or 'soft shock' for civil aviation aircraft.
- speed of the aircraft when crashing.

From all these parameters, for each category, a probability of aircraft impact per unit surface and per year can be derived. From its geometry, a virtual area of the facility can be defined as the mean normal section of cylindrical projection of the facility under the different crash angles. Finally, the probability of an aircraft crash on the facility can be evaluated as the product of the probability of impact per unit surface and per year by the virtual surface of the facility. The need for an aircraft crash protection depends on the probability of crash for each category. If this probability is higher than 10^{-5} per year, the facility design needs to include the corresponding aircraft crash assessment.

4.4. DESIGN BASIS FOR WIND

4.4.1. General

Effect of wind on constructions are generally grouped into two different types:

- Effect of extreme values of wind pressure that can induce high forces in structures with reference to their ultimate capacity.
- Effect of moderate but quasi constant values that can induce resonance effects through the mechanism of the Karman turbulence and then lower values for capacity of the structure due to fatigue. This kind of effect depends on the shape of the structure and generally appears only on tall and slender structures such as chimneys, perpendicular to the wind direction.

It is important that site statistics of wind speed and direction are well known even in the low speed range.

The effect of wind on structures mainly depends on the general layout of the building. It can include over pressures on one side, and under pressure on the lateral, opposite side and on the roof side. Such a distribution depends also on wind direction.

The structural analysis in relation to extreme wind needs to be carried out for all structural elements with respect to all directions of wind and combination of possible over and under pressure compatible with the building geometry.

The environment of a structure has also deep influence on wind effects. Mask effect may result in reduced loads, group effects in increased and eventually resonant loads. Group effects need to be accounted for while mask effect is usually neglected.

In general, only frame or slender structures are really sensitive to wind effects. Tall chimneys either on buildings or on ground are sensitive to wind effects, even more than to earthquake effects. In order to appreciate the possibility of resonance effects, the dynamic behaviour of the chimney as well as the behaviour of aerodynamic flow of wind around it has to be assessed in terms of Karman turbulence frequency.

4.4.2. Basic data

Typically, extreme value, normal (rather frequent) value and frequent value of wind speed have to be known from a site region monitoring (at least 50 year data). If such a database is available, the extreme value of wind speed for structure evaluation needs to be the mean value plus 2.0 standard deviation. The normal (rather frequent) value needs to be the mean value plus 1.0 standard deviation. For the study of frequent wind, the whole distribution of wind speed vs. time needs to be accounted for, but mainly the speed(s) which induces resonance effects. The measure of wind speed needs to be carried out at 10 m height above ground.

In any case, the extreme value cannot be lower than the value provided by the national building code for the same region.

Any national code giving over and under pressure distribution including variation with the height of the considered point above ground and relative values with respect to the building geometry, the wind speed and wind direction can be applied for that purpose, provided site effects are evaluated.

In case site effects are expected to be significant, a monitoring system has to be installed and operated for a significant time period for comparison with regional data.

Concerning rotational winds, statistics in the region need to be recovered. In lack of that, an evaluation of global meteorological data has to be carried out to check if rotational winds are likely. In case they are, standard procedures as for nuclear power plants needs to be followed for an identification of a possible scenario.

4.5. DESIGN BASIS FOR SNOW

4.5.1. General

The effect of snow on construction is generally reduced to the weight of the accumulation of snow on upper horizontal or sub-horizontal surfaces of structures. Generally few structures are significantly sensitive to such loads. This is however the case for light frame construction which are rather common among NFOPs.

Evaluation of this load is similar to that of extreme wind loads, but it is less complex as far as no dynamic effect is expected.

4.5.2. Basic data

Typically extreme and normal (rather frequent) values of snow weight per unit surface have to be known either from a building code and/or from an investigation in the site region. If such a database is available (at least 100 years data), extreme value needs to be taken equal to the mean value of non-zero values of the database plus 2.0 standard deviation, and the normal value needs to be taken equal to the mean value of non-zero values of the database plus 1.0 standard deviation. If a building code is used for design purpose, a site monitoring system operating during the lifetime of facility needs to be installed at the site, data periodically compared with the data used for design and emergency procedures set up for the prompt snow removal in case of extreme precipitation.

Building codes may be useful to specify the non-uniform distribution of loads on the different surfaces according to their geometrical shape, including immediate neighbouring constructions if any.

4.6. DESIGN BASIS FOR EXTERNAL TEMPERATURE

4.6.1. General

In general, any temperature variation of a structure (steel or concrete) from its construction temperature induces forces and displacements, as for construction materials thermal dilatation coefficient is not zero. A mean temperature of construction of the structure has to be identified, as real thermal effects have to be evaluated by difference to the construction temperature state.

The temperature conditions to which a facility is subjected are due to:

- temperature of the air inside the facility
- temperature of the air outside the facility
- temperature of the structure materials when directly exposed to sunlight, for parts of the structure that are outside of the building.

For obvious operational reasons, generally the temperature of the air inside the facility is maintained by the ventilation/isolation system within such limits allowing the process and the workers to operate in correct conditions. These limits and the external temperature statistics provide the basis for sizing of the ventilation/isolation system and for the evaluation of the design basis temperature for all plant parts that are inside the isolation system.

Other parts of the structure are subjected to the influence of the air temperature outside the facility and to sunlight.

A different temperature may occur between the two faces of the same structure element, when this element is in fact a part of isolation system or partitioning of the facility into different temperature zones.

The structure temperature cannot be taken equal to the air temperature at the contact because of the heat flow between the outside and the inside the facility, the presence of the ventilation system and the thermal characteristics of the isolation system, structure, etc.

In the case of non prestressed concrete structures, cracking deeply affects the structure thermal response. Only approximate approaches are in use by the engineering community for the evaluation of such effects.

4.6.2. Basic data

Basic data for external temperature need to consist in the measure of two parameters:

- air temperature outside under shelter
- incident heat flow due to sunlight

This second parameter may have significant influence only if the structural elements are directly exposed to it because in this case, although maximum heat flow cannot last more than half a day, the corresponding temperature occurs with a very short time constant and therefore it can reach the maximum effects as if maximum heat flow is applied constantly.

Conversely, where all the structure members are located inside an isolated and/or ventilated zone, maximum effects need to be derived from the one day period of the heat flow.

These basic data need to be made over a duration of 1 year at least, and statistically processed in order to derive design data. Extreme values need to be taken as the maximum and minimum mean value over half a day in the whole year. Because the real structure temperature is lower and delayed with respect to instant external temperature data, no conservatism is to be accounted for via standard deviation. Conservatism can be put in the evaluation of the structure temperature itself by assuming extreme values as permanent values or at least over a one day period. The external temperature data need to be monitored after construction of the facility in order to check periodically the validity of design data.

4.7. DESIGN BASIS FOR FLOOD

4.7.1. General

According to the site location, external flood can be caused by different phenomena:

- Exceptional sea levels due to waves, tsunamis, storm surge, eventually in conjunction with tide.
- Exceptional river levels due to rain, snow melt and dam failure.

Because of its special consequences on nuclear criticality, presence of water in nuclear facility has to be controlled with a high reliability level.

4.7.2. Basic data

Basic data for water level need to be derived from a study of site hydrogeological environment, including all above mentioned phenomena, using either a probabilistic or a deterministic approach. Typically, highest levels of water due to waves, wind, rain and snow need to be analysed through a probabilistic approach while tsunamis, tide and dam failure through a deterministic approach from the knowledge of their cause.

For frequent events, a simplified approach used in some Member States assumes the historical maximum level (>50 years of measurements) multiplied by a factor of 1.5.

Changes foreseen in the site environment have to be considered if they increase the mean and maximum water level (new dams, land use, etc.).

4.8. DESIGN BASIS FOR EXTERNAL EXPLOSION

4.8.1. General

External explosion hazard may be related to the presence of other plants in the vicinity of the facility using or storing explosive materials such as gas, oil, chemical products or transport of such materials. There are three main types of explosion sources:

- clouds of explosive gas
- tanks full of gas
- solid explosive materials.

A complete study of surrounding industrial activity and transportation by road, river, sea, train or pipe lines has to be made in order to identify chemical nature and quantity of the transported substances, geographical location, frequency of occurrence of relevant accidents, storage or transport conditions and eventually protection against explosions.

As far as the following approach presents significant safety margins, focalization effects may be ignored.

4.8.2. Basic data

According to the above mentioned identification of potential explosion sources, the design can follow the approach of an equivalent explosion of TNT, particularly if the source is relatively far from the facility. For this purpose, two coefficients can be applied to the identified mass of explosive material:

- An equivalent TNT mass ratio is applied to the mass of explosive product and gives the equivalent TNT mass as for its explosive effects. For hydrocarbons, this coefficient is taken equal to 5. For other special products, this coefficient can reach higher values up to around 160. Specialized literature needs to be investigated in that matter, like [43].
- A coefficient for gaseous conditions defining the ratio of the total mass present in the storage or transport that participate in the explosion, depending on storage or transport conditions. If no detailed estimation is made, for hydrocarbon this ratio can be taken equal to 20%.

  Then, with respect to this estimated TNT equivalent mass and distance from the facility, and according to specialized literature, an overpressure triangular wave can be postulated including instant overpressure value and duration. When applying the pressure wave to the building, it is important to take into account reflection effects on walls depending on the relative direction of walls and pressure wave propagation (this coefficient can reach a value of 2) and dynamic effects due to the time variability of the pressure (this coefficient can also reach a value of 2).

5. GEOLOGICAL AND GEOTECHNICAL INVESTIGATIONS

5.1. GENERALITIES

As in the previous chapters, this section provides suggestions on limited site investigation campaigns, according to the general strategy defined in Sections 2 and 3.

Geological and geotechnical investigations at the site need to be performed with the following objectives:

(1) the assessment of possible geological and/or geotechnical problems involving surface rupture due to faulting, liquefaction, collapse and slope instability as a consequence of an earthquake.

(2) the evaluation of the soil characteristics so that reasonable soil characterization can be achieved in the modelling of soil structure interaction, mainly in seismic analysis. For this, it is necessary to determine depth to bedrock, shear wave velocity variation and average shear wave velocity for a depth of 25 m, at least.

(3) the evaluation of geotechnical parameters to be used in the design of the foundation for static and dynamic loads and for radiological dispersion studies through groundwater.

For facilities in Class 1, the basic reference publication [24] needs to be applied.

For facilities in Class 2 or lower, the simplified following approach might be applied.

The amount of geotechnical investigations to be performed needs to be based on the extent of potential problems, the available data, the physical size of the facility and its classification. It is suggested that the soil profile is physically identified (i.e. through drilling) to a depth equal to at least one half of the maximum foundation dimension but not less than 25 m. The depth to firm bearing strata also needs to be determined using geological inference, boreholes or geophysical methods.

The geotechnical investigations need to include determination of the bearing capacity, shear wave velocity representative of the site and other soil parameters for foundation and building design. These need to primarily involve borehole drillings in sufficient number and to sufficient depth, depending on the soil conditions, but at least one below every safety related building.

For competent rock sites where the rock formation continues to a sufficient depth, drilling may not be necessary.

Uncertainties in the mechanical properties of the site materials need to be taken into account through parametric studies, at least on the shear modulus value. One method is to vary the shear modulus between the best estimate value times 1.5 and the best estimate value divided by 1.5.

Table VIII provides a simplified characterization of soil types based on shear velocity (average value of the upper layers), for use in the selection of the seismic input and in soil structure interaction.

TABLE VIII. SOIL CHARACTERIZATION

Characterization	SOIL TYPE		
	1	2	3
DESCRIPTION	Firm Bearing Strata (thickness > 25 m)	Other soil than those defined as 1 & 3, or with lower thickness	Alluvium ground which is thicker than 25m
Shear Wave Velocity V_s (m/sec)	>1100 m/s	300–1100 m/s	150–300 m/s

For a more comprehensive treatment of the subject, the reader is referred to [17, 22, 24].

In parallel with the foundation investigations and the use of available geological and geotechnical data, studies need to be performed at the site to assess possible hazards resulting in permanent soil deformations (including surface rupture, liquefaction, collapse, slope instability). In the event that there are special features of the site such as unusual surface and subsurface topography, then investigations need to be carried out to determine if there is focusing of incoming seismic waves.

The walls of high risk facilities need to not be in direct contact with the soil mass because the evaluation of dynamic soil pressures to embedded structures can be very complex and may require a substantial effort. Unless it can be shown that safety is not compromised by having the EEC1 structures in contact with soil, their walls need to be separated from the soil by EEC2 structures. If this is not practical, then dynamic soil data, which are outside the scope of this report, are required, in particular for EEC1 buried piping. For applicable methods the reader is referred to [24].

5.2. LIQUEFACTION POTENTIAL

Potential consequences of any liquefaction and soil strength loss need to be assessed, including estimation of differential settlement, lateral movement or reduction in foundation soil bearing capacity. Mitigation measures need to be considered in the design of the structure and can include, but are not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacement or any combination of these measures.

The potential for liquefaction and soil capacity loss needs to be evaluated for site peak ground accelerations, magnitudes, and source characteristics consistent with the design earthquake ground motions. The value of peak ground acceleration to be used for liquefaction studies needs to be evaluated on the base of a site specific study. In the absence of such a study, peak ground acceleration may be assumed equal to $S_{DS}/2.5$ where S_{DS} is the spectral peak acceleration estimated at the site.

Two procedures for evaluation of liquefaction hazard are included in Appendix I. In addition to these, the procedures recommended in Ref. [41] can also be used. The assessment of liquefaction potential by any one of the methods needs to yield similar conclusions.

6. BUILDING DESIGN

6.1. SEISMIC DESIGN

6.1.1. Layout considerations

In order to resist earthquakes, the general shape of buildings needs to be as simple and regular as possible, as well in the horizontal plane as in elevation. This simplicity involves the volumes, the regularity and symmetry of mass and stiffness distribution, and the distribution and continuity of any elements called to provide seismic capacity to the structure.

Centroid of masses of floors and centroid of stiffness at each level need to be as close as possible.

With respect to the earthquake intensity, the overall height-to-width ratio of the building needs to be limited in order to avoid potential problems related to foundations behaviour and soil capacity due to uplift.

A foundation raft is suggested. It is suggestedthat the foundation elements for structures important to safety be located at the same elevation and on the same foundation type of soil.

Inverted pendulum structures are not allowed for EEC1 structures.

Structural detailing needs to avoid brittle parts, even for those elements that are not assumed to take significant part of overall forces.

Doors and other openings in floors and walls need to be sized and located in such way they do not introduce significant discontinuity in the resistant scheme of the structure.

Precast panels (or other prefabricated elements) need to be connected in such a way that they behave as an integral unit during an earthquake.

Some non-structural elements may affect the dynamic behaviour of the structure and its capacity. It is typically the case of masonry filling in framed reinforced concrete structures which can lead to shear damage (and rupture) of the so called 'short column' configuration.

In the case of design of new buildings, this solution needs to be avoided. In the case of re-evaluation of existing buildings, their capacity needs to be assessed together with their potential effects on the dynamic behaviour of the structure and on the forces in the structural elements.

6.1.2. Design classes for structures

With reference to the safety classification described above and to the 'single parameter control' approach, the minimum requirements imposed to structures, during and after an earthquake, can be summarized as in the following:

- For Design Class 1 (DC1) structures, full functionality, is required. No cumulative damage is allowed in order to withstand next earthquake exactly as the first one. This implies a quasi-elastic behaviour of the whole structure in an event of the design basis earthquake (DBE). This behaviour is referred to the global structural response, ignoring the unavoidable cracking of concrete before and during the earthquake. This condition is considered sufficient but not necessary to comply with the safety requirements.
- For Design Class 2 (DC2) structures, capability of supporting safety related components, equipment and systems needs to be maintained. This requires limited inelastic deformation and is guaranteed by lower ductility coefficients than for DC3 structures in the event of the DBE. The limited values of the coefficients are defined in Table IX.
- For Design Class 3 (DC3) structures, non-collapse allowing for inelastic behaviour is required. This is guaranteed by ductility coefficients given in Table IX.
- For Design Class 4 (DC4) structures, conventional design standards for industrial buildings can be applied.

In general, the allowable stresses need to be the same for the different categories of design class. The difference between design classes needs to be only in terms of different ductility values associated to the internal forces.

The use of ductility coefficients is associated to a careful application of special detailing provisions defined in national seismic codes. For ductility values higher than 2, such provisions need to be put in place and the ductility values justified.

6.1.3. Geometric modelling

Analysis methods for building design have to be adapted to the general layout of the building. In general, the more complicated is the layout, the higher conservatism needs to be inserted to compensate for calculation uncertainties.

The main issues which affect the modelling strategy are:

– the possibility to decompose the building into regions that have consistent displacements (masses) for each direction of earthquake;

– the possibility to easily evaluate the stiffness of the connections between these regions and to take account of their connectivity order and topology. Inside a single region, there needs to be a strong connectivity in order to keep displacements compatible.

TABLE IX. DUCTILITY COEFFICIENT μ.

Type of Structural System	DC2 **Coefficient** μ	DC3 **Coefficient** μ
SHEAR WALL SYSTEM: A structural system bearing walls providing support for all, or major portions of the vertical loads. Seismic force resistance is provided by		
□ Reinforced concrete walls	2.5	4
□ Reinforced masonry wall	1.5	2.5
MOMENT RESISTING FRAME Seismic resistance is provided by Moment Frames capable of resisting the total prescribed forces. Code detailing rules are very carefully applied.		
□ Steel frame	3	4.5
□ Reinforced concrete	2.5	4
DUAL SYSTEM Mixing of the above systems.	max.2.5	max.4.0
INVERTED PENDULUM STRUCTURES: Structures where the framing resisting the total prescribed seismic forces acts essentially as isolated cantilevers and provides support for vertical load	1.0	max.1.2

Between two connected regions, the scheme of force transfer and associated deformations need to be split into subsets that act in parallel according to the following categories:

– bending beam behaviour;
– shear beam behaviour;
– compression/tension beam behaviour;
– torsion beam behaviour;
– composite beam behaviour,
– frame behaviour.

In some cases two or more model typologies can be connected to a single mass representative of the foundation.

Such hypothesis of decomposition has to be validated with appropriate sensitivity analysis.

Concerning the modelling of any region, a stick model can be used provided the following conditions are met:

- The structure shows a global beam type behaviour.
- The structure has floors which can be assumed to be rigid in their plane, compatible with a 6 degree of freedom mass located at its centroid.
- Local deformation of walls between floors is assumed to be negligible.
- It is assumed that between two consecutive masses, the overall stiffness can be modelled by equivalent material properties assigned to the equivalent sections.

In case a stick model is not a suitable approach to the modelling of the region, 2D or 3D models need to be developed.

6.1.4. Soil structure interaction

The dynamic response of the coupled model of soil and structure needs to be evaluated taking into account the behaviour of the soil region around the foundation and of the seismic waves propagating into it. If SSI effects are deemed to cause beneficial effects, then the analyses may be carried out without incorporating SSI.

When SSI is considered, the relative behaviour of unbounded soil and foundation submitted to seismic waves needs to be analysed first and modelled with a set of springs and dashpots. In a second step, the structure supported on this spring dashpot system needs to be computed. Appendix II provides the formulas for calculating the stiffness of the equivalent spring and the damping values to be assigned to a soil spring dashpot system applied at the base of the structure model for rectangular/circular foundation shape.

Soil modelling might strongly affect the response of structures. Therefore a variation in soil properties needs to be considered for sensitivity purposes.

It is suggested to take into account the uncertainty on soil modulus, as suggested in Chapter 5, by considering 3 different values for the soil shear modulus (G). That range is expected to envelope also the effect of the presence of other buildings of comparable dynamic behaviour in the neighbourhood.

The design of structural elements needs to be carried out with reference to the envelope of all the calculated results.

In case of important stratification with a soft upper layer, also the radiation damping needs to be subjected to sensitivity analysis.

Special attention needs to be paid to the case of the design of a small building beside a large one: in such a case, the soil–structure interaction of the large building can determine the motion imposed at the base of the small one.

6.1.5. Non-structural loads

Non-structural loads need to be accounted for in the calculation of the seismic response. These are:

- Itemized large equipment and their content (i.e. known individually).
- Non-itemized numerous small equipment generally represented as a uniformly distributed permanent load for each floor area. Note that some are suspended under floors or on walls (cable trays, ventilation ducts, etc.).
- Live operating loads.
- Other simultaneous loads as external water or earth static and dynamic pressures.
- Static and dynamic thrust of stored material, depending on its storage conditions, solid, liquid, powdery, etc.

All itemized and non-itemized equipment loads and their content need to be considered.

Live operating loads have to be considered according to their probability of occurrence. For example water in a storage pool and the content of the storage itself need to be accounted for without reduction. For live loads, coefficients taking account for the probable non simultaneousness of maximum values of the live load in each area of the building may be considered. Table X suggests some coefficients for live load inclusion into floor equivalent non structural masses.

The most unfavourable mass configuration needs to be considered in the analysis. In some cases, large masses (cranes) may have different location in the building, and therefore the building has to be designed for any possible position of displaceable items. For heavy storage the designer has to account for several mass configurations of the building.

TABLE X. LIVE LOAD COEFFICIENT

Use of Floor	Live Load Coefficient n
Heavy weight components such as large baggage in the wide space	0.50
Medium weight components in medium size space	0.30
Light weight components such as desks, books and lockers in small size space.	0.25

Note:
The weight of the water in the reactor pool needs to be considered as dead load. The weight of permanently installed equipment and distribution systems at maximum normal operating weight needs to also be considered as dead load.

6.1.6. Spectral analysis

In this section, a general description of spectral analysis is provided. The spectral analysis can be used to evaluate the dynamic response of a structure which is represented by a stick model.

In a spectral analysis the calculation steps are as follow:

(1) computation of the eigenmodes (normalized displacements, forces, reactions, strains, stresses etc. at the different locations in the model) of the structure and associated frequencies and damping ratios;
(2) computation of modal participation factors for each earthquake direction;
(3) computation of the spectral accelerations for each mode and earthquake direction, depending on seismic spectra in each direction and modal frequencies and damping ratios;
(4) computation of the spectral response (displacements, forces, reactions, strains, stresses etc. at the different locations in the model) of each mode for each earthquake direction;
(5) combination of spectral response of modes for an earthquake direction, deriving the overall response of the structure to this earthquake direction;
(6) combination of the three responses to each earthquake direction, deriving the overall response.

It is suggested that an additional horizontal load eccentricity equal to 5% of the building horizontal dimension in any direction is added on to account for torsional moments coming from unmodelled stiffness or mass eccentricity.

In Step 1, the modal damping ratio is calculated by averaging the damping values of each part of the whole system, using the following procedure:

The structure is divided into *p* parts, each one having a unique damping ratio equals to ξ_j. $\overline{K}^j$ is the stiffness matrix reduced to the j-th part. $\overline{\phi}_i$ is the i-th modal vector, reduced to the j-th part.

Averaged modal damping ratio is then given by:

$$\xi_i = \frac{\sum_{j=1}^{p} \xi_j E_j^i}{\sum_{j=1}^{p} E_j^i}$$

where,

$$E_j^i = \overline{\phi}_i^T \overline{K}^j \overline{\phi}_i$$

The value of ξ_j should not exceed 20 %.

In Step 5, for each earthquake direction, a choice of the number of modes retained for response analysis has to be made according to the following criteria: the sum of effective masses of all modes retained in each direction need to be greater than 90% of the total mass of the building. If this criterion is not met, some supplementary modes have to be accounted for if their frequency is less than the cut-off frequency (f_c) of the spectra, or a static correction (missing mass procedure) has to be used.

The modes need to then be combined according to the Complete Quadratic Combination (CQC) [44], for example or similar procedures.

Alternatively, if it is shown that the eigenfrequencies of the model are separated enough, typically more than 10% difference between two consecutive frequencies of significant modes in the considered direction, the Square Root of Sum of Squares (SRSS) rule can be applied, as explained in the following.

The maximum response value, Umax, can be calculated according to the following equation (given for the acceleration):

$$\text{Umax} = \sqrt{\sum_j \left|\beta_j \ \phi_j S_{Dj}\right|^2}$$

where, $\beta_j \phi_j$ = j-th participation function obtained by eigenvalue analysis

$\sum_j$: the summation for j which takes the value from 1 to the maximum number of the superposed mode including possibly the static correction (or 'missing mass mode).

S_{Dj} = Spectral acceleration of the mode in the considered direction

If a missing mass mode is used, it needs to be considered as having a frequency equal to f_c and a damping ratio equal to 5% for the calculation of the cross coefficient with other modes in the use of CQC procedure.

Step 5 needs to be applied for each direction. Maximum scalar values needed for the structural design are evaluated: displacements, accelerations, forces, reactions, strains and stresses.

In Step 6, the overall response to the three earthquake directions is obtained by simple quadratic combination of each earthquake direction response. Alternatively, it is possible to use the Newmark rule:

$$Gmax_{xyz} = \max \begin{cases} 1.0*Gmax_x+0.4*Gmax_y+0.4*Gmax_z \\ 0.4*Gmax_x+1.0*Gmax_y+0.4*Gmax_z \\ 0.4*Gmax_x+0.4*Gmax_y+1.0*Gmax_z \end{cases}$$

Any variable of interest needs to be first calculated in each mode and direction, then combined for each direction through Step 5 and at last combined for the three directions through Step 6.

When checking the structure, maximum scalar values need to be considered with the most unfavourable sign.

The spectral analysis is also the basis for floor response spectra calculation. However, in case the foundation uplift due to earthquake is more than 30% of the foundation area, its influence on the floor response spectra due to the non linear behaviour of the foundation cannot be ignored and specific computation methods have to be applied for their generation.

The horizontal design response spectra defined in Sections 4 and 5 may be modified by ductility and damping values to derive the response acceleration for each mode, S_{Dj} in the following way:

$S_{Dj} = (a_g \cdot S_j \cdot D_j) / \mu_j$

For $T_j \leq 0.1s$: $\mu_j = 1$

$0.1s < T_j \leq 0.5s$: $\mu_j = (2\mu - 1)^{1/2}$

$0.5s < T_j$: $\mu_j = \mu$

where, μ is defined in Table IX, and

a_g = design acceleration applicable at the foundation level

S_j = Standard Response Factor for the j-th mode period, T_j. The factor is determined from §4.2.3

D_j = coefficient related to the damping ratio ζ_j for the j-th mode (modal damping) to be used as indicated at 4.2.3.

The structural damping value is given in Table XI, and the composite modal damping ratio for the soil–structure system needs to be calculated using the formulas given in the Appendix II.3 Modal damping needs to be limited to 20%.

TABLE XI. STRUCTURAL DAMPING VALUE

Type of Structure	Percentage of Critical Damping ζ (%)
Bolted or riveted steel structure	3
Welded steel structure	2
Reinforced concrete or Steel concrete structure	5
Prestressed concrete	4

Note: These values are rather conservative because of the variety of structural details. To select more realistic values to be used with more refined analysis methodologies, appropriate references need to be used.

6.1.7. Simplified dynamic approach

This simplified dynamic analysis method is applicable to all categories of structures with the condition that they are relatively regular structures, with two perpendicular vertical symmetry plans and essentially uniform mass and stiffness distribution.

The main limitations of the method are listed here below, with reference to the stick model structures. Such list needs to be used also to verify the applicability of the method to the structure of interest.

- The 6 degrees of freedom (DOF) mass representation is limited to 3 translational degrees of freedom in any floor and decomposed in two horizontal and one vertical degree of freedom problems.
- Dynamic load distribution is defined according to a single mode shape with a frequency given by a formula depending on the main parameters of the building layout only.
- The vertical motion is neglected.
- Soil structure interaction effects are neglected.

The procedure for the calculation of horizontal loads is as follows:

- Determination of floor masses, with the contribution of equivalent non structural masses.
- Determination of the location of the centre of gravity for the whole building.
- Determination of the building frequency in the considered direction through the following formula:

$$T_b = \alpha H_B / \sqrt{D_B}$$

where,

H_B = height of the building with respect to the foundation level (metres)
D_B = width of the building in the direction parallel to the horizontal excitation (metres)
α = 0.10 for rigid reinforced concrete structures (walls)
0.12 for ordinary reinforced concrete structures (frames).

- Determination of the acceleration distribution in the height of the building (each floor level), as follows:

$$F_i = a_g \cdot C_{Di(hx)} \cdot W_i$$

where,

a_g = design acceleration applicable at the ground surface level as defined in previous Sections
W_i = the total weight of the building at the i-th floor augmented with the contribution of non structural masses
$C_{Di(hx)}$ = function related to the building characteristics and defined as :

$$C_{Di} = (D_1 \cdot D_D \cdot D_{3(hx)}) / \mu_b$$

where, μ_b is defined as in §6.1.6

D_1 = Standard Response Factor for the fundamental period, T_b. The factor is determined from Figures 5 and 6 in §4.2.3
D_D = coefficient as defined in §4.2.3 related to the damping ratio, ζ, to be determined from Table XI

D3(hx) is the load distribution function throughout the height of the building and is defined as:

$D_{3(hx)} = 1 + 0.5\ (h_x/H_b)$ (not smaller than 0.5 when $h_x<0$)

where, h_x = height of the point or storey under consideration with respect to the centre of masses of the whole building (+ sign for above, – sign for down)

H_b = height of the centre of masses of the whole building with respect to the foundation level

If the fundamental period of the structure, T_b is shorter than 0.3 s, the product $D_1\ D_D$ should not be less than 1.5.

In the vertical direction, a constant acceleration equal to the half of a_g can be applied statically.

The simplified dynamic analysis may be carried out separately for each input direction. Responses resulting from the two horizontal directions need to be combined using one of the following methods.

(1) Combination of torsion and orthogonal stresses

- (i) Distribution of the torsional moments is carried out according to the torsional rigidities.
- (ii) At the intersection points of X,Y (column, etc.) Especially at the corner of the building, simultaneous stress in x and y directions is estimated by combining stresses in each direction using SRSS or CQC method.

(2) Considering the orthogonal effect of resultant internal forces due to defined seismic input motions, results can be combined as follows:

In the x direction:

gravity loads +/– 100 % of forces due to x direction motions +/– 40 % of forces due to y direction motions, and

In the y direction:

gravity loads +/– 40 % of forces due to x direction motions +/– 100 % of motions due to y direction motions.

The +/– sign indicates that the design is performed for maximum member strength.

Responses resulting from horizontal and vertical direction need to be vectorially combined.

6.1.8. Earthquake induced element forces

The main goal of this section is to define an additional parameter to be applied to element forces to account for the uncertainties included in the proposed methodologies, in addition to what has been already made with respect to the category of the structure through the use of ductility.

This parameter is related to:

- the complexity of the building;
- the uncertainty due to calculation methodologies;
- the objective (design or re-evaluation) of the calculations.

Only the earthquake induced element forces, including sloshing effects, underground water and soil pressure dynamic increments, are affected by this parameter.

The following Table XII provides the coefficient that needs to be used, according to the selected scenario, as multipliers of the earthquake induced element forces before combination with other loads and comparison with the element capacities.

TABLE XII. CORRECTION FACTOR FOR SEISMIC RESULTS

Value for design / Value for re-evaluation	Very regular layout	Medium regular layout	Irregular layout
3D analysis method	1.0 / 1.0	1.1 / 1.0	1.2 / 1.0
Stick model	1.05 / 1.05	1.2 / 1.1	1.3 / 1.3
Simplified dynamic analysis method	1.1 / 1.1	Not used	Not used

6.1.9. Load combination and acceptance criteria

The earthquake loads need to be combined with normal operating loads. In general, the definition of combinations needs to comply with [22]. Attention needs to be paid to the combination of static normal operation loads and seismic loads resulting from spectral analysis, where the signs of different components are lost. If the response in each direction is governed by the first mode, it is acceptable to consider the relative signs of the first mode.

Thermal loads can generally be ignored in the load combination with seismic effects for reinforced concrete structures.

The general loading combination for earthquake can be:

1.0*normal loads + 1.0*seismic loads

The normal loads need to take the buoyancy (and lateral pressure for external walls) due to underground water if any.

The coefficient 1.0 applied to seismic loads does not depend on the design class of the building because the design class of the building is already taken into account when applying the μ ductility coefficients, as stated in para. 6.1.2.

In general, the acceptance criteria need to be the same for the different design classes. They can be taken from any national building code for accidental conditions. The difference between design classes needs to affect only different ductility values associated to the internal forces according to para.6.1.2.

National building codes or standards for materials such as concrete and steel generally specify an increase in allowable stresses for extreme loads such as earthquakes.

These increased values may be used as the elastic limits for the earthquake resistant design class 1 to 3 also. If such an increase is not specified in the national codes or standards, then an increase of 33% may be taken in the allowable stresses for concrete (compressive and shear), structural steel (tensile, compressive and shear) as well as foundation soil (bearing capacity). However, applied buckling loads need to not exceed 0.75 the critical buckling load.

6.2. AIRCRAFT CRASH

6.2.1. Protection strategies

The protection of NFOPs against ACC needs to take advantage of the generic characteristics of the NFOP layout (usually very dispersed) and of the low inventory of radioactive material they store at the same time.

In this framework, the broadest application of the segregation (physical separation) principle is suggested together with a large use of isolation devices (valves, barriers, etc.) for the confinement of the damage to the most limited area.

However in many NFOPs there are unique systems/structures, safety related, which collects most of the radioactive inventory of the facility (centrifuges, reactors, reaction vessels, etc.). In these cases a protection of this specific system/structure is due as part of the prevention approach.

Particular emphasis needs to be given to the protection against the effects of fire, ACC induced, which could spread to other areas of the facilities, impairing personnel evacuation, operator actions and the confinement of the damage to the hit area.

6.2.2. Local design

Typically, the local design for an aircraft crash relies on the strength of the external panels of the reinforced concrete structure that is subjected directly to the impact.

Depending on which is the safety function of the panels, the acceptance criteria can be different, from the tightness that requires an elastic reversible behaviour and possibly leads to huge protections, to a simple no-penetration criteria which allows significant plastic excursions and possibly later repairing.

There are two sets of data that could be used in order to make an evaluation of the behaviour of these local elements:

- estimation of mass, stiffness, impact area, fuel amount and speed of the impacting aircraft.
- simplified equivalent estimation of load–time function and impact area.

According to the available data at the site, simplified design formulas for the protecting structures can be applied from the bibliography (see for example Ref. [45]) for an estimation of the required stiffness. In particular most of the analytical formulas provided in the bibliography have been validated in test programs supported by the nuclear industry (e.g. EDF, Bechtel, Sandia, etc.) and therefore they deem very appropriate for the range of application requested to NFOPs. The assumption of a rigid missile is considered too conservative and the relevant formulas need to be disregarded for the present application.

Without an explicit analysis, very expensive for the reasons described above, the following generic criteria may be applied for the design of a concrete barrier: they represent the synthesis of many simulations, both numerical and by test, and allow a reasonable sizing of the structures with a limited amount of conservatism.

- The impacting object may be assumed with characteristics similar to one of the heaviest engine of an aircraft (proved to be the worst scenario for the evaluation of local effects): around 2000 kg travelling at 215 m/s, deformable, impacting on an area of 1.5 square metres in perpendicular direction to the external surface
- The concrete thickness may be chosen in the following range: 90 cm to stop perforation of the missile, allowing significant scabbing on the internal surface, 160 cm to avoid most of the scabbing, allowing only a limited amount of penetration.
- Reinforcement quantities. Longitudinal reinforcement proved to be not essential for prevention of local effects: however, it has to be around 0.4 % of the concrete volume to provide a generic capacity. Shear reinforcement (stirrups) proved to play an important role in the prevention of shear punching failures: an amount of 1.5% in volume can limit the plant size of the punching cone to 2 square metres.

6.2.3. Global design

The evaluation of the global dynamic behaviour of the whole building would require a dynamic analysis of a numeric model subjected to impact forces on any potential impact location. A simplified approach may again rely on conservative assumptions, limiting the analysis to the structures supporting the protecting shields. The force transfer needs to consider the deformation of the impacted area through traditional analytical approaches (see for example [45]).

6.2.4. Load combination and acceptance criteria

For *local* analysis, the load combination for local stress/strain analysis may be:

1.0 Normal loads (dead + live) + 1.0 aircraft crash loads

The acceptance criteria depend on the function required to each structural element. For local design, if the only function of the element is to stop the aircraft and maintain the global stability of the building, it may be designed with plastic excursions of reinforced bars reaching $\varepsilon = 2\%$ deformation.

If the structural element supports equipment that needs to guarantee a safety function, the plastic excursions need to be limited to $\varepsilon = 1\%$ deformation. In both previous cases, the acceptance criteria for concrete in compression need to be $\varepsilon = 0.35\%$.

If the element has a tightness function, no plastic excursion can be allowed, and elastic behaviour has to be guaranteed. In this case, however, it is more convenient to design a shielding structure able to protect the safety related buildings.

For the *global* analyses, the load combination for global stress/strain analysis may be taken as:

1.0 Normal loads + α aircraft crash loads

Where α depends on the design class of the building according to the following Table.

DC	1	2	3	4
α	1.0	1.0	1.0	1.0

The coefficient α is taken at 1.0 for all the design classes as the screening value for the design basis set up at chapter 2 is assumed at 10^{-5}.

The acceptance criteria may be those of any national building code for accidental conditions.

Concerning the fuel effects, a dedicated analysis needs to be carried out to prevent fuel access into the facility. Then, the same criteria described above for explosion and fire may be applied.

6.3. WIND

6.3.1. Load combination and acceptance criteria for extreme wind

Extreme wind loads need to be accounted for simultaneously to the normal loads in a linear analysis according to the following load combination:

1.0 normal loads + α wind loads

in which α depends on the design class of the structure according to the following Table

DC	1	2	3	4
α	1.5	1.3	1.2	1.0

Note: the a coefficient is applied to wind loads, not to wind speed.

Acceptance criteria can be taken from any building code (reinforced concrete, steel construction, etc.) where extreme wind or other accidental loads are accounted for.

6.3.2. Load combination and acceptance criteria for frequent wind

Frequent wind effects depend on the dynamic behaviour of the structure, frequency, damping, geometry, the surface state of its external surface, the use of antivortex device and the wind speed spectra, the life time of the structure, the wind direction if geometry is not horizontally isotropic (like a chimney for instance). Their evaluation allows the calculation of cumulative fatigue: stress amplitude over the number of cycles as a function of wind speed and summation of the fatigue for the number of cycles for each wind speed at its time of occurrence, with summation for all wind speeds. The acceptance criteria need to be selected according to standard fatigue curves of the structure material.

In general, reinforced concrete chimneys show a rather low sensitivity to fatigue because of the compression due to dead load, which prevent the reinforcing bars from tensioning under moderate wind speed. Steel chimneys are much more sensitive to these effects. Special antivortex devices can be put in place in order to break the Karman turbulence: the efficiency of such devices needs to be validated by aerodynamic studies before they are accounted for.

In the calculation of stress range, depending on the design class of the structure, the frequent wind effects need to be multiplied by an α coefficient according to the following Table:

Design class:	1	2	3	4
α	1.5	1.3	1.2	1.0

Load combination for stress analysis needs to be:

1.0 Normal loads + α wind loads

These wind loads need to take into account resonance effects in the direction perpendicular to wind direction due to Karman turbulence. Detailed scientific literature has to be followed in order to calculate this effect. When damping devices are used to reduce these effects, an in situ validation of their efficiency needs to be carried out before operation.

For the calculation of these effects, if no special damping device is used, damping ratio needs to be taken equal to 2% for steel construction and 3% for concrete construction.

6.4. SNOW

6.4.1. Load combination and acceptance criteria

Extreme snow effects need to be taken into account together with the normal loads according to the following combination:

1.0*normal loads + α*snow loads

with a given by the following Table:

Design class:	1	2	3	4
α	1.5	1.3	1.2	1.0

The acceptance criteria may be taken from any classical building code for accidental situations.

Concerning normal snow load (rather frequent), it needs to be considered simultaneously with normal loads and normal (rather frequent) wind.

The load combination may be:

1.0*normal loads + 1.5*(normal wind + normal snow loads)

not depending on the design class of the structure. Capacity needs to be checked according to acceptance criteria of building codes for normal situations.

6.5. EXTERNAL TEMPERATURE

6.5.1. Layout consideration

It is suggested generally as a good design practice that the whole structure is located inside an isolation system, in order to reduce the induced thermal stress field.

6.5.2. Load combination and acceptance criteria

The thermal loads can be analysed with the hypothesis of a linear behaviour of the structure for all construction materials.

However, the reinforced concrete case is a special one because of cracking. In this case, a 0.5 coefficient may be applied on the results of the linear analysis and no further analysis is necessary if results are correct with respect to acceptance criteria. If not, more detailed approaches based on theoretical crack width, pattern, and stress redistribution can be used, with more realistic acceptance criteria.

Since thermal loads is a cyclic one, with one year period, provisions need to be taken in order to avoid cumulative plastic strain in some localized zone of the structure, whether it is a steel structure or a concrete structure. For concrete structure reinforcement needs to be designed to avoid concentrated cracking.

Load combination for extreme thermal loads needs to be:

1.0*normal loads + α*thermal loads

Thermal loads have to be evaluated by difference between extreme and construction temperature conditions.

α depends on the design class according to the following Table:

	DC	1	2	3	4
Linear approach, reinforced concrete	α	0.7	0.6	0.5	0.5
Linear approach, steel or prestressed concrete	α	1.4	1.2	1.0	1.0
Non-linear approach, reinforced concrete	α	1.2	1.0	1.0	1.0

The acceptance criteria can be taken from any construction code for accidental conditions in case of linear approach and according to more realistic criteria for the non-linear approach.

6.6. EXTERNAL FLOOD

6.6.1. Layout consideration

All items that are important to safety need to be placed at an elevation higher than the design water level at the site, after consideration of all the protecting measures. All openings need to be checked: sufficient margin needs to avoid any water inlet into the building.

All external walls need to be designed in order to withstand the external water pressure on top of the earth pressure. As buoyancy pressure on the foundation may affect the stability of the building in case of an earthquake, it may be necessary to install a permanent drainage system.

6.6.2. Load combination and acceptance criteria

External flood loads can be used in a static linear analysis of the building that can consider the local bending of walls due to water pressure and its dynamic effects. If there is a possibility that flood carries sand or other solid materials, a suitable value for density needs to be considered.

If the flood is expected to occur with a strong flow, dynamic effects need to be considered.

Load combination for external flood needs to be

$$1.0 \text{ Normal loads} + \alpha \text{ water pressure loads}$$

with α according to the following Table depending on the design class of the building:

DC	1	2	3	4
α	1.5	1.3	1.2	1.0

Acceptance criteria need to be:

(1) the static stability of the whole building against horizontal sliding displacement and overturning, taking account of buoyancy.

(2) capacity criteria for accidental loads, taken from any conventional reinforced concrete building code.

(3) Low cracking criteria for some water tightness if there is no tightness membrane or similar device. Such criteria may be taken from standard building codes for reinforced concrete.

(4) In case a water tightness membrane is used, its efficiency has to be proven by test, with special care to joints between buildings, underground junctions etc.

6.7. EXTERNAL EXPLOSION

6.7.1. Layout consideration

Generally, masonry on steel or concrete frame are not efficient against explosions. Continuous concrete walls and floors are suggested for that purpose.

It is possible to put an independent protection between potential locations of explosion and the facility to be protected. In such a case, the protection effect has to be validated either by a specific study or by reference to previous experience.

It needs to be noticed that the pressure wave can penetrate inside the facility through doors or air ducts for instance. This can reach directly safety functions located inside the building. Such cases have to be investigated and safety functions assessed with respect to that conditions.

6.7.2. Load combination and acceptance criteria

If the analysis is linear, the load combination for an accidental explosion needs to be:

$$1.0 \text{ normal loads} + \alpha \text{ explosion loads}$$

with alpha depending on the design class of the building according to the following Table:

DC	1	2	3	4
α	1.0	0.9	0.9	0.8

In this case the acceptance criteria may be those of a conventional building code under accidental loads.

If the analysis is non-linear, $\alpha = 1.0$ for all design classes, but the acceptance criteria for concrete needs to be selected according to the following Table (max strain in the reinforcement):

DC	1	2	3	4
$\varepsilon(\%)$	1.0	1.5	1.5	2.0

7. EQUIPMENT AND PIPING DESIGN

7.1. INTRODUCTION

All equipment, components and distribution systems need to be designed to guarantee their safety function during and after design basis external events. Their capacity may be demonstrated by one of the following methods:

(1) Analytical methods, simplified, according to this report.
(2) Other analytical methods, as specified in the references.
(3) Numerical analysis using adequate software
(4) Dynamic test.
(5) Pseudo-dynamic test.
(6) Static test.

In this section, only analytical simplified methods (No. 2 above) for the assessment of structural capacity of equipment, components and piping (with their anchoring) are described for the case of earthquake and aircraft crash. For other external events, reference is made to the methodologies set up for the nuclear power plants, as no significant simplification is foreseen.

7.2. SEISMIC DESIGN

7.2.1. General methodology for seismic force evaluation

The seismic coefficient C_{Di} from the simplified dynamic approach (see previous chapters) may be used to evaluate the static equivalent horizontal and vertical seismic loads on equipment or piping systems, hereafter labelled as F_{Eh} and F_{Ev} respectively, at the height of the equipment, h_x [55].

$$F_{Eh} = a_g \, . \, C_{Di} \, . \, D_4 \, . \, W_e$$
$$F_{Ev} = 0.5 \, F_{Eh}$$

where

W_e = weight of equipment or equivalent weight of piping system

D_4 = Amplification factor due to floor response spectrum as show in Figure 7 and $D_4 \geq 1.0$.

with $A_f = [(\beta'_b + \beta'_e)^2 + m_r]^{-1/2}$

$\beta'_b = \beta_b \, \mu_b$ and $\beta'_e = \beta_e \, \mu_e$ for $T_b \geq 0.5$ s

$\beta'_b = \beta_b \, (2 \, \mu_b - 1)^{1/2}$ and $\beta'_e = \beta_e \, (2\mu_e - 1)^{1/2}$ for $0.5 > T_b > 0.1$ s

$\beta'_b = \beta_b$ and $\beta'_e = \beta_e$ for $T_b \leq 0.1$ s

β_b = damping for building as defined in Table XI

β_e = damping for equipment

examples:	light weight welded assembly	1%
	welded assembly	2%
	bolted assembly	5%
	heavily insulated piping	3–5%
	lightly insulated piping	3%
	small bore piping	2%
	raceways	5–10%

m_r = mass ratio m_e/m_b

m_e = mass of equipment

m_b = effective mass of building interacting with the equipment (in most cases m_b may be reasonably assumed to be the mass of the building at the equipment level).

μ_b = ductility coefficient defined for the building as defined in Table IX

μ_e = ductility coefficient defined for the equipment, with μ_e = 3.0 for distribution systems and = 2.0 for equipment. Larger values may be used based on experimental verification.

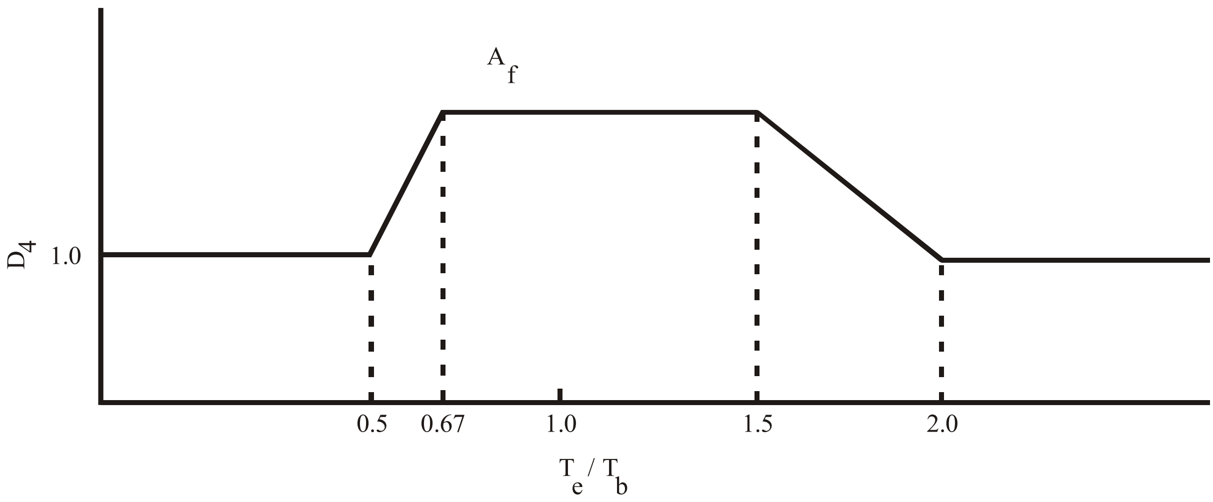

FIG. 7. Amplification factor for piping design.

T_e: Fundamental period of equipment
T_b: Fundamental period of building

In cases where T_e is not evaluated, T_e needs to be assumed to be equal to T_b.

Each horizontal component F_{Eh} needs to be taken separately along the principal horizontal axes of the equipment and combined with the vertical component F_{Ev} taken in the most conservative of the two possible directions.

7.2.2. Simplified analytical evaluation of floor response spectra

In case modal analysis is used to evaluate the dynamic response of the structure, the floor response spectra can be used to evaluate the horizontal and vertical seismic loads on the equipment or piping systems.

According to [46], the floor response spectrum may be obtained directly applying the following equations from the two standard design ground spectra which correspond to the damping value h_B of the structure and h_A of the equipment or piping systems, using modal characteristics of the structure obtained as the result of an eigenvalue analysis (T_{Bi} is the natural period of the mode which provides significant excitation at the elevation where equipment or piping are installed).

$$S_E = \sqrt{\sum \left(\beta U_i . S_{Ei}\right)^2}$$

$$S_{Ei} = \frac{1}{\sqrt{\left\{\left\{1-\left(T_{Bi} / T_A\right)^2\right\}^2 + 4\left(h_A + h_{Bi}\right)^2 \left(T_{Bi} / T_A\right)^2\right\}}} \times \sqrt{\left\{\left(T_{Bi} / T_A\right)^2 . S\left(T_{Bi} . h_{Bi}\right)^2 + S\left(T_A . h_A\right)^2\right\}}$$

where

S_E: Floor response spectrum taking into account every evaluated mode of the structure.

ß U_i : The i-th mode excitation function value of the floor on which the equipment is installed. (It is suggested that the eigenvalue of the structure is evaluated from the first to the i-th mode up to 30 Hz)

S_{Ei}: The maximum value of absolute acceleration response of the equipment and piping system under the i-th mode acceleration of structure.

h_A: Damping factor of equipment and piping system.

T_A: Natural period of equipment and piping system.

h_{Bi}: Damping factor of structure.

T_{Bi}: Natural period of structure.

$S(T_{Bi}\ h_{Bi})$: The standard design ground spectrum corresponding to T_{Bi} h_{Bi} of the structure.

$S(T_A\ h_A)$: The standard design ground spectrum corresponding to T_A h_A of the equipment.

Notes

(1) The mass m_A of the equipment and piping system needs to be sufficiently smaller than the mass m_{Bi} of the structure, and the response of the structure needs to not be affected by the response of the equipment and piping system.

(2) At least 10% broadening of floor response spectrum needs to be taken into account to cope with the uncertainty of eigenvalue analysis of the structure and the equipment and piping systems.

(3) If the structure and/or the equipment and piping system are not liner system the eigenvalue and damping value can be evaluated applying the equivalent linear analysis.

As a next step, an eigenfrequency analysis and response analysis for the equipment can be performed by using the floor response defined above and assuming an appropriate damping for the piping system. Much software has been developed for this purpose. The adequacy of software needs to be demonstrated by a comprehensive test programme.

7.2.3. Seismic design of containers

According to the results of the calculation of the natural period obtained using a model of a single mass system, the design seismic load is calculated by the equations of § 7.2.1. The obtained design seismic load is used to perform stress evaluation for the barrel body, support legs, anchor bolts and other parts.

Buckling analysis, with analytical or numerical methods is usually requested.

Examples of the calculation methods of standard natural period and stress are presented in Refs [47, 48].

7.2.4. Seismic design of pumps and valves

In principle, calculation of the natural period of each piece of equipment may be performed using a model of a single mass system under appropriate support conditions compatible with the shape of the equipment. The motor portion of the equipment mounted on the shaft can be modelled as a single rigid body, and the natural period does not have to be calculated. After the determination of the design seismic load by the equations in § 7.2.1, stress evaluation of anchor bolts needs to be performed, according to next sections.

Examples of calculation of standard natural period and stress for components are presented in [39].

7.2.5. Seismic design of equipment and piping

Piping system design, including other distribution systems (raceways and ducts) can be carried out by one of the following methods.

7.2.5.1. Constant pitch design

This method has been developed for conventional industrial plants and small bore piping in hot cells of fuel reprocessing plants. The main goal is the adjustment of the pitch of supports and making the lowest eigenfrequency higher than the dominant response frequency of floor response. The span of systems evaluated with such method should not be longer than the length shown in Table V.1. in Appendix V. More details are given in Refs [49, 50].

Constant pitch design is a simple and conservative design method, but care needs to be used in case of dominant thermal expansion effects. In this case, a dynamic analysis needs to be applied. Full dynamic analysis needs to also be used when a minimization of the number of supports for large diameter piping is requested.

7.2.5.2. Simplified response analysis

Piping systems can be modelled as single degree of freedom systems. By using Rayleigh Method, the fundamental eigenfrequencies can be calculated as well as their equivalent mass value. Then internal forces can be obtained by a static elastic analysis. For fundamental modes (one or three), the simplified dynamic approach described in previous sections can be applied.

7.2.5.3. Allowable stresses

The maximum primary stress in equipment and piping due to earthquake and other applicable loads should not exceed f_y (specified minimum yield stress).

The maximum stress ranges in piping considering combined axial (membrane) and bending (primary plus secondary) due to earthquake and other applicable loads should not exceed 2.0 f_y (specified minimum yield stress) of the piping material at the design basis temperature. Higher stresses are allowed if they are substantiated by a fatigue and ratcheting analysis [39].

7.2.6. Other considerations

(a) Piping running between different buildings/structures
For the portion of piping running between different buildings or structures, the relative displacement of the two buildings/structures needs to be taken into consideration.

(b) Pipe connection portion to equipment
In principle, support needs to be made as near to the equipment as possible. Also, when the operating temperature of the equipment is high, the thermal expansion of the equipment needs to be taken into consideration. In addition, the nozzle reaction force acting on the equipment needs to be within the allowable range.

Branch connections, connections between components as well as that of component to wall, ceiling and/or floor need to be flexible. If a component is supported by shock isolation mounts, the vibratory motion (e.g. rocking and translation) of the component needs to be taken into account in the design of the shock mounts (e.g. to avoid components overturning or jumping off its mounts). In addition, all connections made to such a component need to be flexible enough to accommodate relative movement without excessive force or stress to such connections.

Some good practices on detailing of connecting cable and piping between components are shown in Appendix VI.

(c) Buried piping

In this case, the behaviour of the soil during earthquake, the relative displacement between building/structure and ground, and the thermal expansion of the piping need to be estimated.

(d) Adjacent piping

Arrangements need to be made to ensure that there is no mutual interference between piping caused by earthquake induced displacement.

(e) Overhead cranes, bridges and platforms and other overhead items

Overhead cranes, bridges, platforms, etc., located above critical components need to be designed to resist the seismic force induced by the acceleration specified for the earthquake resistant structures, even if their operation is not essential.

Provisions need to be taken to preclude damage due to breaking of plaster, window glass, lighting fixtures, and other brittle components above the critical components. (see Table III.1. design provision in Appendix III).

(f) Leak tightness requirement

For equipment required to be leaktight during and after an earthquake, such as glove boxes, leaktightness needs to be assessed by dedicated analysis or demonstration test.

7.3. EQUIPMENT DESIGN FOR AIRCRAFT CRASH LOADS

In case some safety related equipment inside the building have to meet functionality criteria during and after an aircraft crash, floor response spectra need to be computed from the building model dynamic response. Local transfer functions need to then be evaluated according to the equipment location and supporting scheme.

A conservative approach may rely on the following assumptions:

- In case a military aircraft is considered as a reference impacting missile, the mass is around 14 t and the speed is around 200 m/s. An equivalent load function on a rigid target has peak at 100 Mn for 20 ms. Most of the frequency content is in the range 20–40 Hz where most of the structural floors and panels shows their first bending mode.
- A 'hard' impact on a rigid structure induces a high energy transfer to the structure: an acceleration of 1 g in the range 20–40 Hz at 10 m from the impact point represents a reference value for design of the protection of safety related equipment
- The safety related equipment may be protected either by moving it to a more protected area or by base isolation
- Protection needs to be provided against internal debris, dust and fire

7.4. ANCHORAGE DESIGN

Most failures related to equipment and distribution systems, including piping systems, occur at their connections to the supporting structure. These failures are at least as dependent on the displacement demand imposed by differential support movements, (seismic anchor motions) as they are on inertia forces in the equipment.

Some simplified design techniques for anchoring and supports are discussed in the following, but it needs to be reminded that they need to be followed by effective quality control measures to guarantee their performance under seismic conditions.

In most cases, the capacity of the anchoring device and bolts are based on the strength of concrete and therefore the design is based on concrete evaluation. However, this practice cannot be applied for some types of anchor bolts which are installed after concrete maturing which are sized according to their capacity.

A method for the calculation of pulling force on anchoring device for 'concrete sized' anchoring is shown in Appendix IV.

Design methods for anchor bolts can follow the procedure outlined here below.

(i) Cast-in-place anchors, with at least 6 diameters embedded length, need to be used wherever possible, to obtain maximum pull-out strength. Where it is absolutely necessary to use drilled-in (expansion type) anchors, they need to be of a proven type, easy to install and resistant to slippage or loosening under severe vibration or impact loading.

(ii) Where high strength studs or anchor bolts are applied, they need to be preloaded to 90% of their specified minimum yield strength or reduce the risk of prying or uplift during an earthquake, minimizing fatigue effects and as a convenient means of pretesting the anchor connection. Where low carbon, structural steel anchors are used and/or sustained preloading is not dependable, the effects of prying and fatigue need to be taken into account. In addition, consideration needs to be given to the adverse effects of flexible mounting plates, long reach bolts or flexible concrete slabs in which anchor bolts are installed.

(iii) Transverse shear forces need to be assumed to be applied directly to the bolts, unless shear keys are provided. Without such keys, shearing, as well as tension stresses, need to be taken into account, using suitable interaction formulae (square law relationship).

(iv) Consideration needs to be given to the use of redundant anchors.

(v) Care needs to be taken in spacing of anchors and in the distance between anchors and any free concrete edge, wall or corner, to ensure maintenance of adequate pull-out strength. Otherwise, a suitable strength reduction factor needs to be applied to such anchors. Consideration needs to also be given to bolts for anchoring equipment to floors and especially walls and ceilings, where adequate spreader plates are used on the opposite side of the floor, wall or ceiling.

(vi) For cast-in-place anchors, the minimum factor of safety against failure in any mode, including pull out, needs to be 2.5.

(vii) For drilled-in, expansion type anchors, the minimum factor of safety, against failure in any mode, including pull-out, needs to be 4.

It needs to be understood that positive anchorage in the form of anchor bolts needs to be used to carry uplift due to overturning effects for all equipment. Anchor bolts in the absence of engineered shear keys need to be designed to carry applicable shear, except where the shear friction capacity between the component and its foundation can be shown to carry applicable lateral loads with a safety factor of at least 2.0.

Capacities of anchor bolts of various type and size and under different loading and geometric conditions are typically given in national codes and manufacturer's installation specification.

8. SLOSHING EFFECTS

Sloshing may be produced in a pool or tanks by strong earthquakes. This phenomenon may generate waves in the pool which may interact very strongly with the bridge, from which the reactor and its control system are suspended and submerged structures near to the water surface. Sloshing can begin several tens of seconds after arrival of the first higher frequency seismic waves at the site as its typical natural frequencies are usually much lower than the structural ones.

The evaluation of the sloshing phenomenon can be of interest for two main reasons:

- the estimation of the dynamic interaction between the structure and the pool or tank
- the evaluation of the wave height (for water runoff) and therefore of the hydrodynamic pressure on the container wall

For both goals, an equivalent system with two masses and stiffnesses may be used. The liquid may be replaced with a mass M_0 rigidly fixed to the tank at an elevation H_0 above the bottom, plus a mass M_1 attached through springs of total stiffness K at elevation H_1. For a cylindrical tank with flat bottom these parameters are given by [51]:

$$M0 = [(\tanh 1.7R/H)/(1.7\ R/H)]*M$$
$$M1 = [(0.71 \tanh 1.8H/R)/(1.8H/R)]*M$$
$$H0 = 0.38\ H\ [1.0+\alpha(M/M_0-1)]$$
$$H1 = H[1.0 - 0.21\ M/M_1(R/H)^{\wedge 2}+0.55\beta R/H\ [0.15(RM/Hm_1)^{\wedge 2}-1.0]^{\wedge -1/2}$$
$$K = 4.75\ g\ M_1{}^{\wedge 2} * H/(MR^{\wedge 2})$$

where g is the acceleration of gravity, H the height of the tank and R its radius.

An alternative approach is provided in [52] and [44].

The corresponding solution for a rectangular tank that measures 2L in the direction of motion is

$$M0 = [(\tanh 1.7\ L/H)/(1.7\ L/H)]*M$$
$$M1 = [(0.83 \tanh 1.6H/L)/(1.6H/L)]*M$$
$$H0 = 0.38H[1+\alpha(M/M_0 - 1.0)]$$
$$H1 = H\ [1.0 - 0.33M/M_1(L/H)^{\wedge 2}+0.55\beta L/H\ [0.28\ (LM/HM_1)^{\wedge 2} - 1.0]^{\wedge -1/2}$$
$$K = 3.0\ g\ M_1{}^{\wedge 2} * H/(ML^{\wedge 2})$$

For both shapes of container, $\alpha = 1.33$ and $\beta = 2.0$, if the hydrodynamic moment on the tank bottom is to be included in the computation, while $\alpha = 0.0$ and $\beta = 1.0$, if only the effects of hydrodynamic pressures on the container walls are of interest.

The solution for a cylindrical tank with a hemispherical bottom may be taken to be equal to the one for a tank with flat bottom of the same radius and same volume as the tank in question.

The amplitude of the height of waves set up by the vibration may be taken equal to the horizontal displacement amplitude x of mass M1 times the factor

$\eta = (0.69\ K\ R/M_1 g)/[1.0 - 0.92\ (x/R)(KR/M_1 g)^{\wedge 2}]$ in cylindrical tanks,

and

$\eta = (0.84\ K\ L/M_1\ g)/[1.0 - (x/L)(KL/M_1 g)^{\wedge 2}]$ in rectangular tanks.

These expressions are satisfactory provided ηx does not exceed about 0.2 R, o.2 L, or 0.02 H. Beyond these limits non-linear phenomena become important.

Energy dissipation due to viscosity of the liquid can be expressed as an equivalent percentage of critical damping. This quantity decreases rapidly with increasing linear dimensions of the container and is only a small fraction of 1% for tanks of practical interest.

For small values of H/R or H/L, the approximation $T_1 \cong 1.07\ R/H^{\wedge -1/2}$ and $T_1 \cong 1.25\ L/H^{\wedge -1/2}$ are useful for estimating the fundamental period of liquids in cylindrical and rectangular tanks respectively (T1 is in seconds and H, R, L in metres). The error introduced by these expressions does not exceed 2% when H/R is smaller than 0.25.

Provision needs to be made for water overflowing pools (reactor or fuel storage) and potential loss of essential coolant or shielding. In addition, the risk of radioactive material escaping from the facility needs to be taken into account in the event of earthquake induced sloshing. Consideration needs to be given to containing such overflow and returning it to the pool.

Sloshing in closed tanks can produce a strong vacuum behind the surface wave. Vacuum breakers need to be considered in the tops of tanks to avoid collapse in the event of a large earthquake.

9. OTHER CONSIDERATIONS

9.1. SEISMIC SCRAM SYSTEM

The design bases for reactor protection and shutdown function associated with seismic event need to be addressed in the safety analysis report for facilities built in seismic areas.

As an example, for Class A research reactors (see Chapter 2) a seismic scram system (automatic seismic trip system (ASTS)) needs to be provided.

A monitoring system needs to be provided even in case an ASTS is not installed, to drive the operator action and the post event inspections. In this framework, the system needs to be safety related.

For NFOPs, consideration needs to be given to automatic actions to attain a safe state in case of an earthquake. The facility needs to have protection capabilities in all operating modes and conditions. Operational limits and conditions of seismic scram system including surveillance tests and intervals need to be based on safety analysis regarding seismic events.

Appendix VII gives additional information on automatic seismic trip systems for nuclear power plants, which may drive the decision for post event operator actions also for other installations.

9.2. MONITORING SYSTEMS AT THE SITE

It is suggested that each facility have a minimum environmental monitoring at the site for the external events that proved to be sizing for the design of the facility. In case of seismic areas, at least one strong motion monitoring system at the site is suggested.

Data can be used both to confirm the design basis and as a background information in case the facility needs periodic safety reviews or life extension.

9.3. QUALITY ASSURANCE

The seismic design process requires the use of sound engineering/scientific principles and appropriate design standards. Design requirements, input, process, outputs change, records and organizational interface are controlled.

IAEA safety standards [53] provides detailed recommendations on the QA management for NFOPs.

9.4. EVALUATION OF RADIATION EXPOSURE OF THE PUBLIC DURING AND AFTER EXTERNAL EVENTS

Safety features to withstand external events and potential associated accidents need to be taken into account in the design process.

Therefore it is required that the nuclear facility is designed to accommodate external events by shutting down the reactor, removing the residual heat and controlling basic radiological parameters.

A radiological dispersion analysis through air, water and groundwater is required for any nuclear installation. Results need to comply with basic requirements of Section 2.

The emergency planning needs to be based on such a study and needs to rely on the availability of escape routes from the site and communication lines during and after any design basis external event.

The need of an emergency evacuation plan is discussed in Ref. [1], with reference to the radiological hazard posed by the research reactors. Similar criteria could be used for other NFOPs.

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Appendix I

SIMPLIFIED PROCEDURES FOR ASSESSMENT OF LIQUEFACTION POTENTIAL

Saturated alluvial sandy layers, which have the water table within 10 m from the ground surface, and have D_{50} values on the grain size accumulation curve between 0.02 and 2.0 mm, are vulnerable to liquefaction for the depth between 0 and 20 m. The liquefaction potential of these layers can be estimated according to any of the two methods outlined in this Appendix [I.1].

I.1. ESTIMATIONS OF LIQUEFACTION (METHOD 1)

For soil layers which are judged to be vulnerable, liquefaction potential need to be checked based on liquefaction resistance factor F_L defined by the following equations:

$$F_L = R/L \tag{I.1}$$

where

F_L : liquefaction resistance factor
R : resistance of soil elements to dynamic loads, and

$$R = R_1 + R_2 \tag{I.2}$$

R_1 and R_2 should be determined in accordance with Figs A-1 and A-2 respectively

L : dynamic loads to soil elements induced by earthquake motion evaluated as:

$$L=r_d*K_s*(\sigma_v/\sigma'_v) \tag{I.3}$$

$$r_d=1.0 - 0.015*z \tag{I.4}$$

z depth from the actual ground surface (m)

K_s seismic coefficient for evaluation of liquefaction, taken as:
Design Intensity Level 1, $K_s = 0.13$
Design Intensity Level 2, $K_s = 0.15$
Design Intensity Level 3, $K_s = 0.17$
σ_v: total overburden pressure (daN/cm^2)
σ_v: effective overburden pressure at the static condition (daN/cm^2)

Soil layers having liquefaction resistance factor F_L smaller than 1.0 need to be judged to liquefy during earthquakes. Figs. I.1 and I.2 are graphic illustrations of the first term R_1 and the second term R_2 represented in the following equations which were proposed based upon the results of laboratory dynamic triaxial tests on soil specimens taken from several sites in Japan.

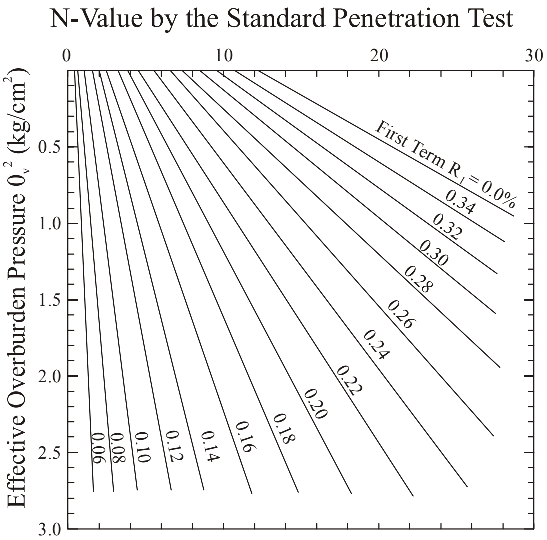


FIG. I.1.

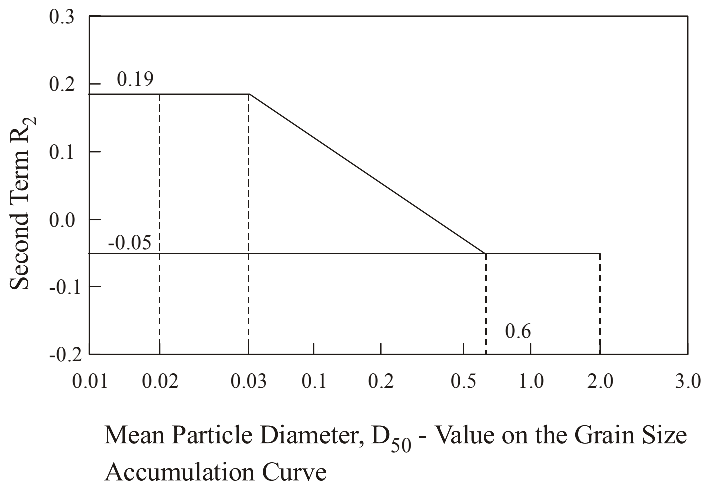


FIG. I.2.

$$R = 0.0882\left[\frac{N}{\sigma'_v + 0.7}\right]^{1/2} + 0.19 \;\; (0.02mm \le D_{50} \le 0.05mm)$$

$$R = 0.0882\left[\frac{N}{\sigma'_v + 0.7}\right]^{1/2} + 0.225\log\left(\frac{0.35}{D_{50}}\right)\left(0.05mm < D_{50} \le 0.6mm\right)$$

$$R = 0.0882\left[\frac{N}{\sigma'_v + 0.7}\right]^{1/2} - 0.05 \;\; \left(0.6mm < D_{50} \le 2.0mm\right)$$

N is the value obtained from the standard penetration test

I.1.1. Treatment of soil layers which were judged to liquefy

For those soil layers which were judged to liquefy according to the estimation of subsection I.1 and are within 20 m of the actual ground surface, bearing capacities and other soil constants need to be either neglected or reduced in the seismic design, by multiplying the original bearing capacities by reduction factors D_E which are determined in accordance with F_L values in Table I.1.

TABLE I.1. F_L VS. D_E RELATION

F_L	Depth, Z (m)	Reduction Factor, D_E
$FL \leq 0.6$	$Z \leq 10$ $10 < Z \leq 20$	0 1/3
$0.6 < F_L \leq 0.8$	$Z \leq 10$ $10 < Z \leq 20$	1/3 2/3
$0.8 < F_L \leq 1.0$	$Z \leq 10$ $10 < Z \leq 20$	2/3 1
$1.0 < F_L$	–	1

I.2. ESTIMATIONS OF LIQUEFACTION (METHOD 2)

An alternative simplified procedure to evaluate liquefaction potential of sandy soils is as follows:

The cyclic shear stress ratio developed in the field due to earthquake excitations can be computed from:

$$(\iota_d / \sigma_o') = \gamma_n (a_g / g) (\sigma_o / \sigma'_o) \gamma_d \qquad \text{(I.5)}$$

in which

a_g = design horizontal acceleration defined in Chapter 6, equation (6.1)
g =gravity acceleration
σ_o = total overburden pressure (daN/cm^2)
σ'_o = effective overburden pressure (daN/cm^2)
γ_d = stress reduction factor defined by $\gamma_d = 1 - 0.015z$
z = depth from the ground surface n meter
γ_n = 0.1 (M – 1)
M = magnitude of the biggest earthquake which can conceivably cause liquefaction at the site

The difference in number of cycles of stress due to different magnitude earthquakes is taken into account in Eq. (I.5) with the reduction factor γ_d.

The soil resistance to liquefaction, ι_1 / σ'_o, can be correlated with some form of modified penetration resistance according to the following procedure:

(a) Compute SPT N_a value normalized to the effective overburden pressure and fines content by:

$$N_a = 1.7\ N/(\sigma'_o + 0.7) + \Delta N_f \qquad \text{(I.6)}$$

in which N = measured SPT N-value

ΔN_f = modification factor in terms of fines content (percentage of fines smaller than 0.074 mm) as shown in Fig. I.3.

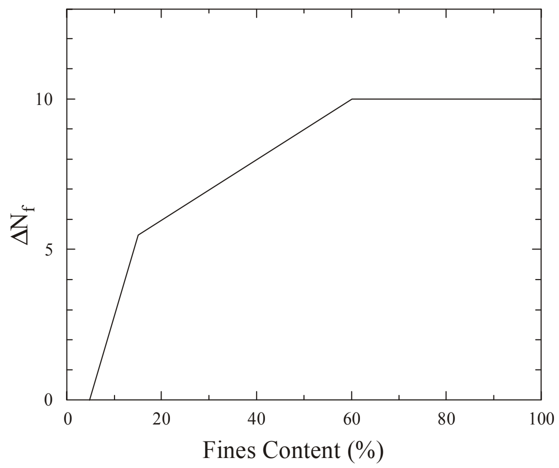


FIG. I.3. Relationship between ΔN_f and fines content.

(b) Determine liquefaction resistance with limiting strain potential of γ per cent by Fig. I.4. The limiting strain potential is the maximum cyclic shear strain likely to be developed by the earthquake excitations.

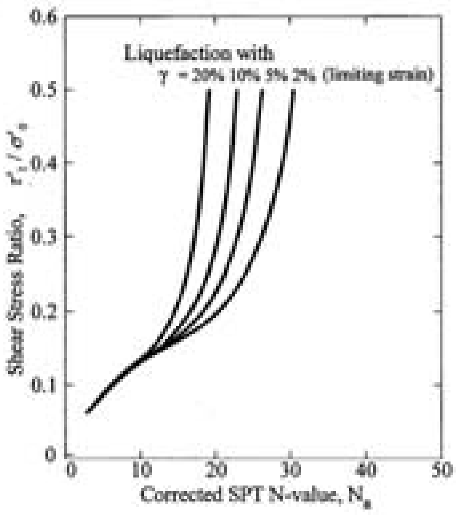


FIG. I.4. Relationship between cyclic stress ratio, N_a value and limiting strain potential of sandy soil deposits

The factor of safety against liquefaction with limiting strain potential of γ per cent, F_1 can be determined by:

$$F_1 = \frac{\tau'_1 \,/\, \sigma'_0}{\tau_d \,/\, \sigma'_0} \tag{I.7}$$

Soil layers with a safety factor less than one can be considered to liquefy during the earthquake. Since the damage of liquefaction takes very different forms depending on the shear strain developed, the following guideline may be tentatively given to specify the degree of liquefaction.

TABLE I.2. DEGREE OF LIQUEFACTION

Factor of Safety	Shear Strain	Degree of Liquefaction
$F_1 < 1$	20%	Extensive
$F_1 = 1$	5–10%	Intermediate
$F_1 = 1$	2–5%	Slight
F1 = >1	2%	No significant

Treatment of soil layer which is judged to liquefy. The soil layer which is judged to liquefy needs to be densified or stabilized so that it can withstand earthquake shaking.

REFERENCE

[I.1] THE JAPAN SOCIETY OF CIVIL ENGINEERS, Earthquake engineering committee, Earthquake Resistant Design for Civil Engineering Structures in Japan, Tokyo, 1992.

Appendix II

SOIL STRUCTURE INTERACTIONS

II.1. GENERAL

This appendix proposes a method for considering the soil–structure interaction to be used with the dynamic stick model defined in Section 6.2. It consists in adding a soil spring-dashpot system at the base of the model. The numerical values of the spring constants (stiffness) together with the damping coefficients are given in Section II.2 of this Appendix. Section II.3 gives information to determine the equivalent modal damping factors to be used in the structural analysis.

In a soil–structure interaction analysis, the dynamic stiffness coefficients of the soil are originally frequency dependent. Here, however, it is suggested to use the static stiffness coefficients as spring constants which are frequency independent. The proposed method is based on classical formulae which may be applied in rather regular subsoil conditions. In case of very irregular subsoil conditions, special methodology needs to be used in accordance with specialists.

Formulae are given for a superficial foundation on a homogeneous half-space and on a soft layer, and for an embedded foundation in a case of homogeneous half-space according to [II.1-II.6]. The designer needs to compare the actual situation with these cases. Soil properties results from geotechnical investigations defined in Section 5.

For the application of the formula, the shear modulus of the soil is needed. It can be evaluated from the shearwave velocity, V_s, by;

$$G = \rho V_s^2 \qquad \text{(II.1)}$$

where, ρ is the mass per unit volume of soil, which in lack of data can be taken from Table II.1 which provides also the Poisson’s ratio, ν.

II.2. EQUIVALENT SPRING AND DAMPING

II.2.1. Surface foundation on a homogeneous half-space

The equivalent spring constants and the damping coefficients are given in Table II.2.

TABLE II.1. SOIL PROPERTIES.

Soil type according to Table 6 at § 5.1.1	1	2	3
ρ (daN/m^3)	2200	2000	1800
ν	0.3	0.4	0.45

It needs to be noted that the formulae for the spring constants depend on the assumed conditions such as the soil–foundation interface.

Those in Table II.2 are results based on the assumption that the soil is subjected to the stress distribution induced by a rigid foundation.

For the damping coefficients, the following applies:

$$C_1 = 0.5 \tag{II.2}$$

$$C_2 = 0.30/(1 + B_\varphi) \tag{II.3}$$

where,

$$B_\varphi = \frac{3(1-\nu)I_0}{8\rho R^5} \tag{II.4}$$

For a rectangular foundation, the radius R, to be used in Eq. (II.4), is an equivalent radius equal to:

$$R = \sqrt{BL/\pi} \text{ for translation} \tag{II.5}$$

$$R = \sqrt[4]{BL^3/3\pi} \text{ for rocking} \tag{II.6}$$

$$C_3 = 0.8 \tag{II.7}$$

TABLE II.2. SPRING CONSTANTS AND DAMPING COEFFICIENTS FOR FOUNDATIONS ON HOMOGENEOUS HALF-SPACE

Direction of Motion	Equivalent Spring Constant for Rectangular Foundation	Equivalent Spring Constant for Circular Foundation	Equivalent Damping Coefficient
Horizontal	$K_H = 2(1+\nu)G\beta_x\sqrt{BL}$	$K_H = \dfrac{32(1-\nu)GR}{7-8\nu}$	$C_H = C_1 K_H R\sqrt{\rho/G}$
Rocking	$K_R = \dfrac{G}{1-\nu}\beta_\psi BL^2$	$K_R = \dfrac{8GR^3}{3(1-\nu)}$	$C_R = C_2 K_R R\sqrt{\rho/G}$
Vertical	$K_V = \dfrac{G}{1-\nu}\beta_v\sqrt{BL}$	$K_V = \dfrac{4GR}{1-\nu}$	$C_V = C_3 K_V R\sqrt{\rho/G}$
Torsion	——	$K_T = \dfrac{16GR^3}{3}$	$C_T = \dfrac{\sqrt{K_T I_T}}{1+2I_T/\rho R^5}$

ν = Poisson's ratio of soil medium
G = shear modulus of soil medium
ρ = density of soil medium
R = radius of the circulate foundation
B = width of the foundation perpendicular to the direction of horizontal excitation
L = length of the foundation in the direction of horizontal excitation
I_0 = total mass moment of inertia of structure and foundation about the rocking axis at the base
I_T = polar mass moment of inertia of structure and foundation

Constants β_x, β_ψ, and β_v for a rectangular foundation should be evaluated in the following Fig. II.1.

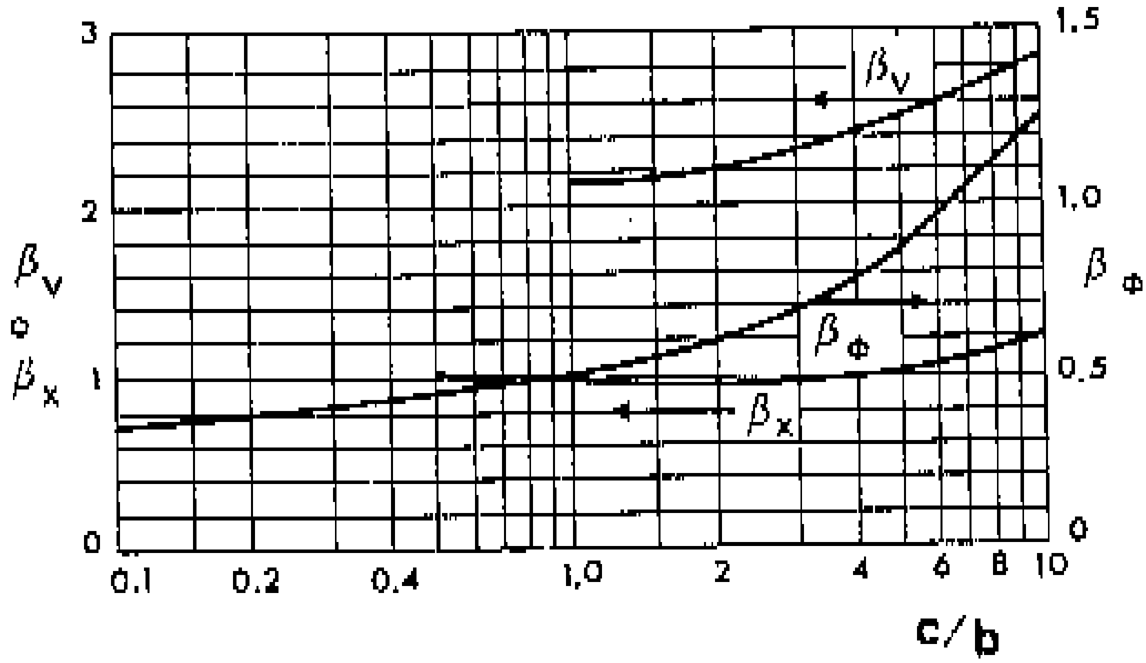


FIG. II.1. Constants for the formulas in Table II.2.

where:

c = length of the foundation in the direction of horizontal excitation
b = width of the foundation perpendicular to the direction of horizontal excitation

II.2.2. Rigid circular foundation on a stratum over a rigid bedrock

The spring constants are given in Table II.3. The damping coefficients should still be evaluated from Table II.2.

TABLE II.3. SPRING CONSTANTS FOR RIGID CIRCULAR FOUNDATION ON A STRATUM OVER A RIGID BEDROCK

Direction of Motion	Equivalent Spring Constant (Static Stiffness)	Range of Validity
Horizontal	$K_H = \frac{8GR}{2-\nu}\left(1+\frac{1}{2}\frac{R}{H}\right)$	$H/R > 1$
Rocking	$K_R = \frac{8GR^3}{3(1-\nu)}\left(1+\frac{1}{6}\frac{R}{H}\right)$	$4 \geq H/R > 1$
Vertical	$K_V = \frac{4GR}{1-\nu}\left(1+1.28\frac{R}{H}\right)$	$H/R > 2$
Torsion	$K_T = \frac{16GR^3}{3}$	$H/R \geq 1.25$

H = thickness of stratum

II.2.3. Rigid circular foundation embedded into a stratum over a rigid bedrock

The spring constants are given in Table II.4. The damping coefficients are calculated from Table II.2.

TABLE II.4. SPRING CONSTANTS FOR EMBEDDED CYLINDRICAL FOUNDATION ON A STRATUM

Direction of Motion	Equivalent Spring Constant (Static Stiffness)	Range of Validity
Horizontal	$K_H = \frac{8GR}{2-\nu}\left(1+\frac{1}{2}\frac{R}{H}\right)\left(1+\frac{2}{3}\frac{D}{R}\right)\left(1+\frac{5}{4}\frac{D}{H}\right)$	$D/R < 2$ $D/H \leq 0.5$
Rocking	$K_R = \frac{8GR^3}{3(1-\nu)}\left(1+\frac{1}{6}\frac{R}{H}\right)\left(1+2\frac{D}{R}\right)\left(1+0.7\frac{D}{H}\right)$	
Vertical	$K_V = \frac{4GR}{1-\nu}\left(1+1.28\frac{R}{H}\right)\left(1+\frac{1}{2}\frac{D}{R}\right)\left(1+\left[0.85-0.28\frac{D}{R}\right]\frac{D/H}{1-D/H}\right)$	
Coupled horizontalrocking	$K_{HR} = 0.40 K_H D$	
Torsion	$K_T = \frac{16GR^3}{3}\left(1+2.67\frac{D}{R}\right)$	

D = depth of embedded foundation

II.3. MODAL DAMPING

For each foundation motion, the reduced radiation damping factor of a soil is calculated from the damping coefficient determined in Section II.2:

$$\xi_{SSI} = 0.5 \frac{C}{2K} \tag{II.8}$$

where, C and K are the damping and the stiffness coefficients. The factor 0.5 is intended to take into account the fact that the actual radiation damping is less than that for a 'regular' half-space, due to waves reflection in horizontal soil layers.

It should be noted that for low frequencies, the damping factor may decrease to approximately zero in case of a foundation on an elasticstratum which may trap the reflected waves. The overall soil damping factor is shown as:

$$\xi_s = \xi_g + \xi_{SSI} \tag{II.9}$$

This value should be limited to 30%

where, ξ_g is the hysteretic soil damping defined in Table II.5.

TABLE II.5. HYSTERETIC SOIL DAMPING.

Soil type according to Table 6 § 5.1.1	1	2	3
ξ_g	3 %	5 %	10 %

The modal damping factor can be calculated by averaging the damping values of each element or subsystem, such as the soil, using the following formula:

$$\xi_i = \frac{\sum_{j=1}^{p} \xi_j E_j^i}{\sum_{j=1}^{p} E_j^i} \tag{II.10}$$

where,

$$E_j^i = \overline{\phi}_i^T \overline{K}^j \overline{\phi}_i \tag{II.11}$$

The structure is divided into p parts, each has a damping factor equals to ζ_j. $\overline{K}^j$ is the stiffness matrix reduced to the j-th part. $\overline{\phi}_i$ is the i-th modal vector, reduced to the j-th part. This value should not exceed 20 %.

REFERENCES

[II.1] KAUSEL, E., et al.., Dynamic analysis of embedded structures: Transactions, 4th SMiRT, SAN –FRANCISCO, Paper K2/6, Vol. 1977.

[II.2] KAUSEL, E., USHIJIMA, R., Vertical and torsional stiffness of cylindrical footings, MIT Research Report R79-6, Dept. of Civil Engineering, Massachusetts Institute of Technology, February 1979.

[II.3] LUCO, J.E., Impedance functions for a rigid foundation on a layered medium, Nuclear Engineering Design, Vol.31, n. 57, 1974.

[II.4] GAZETAS, G., Analysis of machine foundation vibrations: state of the art, Soil Dynamics and Earthquake Engineering, Vol. 2, N° 1., 1983.

[II.5] NEWMARK, M., ROSENBLUETH, E., Fundamentals of earthquake engineering, Prentice-Hall, Inc., Englewood Cliffs, N.J., P. 98, 1971.

[II.6] GAZETAS, G., Foundation vibrations, Foundation engineering handbook, 2nd Edition, H.Y. Fang, ed., Van Rostrand Reinhold, 1991.

Appendix III

EARTHQUAKE RESISTANT DESIGN PROVISIONS

TABLE III.1. EARTHQUAKE RESISTANT DESIGN PROVISIONS

Item	Design Provision
Concrete block partition walls	Properly reinforce walls, dowel to floor or tie into steel work, to avoid collapse in an EQ.
Instrument stands and equipment platforms.	Add cross bracing; brace back to wall if tall. Anchor well to resist EQ forces and overturning moments.
Cranes, hoists, jibs, moving bridges working platforms	Make design provisions for tethering or clamping hoists/cranes in a safe position when out of service. Lower loads onto safe areas when hoisting/handling operations are over (administrative).
Building to building clearance	Provide enough rattle space or use soft caulking.
Safetyrelated equipment close to NSQ equipment or structures.	Increase normal separation. Cage or barricade NSQ equipment. Protect safetyrelated equipment. Secure NSQ structures or equipment to prevent collapse. Add redundant or diverse safetyrelated equipment (well separated). Use fail-safe equipment.
Ladders, handrail, guard rails, stairways, etc.	Secure and lock handrails, ladders, etc. Mount equipment on separate SQ supports
False or suspended ceilings. Loose furniture.	Secure ceilings and furniture close to sensitive equipment. Add curbs and railings around critical control consoles to prevent impact from furniture moving in an EQ.
Drilled-in expansion anchors in lieu of cast-in (where essential)	Qualify by testing. Cast-in anchors suggested. Highstrength anchor bolts preferred (preloaded). Redundant anchors desirable. Avoid grouted-in anchors. Through-wall anchors are best.
Long, vertical pipes supported at top and bottom only.	Use lateral restraints at suitable intervals, to provide for horizontal EQ effects.
Small branch pipe or tubing connections	Motion limits. Good anchorage. Proper flexibility to allow for differential movement in an EQ.

Item	Design Provision
Field-run tubing, small piping and electrical conduits, small valves and fittings	Route carefully or protect well to avoid impact interaction with larger pipes, ducts, etc. during an EQ. Use adequate clamps and supports.
Conventional pipe hangers	Add lateral restraints at suitable intervals. Replace rigid rods by swivel type. Avoid use of threading in the plane of maximum stress.
Tall, overhung valves and valve operators	Add lateral restraints or motionlimiting stops, as necessary, to limit EQinduced stresses.
Overhead ductwork	Strengthen (lock) duct joints. Add end restraints. Use adequate supports. Use backup supports, where consequences of falling in an EQ are serious.
Building wall penetrations	Use adequate clearance around penetrations, sealed with flexible, fireproof 'boots' on the inside; or weld penetrations to embedments on the inside and use soft bedding, on the outside, with flexible terminations or bellows.
Underground building wall penetration	If underground water level is permanently higher than the penetration level, qualify tightness device joints for relative displacement and water pressure
Water, fuel or lubricant lines and storage tanks	Adequate support bracing. use protective curbs and proper drainage, sprinklers, halon or other fireprotection feature to mitigate effect an EQ.
Tank and equipment supports	Add bracing. Double up anchors, with suitable spacing. Tie back to wall where bracing is insufficient or tank is too tall.
Highpressure gas storage bottles.	Secure bottles to storage racks at top and bottom.
Instrument air reservoirs.	Properly support supplyside check valves. Improve anchorage.
Storage batteries	Strengthen battery racks and anchors. Tie batteries to racks (top and bottom). Place batteries closer to floor level.
Radioactive fuel/waste storage, hot cells, ventilated glove boxes, etc.	Secure storage areas. Confine and SQ cooling and shielding water systems. SQ ventilation systems, where necessary for safety

Key: EQ = earthquake; SQ = seismically qualified; NSQ = not seismically qualified

Appendix IV

COMPUTATION OF PULLING FORCE ON ANCHOR DEVICE

A general scheme for the evaluation of the actions at the anchoring of equipment is shown in Fig. IV.1.

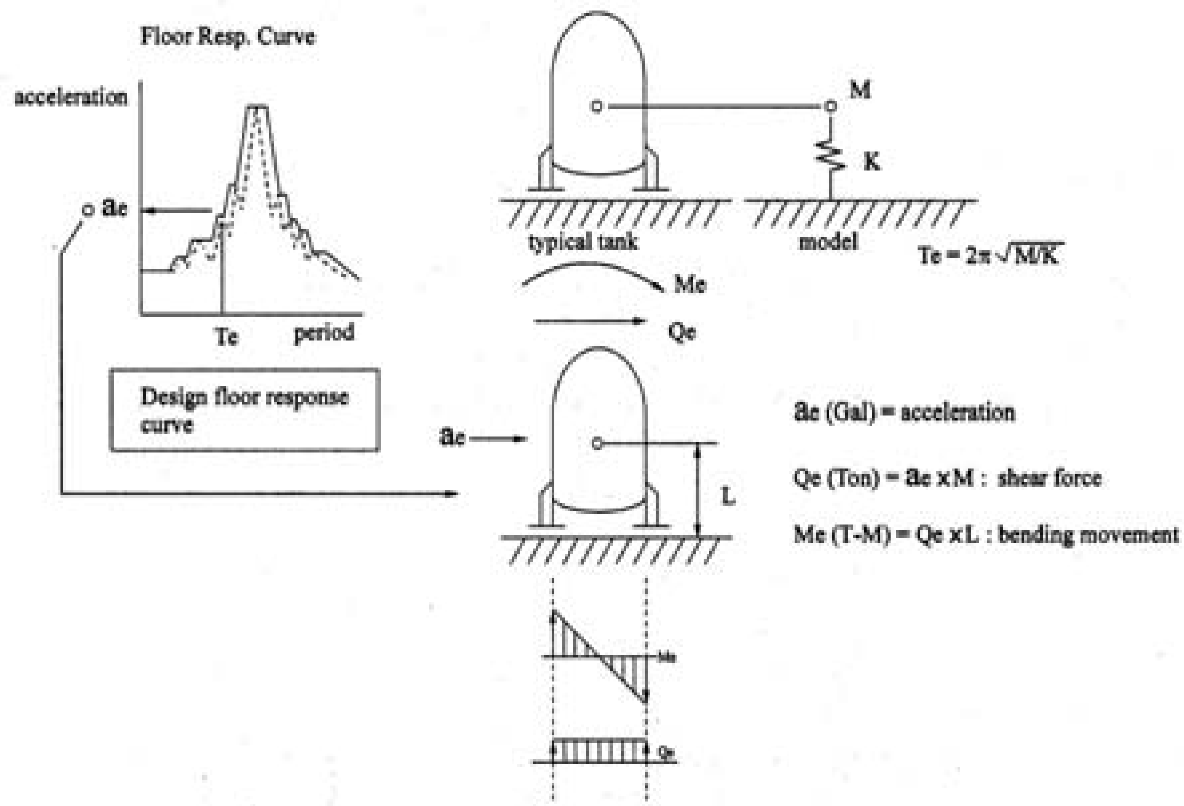


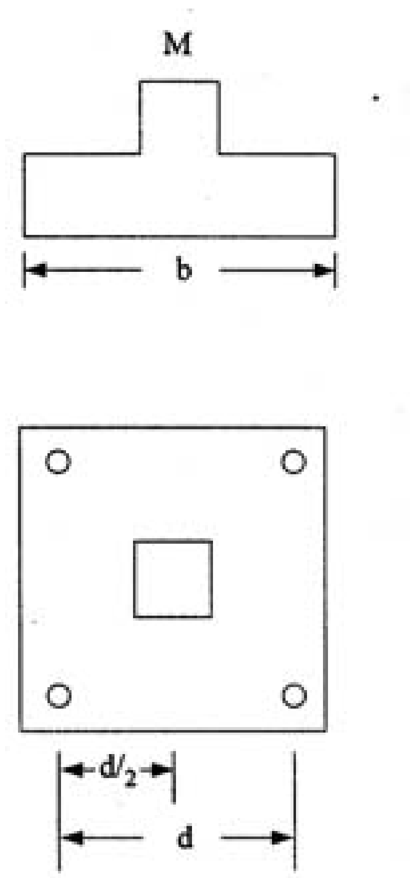
FIG. IV.1. Schematic explanation on calculation of pulling force.

The equilibrium of tensile and compressive forces developed in the anchorage system as well as equilibrating the applied moment to the couple formed by the section tensile force T_{bn} and compressive force need to be considered. The distribution of section tensile force T_{bn} to individual bolts should be a function of tensile strain in the bolt which in turn is a function of an individual bolt's distance from the neutral axis (N.A.).

The following procedures are considered acceptable methods of determining tensile and compressive forces on anchoring devices.

IV.1. SIMPLIFIED CONSERVATIVE METHOD

$$T_b = \frac{M}{dn}$$



where:

M = applied overturning moment
d = distance between bolts
b = width of base or foundation
n = number of bolts
T_b = tensile force per bolt

IV.2. ELASTIC METHOD - SQUARE FOUNDATIONS

$$\sigma_c = (MC/I) + (P/b^2)$$
$$\sigma_t = (MC/I) - (P/b^2)$$

$$x = b\frac{c_t}{\sigma_c + \sigma_t} = b\frac{c_t}{2MC/I}$$

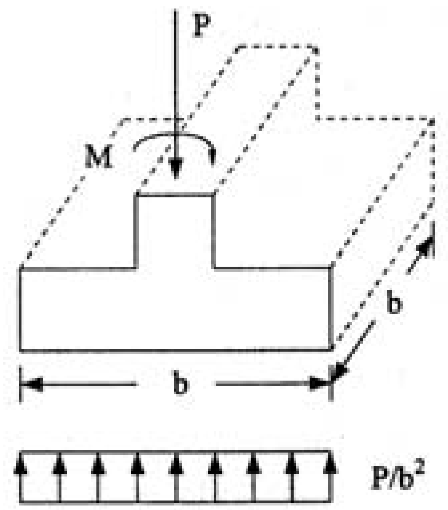


FIG. IV.2. Bearing force of foundation.

then:

$$T_b n = \frac{1}{2}\left(\frac{bc_t}{2MC/I}\right)\sigma_t b$$

where:

C = the distance from neutral axis to the critical side edge

I = moment of inertia of the section.

Allowable

f_c: $0.35 f'_c$

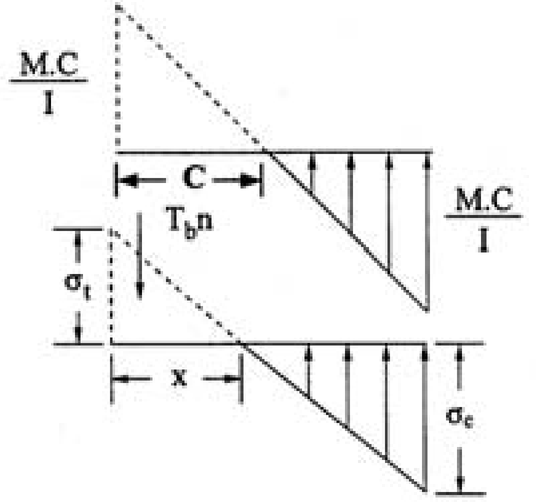


FIG. IV.3. Reaction force calculation for anchor bolts.

IV.3. CONCRETE COLUMN ANALOGY

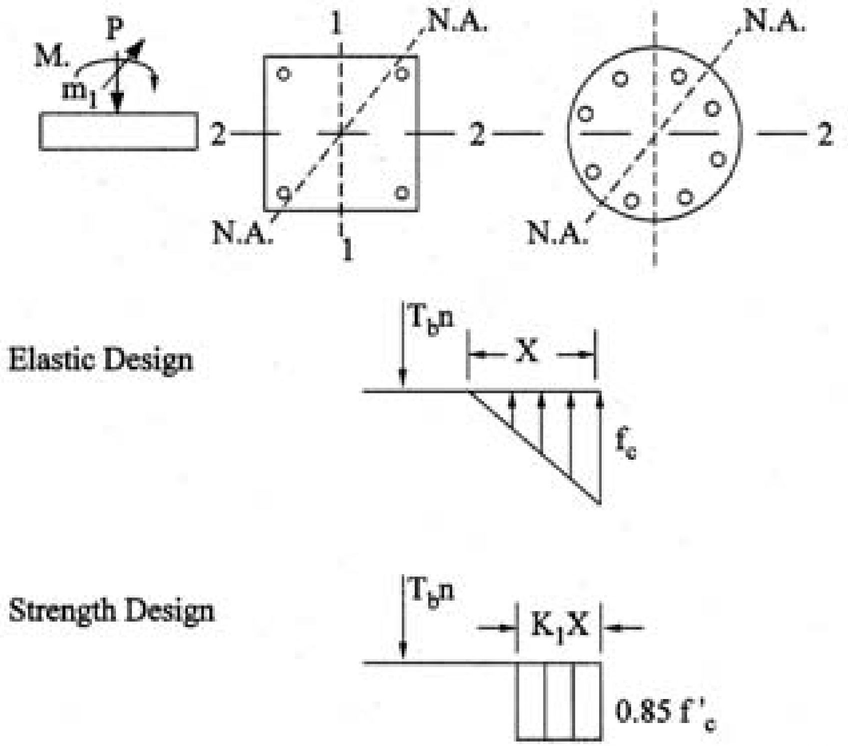


FIG. IV.4. Elastic design and plastic design

Determination of T_bn is based on methods of designing reinforced concrete columns in accordance with national building codes.

IV.4. ISSUES AFFECTING THE PULLING FORCE ON ANCHOR

The geometry of the pits of the bolts can strongly affect the allowed pulling force beard by an anchor bolt, In the following, some simple rules are collected for an easy and effective design of anchoring devices [IV.1].

IV.4.1. Pit placement standard

The pit placement standard of anchor bolts is shown in Table IV.1 and Fig. IV.5 which depends upon the type of anchor bolt used.

IV.4.2. Reduced pit placement

When distances between the pits are shorter than the placement distance in the pit placement standard, the allowable pulling out load for a single anchor bolt should be reduced.

The allowable load for pulling out in this case is to be the value determined multiplying the value obtained from the type of used bolts by the reduction ratio shown in Table IV.2. (i) to (iii) (refer to Figs IV.5–IV.6).

TABLE IV.1. PIT PLACEMENT STANDARD

Anchor bolt type	Pit placement standard
Embedded L and LAtype anchor bolts, chemical expansion anchor bolts	At or greater than 10d d: Anchor bolt's nominal diameter
Embedded J and JAtype bolts, headattached bolts, and external thread type mechanical expansion anchor bolts	At or greater than 2L L: Anchor bolt embedded length
Box-out anchor bolts	Placement distance between boxes (A) that is at or greater than 10cm

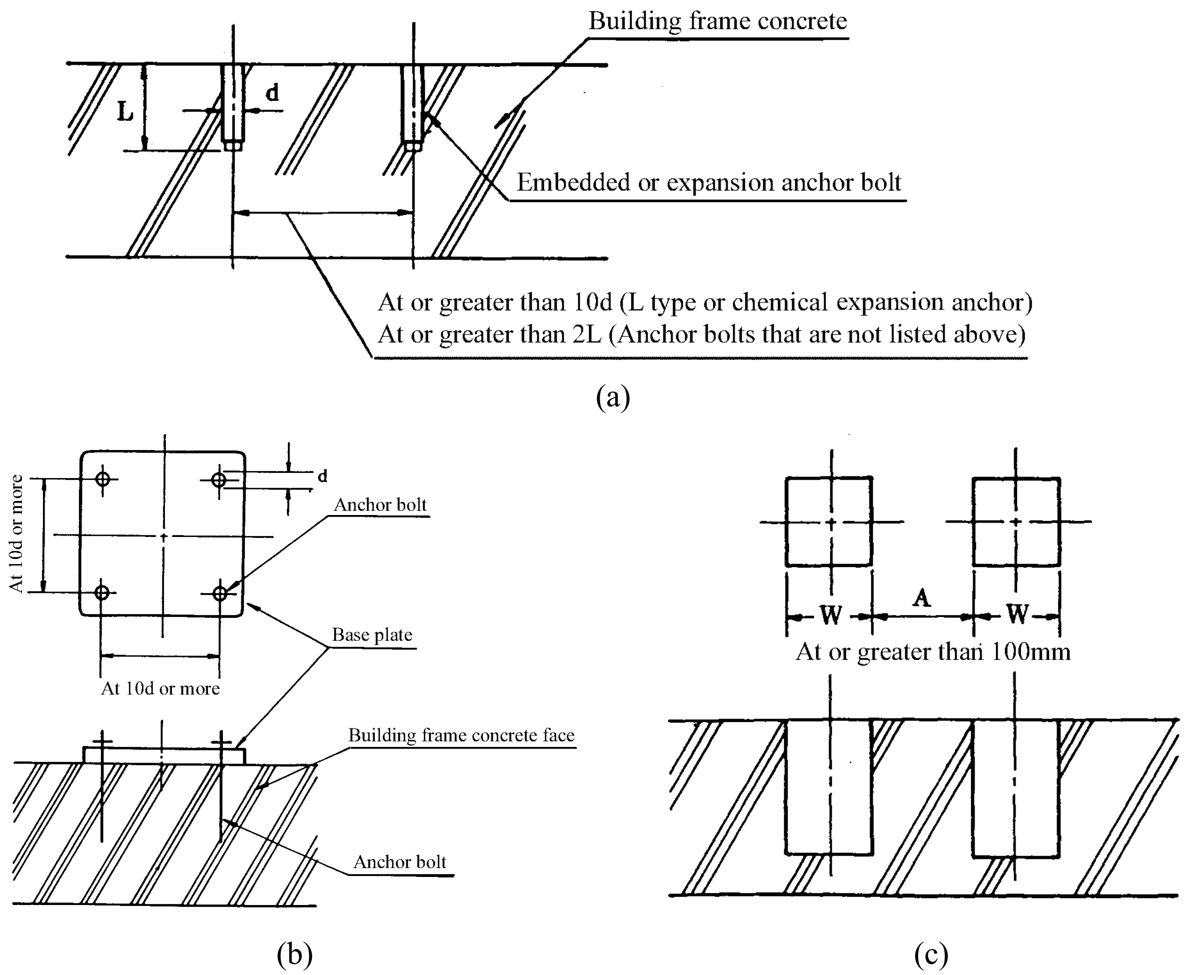


FIG. IV.5. Pit placement standard

TABLE IV.2. (i) Reduction ratio of the allowable load for pulling out according to the placement distance of embedded L and LAtype anchor bolts and chemical expansion anchor bolts.

Number of anchor bolts	Reduction ratio (η)
2 bolts	$\frac{1}{100}\left(2\cdot\frac{P}{d}+80\right)$
3 or 4 bolts	$\frac{1}{100}\left(6\cdot\frac{P}{d}+40\right)$

(Note) 1. P: Placement distance of the anchor bolt
d: Nominal diameter of the anchor bolt
2. It is to be 10d>P>5d.

TABLE IV.2. (ii)Reduction ratio of the allowable load for pulling out according to the placement distance of embedded J and JAtype anchor bolts, headattached bolts and externalthreadtype mechanical expansion anchor bolts.

Number of anchor bolts	Reduction ratio (η)
2 bolts	$\frac{1}{10}\left(2.5\cdot\frac{P}{L}+5\right)$
3 or 4 bolts	$\frac{1}{10}\left(5\cdot\frac{P}{L}\right)$

(Note) 1. P: Placement distance of the anchor bolt
L: Embedded length of the anchor bolt
2. It is to be 2L>P>L.

TABLE IV.2. (iii) Reduction ratio of the allowable load for pulling out according to the distance between boxes of box-out anchors.

<table>
<tr><th>Number of anchor bolts</th><th>Reduction ratio (η)</th></tr>
<tr><td>2 bolts</td><td rowspan="2">$\frac{A}{10}$</td></tr>
<tr><td>4 bolts</td></tr>
</table>

(Note) 1. A: Distance between boxes of box-out anchors (cm)
2. It is to be 10cm>A>5cm.

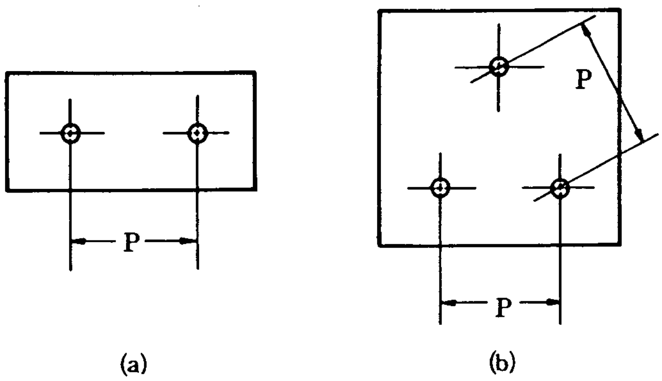


FIG. IV.6. Reduced pit placement.

The allowable shear force for anchor bolts that have been placed at the corner or at the side of the foundation is described in Fig. IV.7.

Placing location: A firm foundation (corner, side)

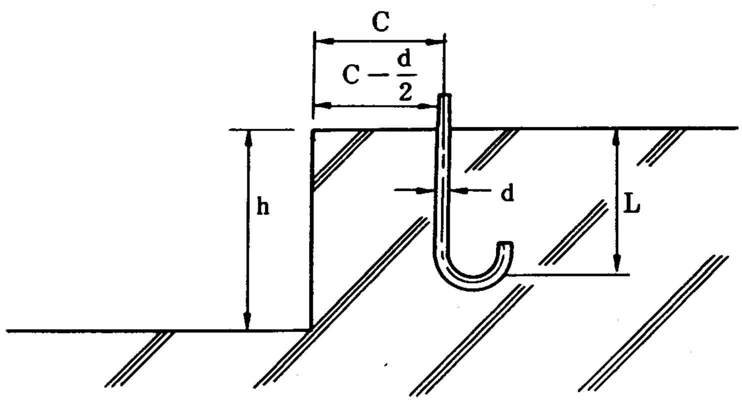


FIG. IV.7. Shear force for anchor bolts that have been placed in the corner or side of foundations.

Determine the bolt's allowable shear force Qa with the equation that appears below, and take the smaller value.

$$Qa=\frac{\pi}{4}d^2fs$$

$$Qa=3\pi C\,(C+d)p$$

Where d: The anchor bolt's nominal diameter (cm)

fs: The allowable shear stress of anchor bolts in pure shear mode

C: The distance from the centre of the anchor bolt to the foundation's side (cm)

$$(C-\frac{d}{2}>5cm)$$

p: The correction factor based on the specified design strength of concrete

$$p=\frac{1}{6}Min\left(\frac{Fc}{30},\ 5+\frac{Fc}{100}\right)$$

Fc: The specified design strength of concrete (daN/cm^2)

Note 1. It is suggested to be L>6d. (d: anchor bolt nominal diameter)

2. It is to be h>C. Calculations are to be made with the (3.20) equation if h>C.

IV.4.3. Examples

Fig. IV.8 shows some examples of anchor bolt types. The allowable load for pulling out of these anchor bolts needs to be higher than that of the allowable load for pulling out embedded headattached bolts.

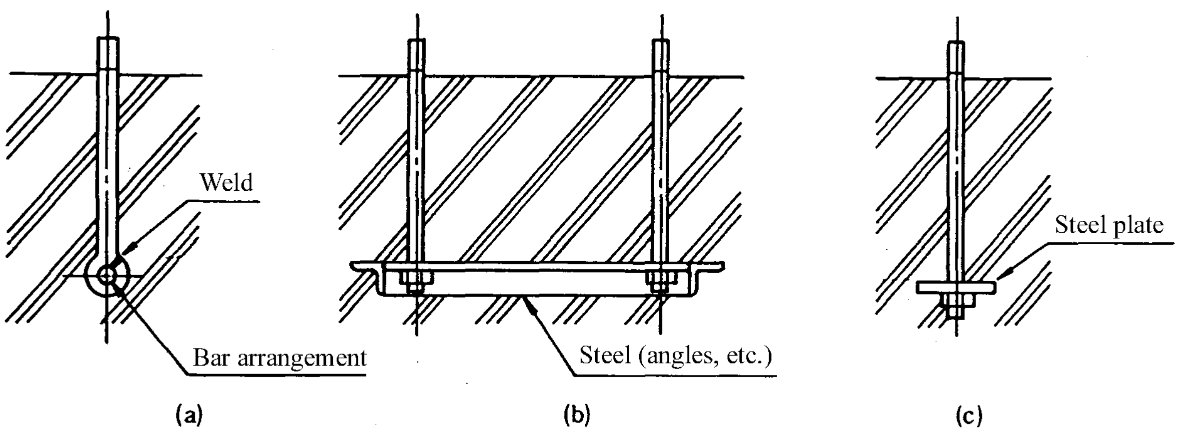


FIG. IV.8. Example of effective anchor bolts.

IV.4.4. Capacities

a) Ltype bolts

The strength of Ltype bolts is determined on the basis of its bond strength. Due to the fact that a weakening of bond strength can be expected over many years of use in locations that are exposed to vibrations, it is suggested to be careful to use safer values.

b) When hanging heavy structure on ceiling slabs or walls, it is necessary to examine not only the strength of anchor bolts, but it is also suggested to check the slab bearing capacity

REFERENCE

[IV.1] THE JAPANESE ENGINE GENERATOR ASSOCIATION, Guideline Of Seismic Design, Tokyo, 1999.

Appendix V

SIMPLE SEISMIC DESIGN PRACTICE FOR PIPING SYSTEMS — THE CONSTANT PITCH DESIGN METHODOLOGY

V.1. OUTLINE OF SIMPLIFIED SEISMIC EVALUATION PRACTICE OF PIPING

Usually, seismic evaluation of piping is carried out with sophisticated methodologies. However, when a simplified seismic evaluation is needed, it can be easily carried out through approximate methods based upon the control of the span length between piping supports and the capacity to absorb relative displacements [V.1-V.3]. This method is called the constant pitch design method.

V.2. GENERALITIES ON THE SIMPLIFIED SEISMIC EVALUATION FOR PIPING

A procedure for a simplified seismic evaluation for piping is as follows:

(1) Seismic capacity of piping is evaluated by the span length between piping supports.

(2) Evaluation of allowable span is carried out:
   - for the maximum span length of piping
   - for the span length of piping with concentrated mass.

(3) Evaluation of the capacity of accommodating relative displacements is performed based on the span length between both ends supported by different structures, or on the span length to the first support of tee with a branch, whose diameter is smaller than the half of diameter of the main piping.

(4) Steps (2) and (3) are performed for each direction, two horizontal and vertical.

(5) Evaluation on capacity of absorbing displacements associated with weight and thermal effects is out of the scope of this evaluation. In the case of piping containing bellows, capacity of absorbing displacement on bellows should be evaluated.

The flow chart of simplified seismic evaluation for piping is shown in Fig. V.1.

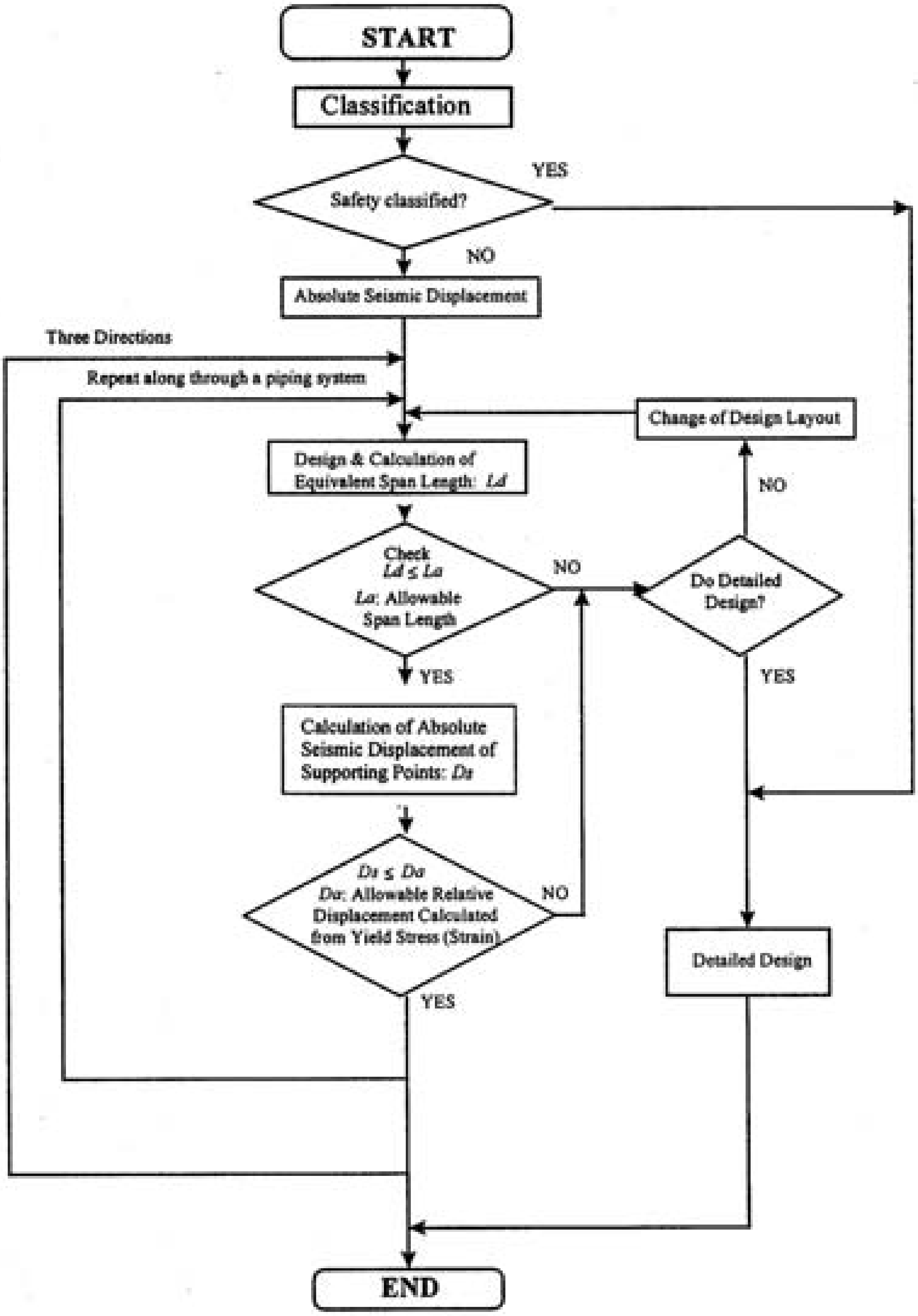


FIG. V.1. Flow of simplified piping design [V.4].

V.3. CALCULATION OF ALLOWABLE SPAN LENGTH

V.3.1. Criteria for constant pitch design method

$$L \leq La$$

Where, L: Span Length, La: Allowable Span Length.

However, this span length needs to be checked against relative displacements.

V.3.2. Allowable span length

Allowable span length is calculated according to the following rules. The allowable span length, without considering an additional mass and a concentrated mass, is called basic allowable span length.

(1) Basic allowable span length values are shown in Table V.1 and V.2. Numerical diameters in these tables are based on some typical standards. For other sizes, basic allowable span length can be corrected linearly according to the real size. However, basic allowable span length of diameter under 48.6 mm is the value for diameter: 48.6 mm, and basic allowable span length of diameter over 609.6 mm is the value for diameter: 609.6 mm. The applicable range of these tables is for piping less than 1000 mm diameter.

(2) In the case of an additional mass or a concentrated mass, the basic allowable span length should be multiplied by Fn_d and Fn_c to the basic value according to eq. (V.1) and eq. (V.2) respectively.

(3) In the case of an additional distributed mass, compensation coefficient Fn_d is calculated by the following equation.

$$Fn_d = \left(1 + \frac{\Gamma}{\Gamma_p}\right)^{-0.25} \qquad \text{(V.1)}$$

or the following value may be taken

Fn_d=1.0 at $\Gamma/\Gamma p \leq 10.5$,

where, Fn_d : compensation coefficient for additional weight
Γp : total weight of both weight of piping and weight of fluid in piping per unit length.(N/m)
Γ: additional weight per unit length.(N/m)

(4) In the case of a concentrated weight such a valve, the compensation coefficient Fn_c is obtained by Fig. V.2 or Table V.3. Fig. V.2 shows compensation

coefficient for an concentrated weight F_c, and rate of additional weight: r_w. Rate of additional weight: r_w is calculated by the following equation. However Fc=1.0 in the case $r_w \leq 10.25$.

$$r_w = \frac{w}{Wa}\left(1+\frac{\Gamma}{\Gamma p}\right)^{-0.75} \tag{V.2}$$

where, r_w : rate of additional weight
w : concentrated weight on the span of piping
Wa : basic concentrated weight (referred to Table V.1 or Table V.2)

TABLE V.1. BASIC ALLOWABLE SPAN LENGTH (ρ=0.5)

Nominal Diameter (in)	(mm)	Diameter (mm)	Basic Allowable Span Length: La (m)	Basic Concentrated Weight: *Wa*(N)
1½	40	48.6	6.6	407
2	50	60.5	7.1	605
2½	65	76.3	7.9	1,116
3	80	89.1	8.6	1,545
3½	90	101.6	9.0	1,986
4	100	114.3	9.5	2,532
5	125	139.8	10.2	3,802
6	150	165.2	10.8	5,357
8	200	216.3	12.2	9,629
10	250	267.4	13.2	15,208
12	300	318.5	14.2	22,361
14	350	355.6	15.0	28,851
16	400	406.4	16.0	40,325
18	450	457.2	16.8	53,612
20	500	508.0	17.6	67,633
22	550	558.8	18.4	83,563
24	600	609.6	19.1	103,946

ρ: Density of fluid: ρ = 1.0 for water

TABLE V.2. BASIC ALLOWABLE SPAN LENGTH (GAS)

Nominal Diameter (in)	(mm)	Diameter (mm)	Basic Allowable Span Length: La (m)	Basic Concentrated Weight: *Wa*(N)
1½	40	48.6	7.0	304
2	50	60.5	7.8	445
2½	65	76.3	8.7	839
3	80	89.1	9.5	1,126
3½	90	101.6	10.1	1,414
4	100	114.3	10.7	1,775
5	125	139.8	11.7	2,616
6	150	165.2	12.7	3,616
8	200	216.3	14.8	6,349

10	250	267.4	16.4	9,863
12	300	318.5	18.0	14,281
14	350	355.6	19.0	18,110
16	400	406.4	20.3	25,339
18	450	457.2	21.5	33,995
20	500	508.0	22.7	41,112
22	550	558.8	23.8	51,143
24	600	609.6	24.9	64,243

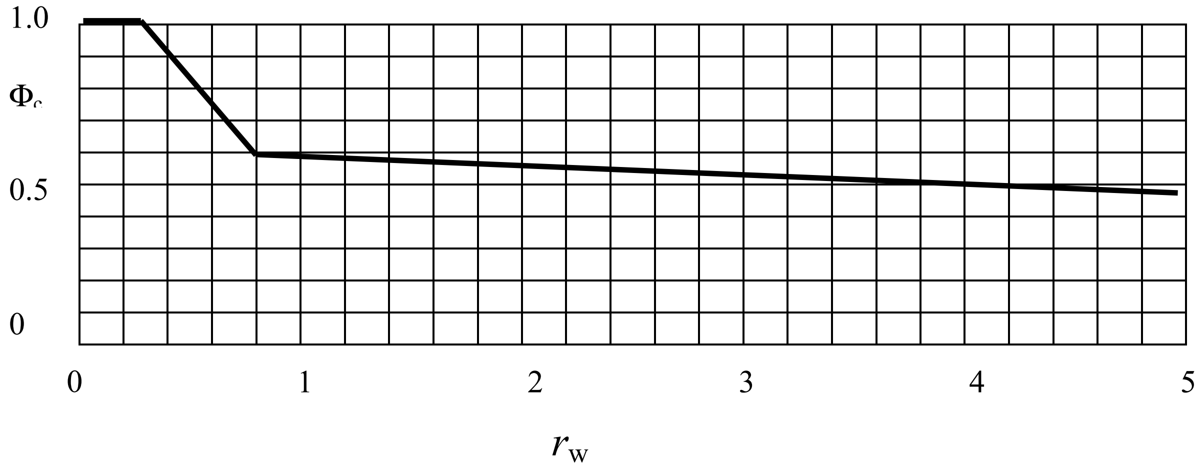


FIG. V.2. Compensation coefficient for concentrated weight.

TABLE V.3. COMPENSATION COEFFICIENT FOR CONCENTRATED MASS

Range of r_w	Compensational coefficient for concentrated weight: Fn_c
$r_w \leq 0.25$	$\Phi c = 1.0$
$0.25 < r_w \leq .1$	$\Phi c = 1.13–0.53\ r_w$
$1 < r_w$	$\Phi c = 0.636–0.036\ r_w$

V.4. EQUIVALENT SPAN LENGTH

Equivalent span length of othertypes of piping than the straight one is calculated according to the following rules:

(1) Equivalent span length of 3-dimensional one (two horizontal directions and one vertical direction) should be evaluated for 3 directions of the seismic motions.
(2) Equivalent span length is calculated along an axis of piping between the nearest piping supports.
(3) When an axis of piping is parallel to a direction of seismic motion, equivalent span length can be calculated without the span of the first leg of the system, that is, L_1 in Fig. V.3.

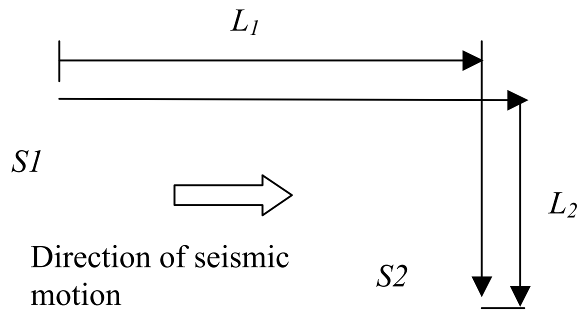


FIG. V.3. Equivalent span length for Lshape.

In Fig. V.3, when piping supports *S1* and *S2* support for direction of seismic motion, equivalent span length is calculated without considering span length L_1, where L is along the axis of the first leg parallel to the direction of seismic motion, then $L = L_2$.

(4) When there are different diameters of piping in evaluated piping systems, the equivalent span length is calculated by the following equation.

$$L = l + l_1\sqrt{\frac{d}{d_1}} \tag{V.3}$$

where,

L: equivalent span length (m)
d: the maximum diameter of evaluated pipings
d_1: diameter of specified piping
l: allowable span length for diameter d and
l_l: allowable span length for diameter d_1.

(5) In case of piping with tee as shown in Fig. V.4, the following is applied:

(a)When a diameter of a branched leg is larger than one half diameter of the main piping, ($L_1 + L_2$), ($L_1 + L_b$) and ($L_2 + L_b$) should be shorter than the allowable span length. In Figure V.4, piping supports, *S1*, *S2* and *Sb* provide support to directions perpendicular to axes of each piping. Moreover, L_1, L_2 and L_b are calculated according to items (1) to (4).

a) (b)When a diameter of a branched leg is smaller than one half diameter of the main piping, both ($L_1 + L_2$) and L_b should be shorter than the allowable span length.

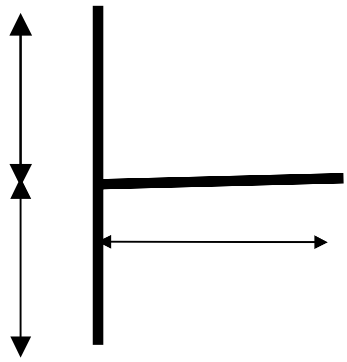


FIG. V.4. Equivalent span length for teeshape piping.

V.5. CALCULATION OF ALLOWABLE RELATIVE DISPLACEMENT OF SUPPORTS

V.5.1. Criteria for allowable displacement

In case both ends are supported by different supporting structures including piping with tee, evaluation of allowable displacement needs to be performed. Also, in the case of piping with tee, evaluation needs to take into account the effect from a branch point to the first support, where its diameter is smaller than one half of diameter of the main piping.

Evaluation of allowable displacement is performed by the following equation.

$$Ds \leq Da \qquad \text{(V.4)}$$

where,

- *Ds*: Relative displacement between supports or between a branch and the first support in the direction of seismic motion
- *Da*: Allowable displacement in the direction of seismic motion, which is calculated by Eq. (E-5)

V.5.2. Allowable displacement of piping

Allowable displacement of piping is calculated by the following equation.

$$Da = Lj.f \qquad \text{(V.5)}$$

where

- *Da*: Allowable displacement in the earthquake direction

Lj: Projective span length (in mm) in a plane perpendicular to the seismic motion and that can be calculated according to the method described in chapter 4.4

f: Allowable displacement per unit length (mm), calculated by the following equation

$$f = \frac{C \cdot \varepsilon y \cdot Lj}{D} \tag{V.6}$$

where,

C: Coefficient is defined by allowable strain of piping material, and the value can be used up to yield strain; *C*=0.67,

ε: Yield strain of piping material is calculated by the following equation.

$$\varepsilon = \frac{Sy}{E} \tag{V.7}$$

where,

S*y*: Yield stress or 0.2% strength of material at the design temperature
E: Young's modulus of material at the design temperature
D: Diameter of piping.

V.6. PROJECTIVE SPAN LENGTH

Projective span length is calculated according to the following fundamental rules:

(1) Projective span length of two horizontal directions is evaluated to two seismic motions.
(2) Projective span length: *Lj* is span length along an axis of piping between the adjacent supports in plane which is perpendicular to seismic direction.
(3) When they have parts of different diameters in evaluated piping, projective span length is calculated by the following equation.

$$Lj = l + l_1\sqrt{\frac{d}{d_1}} \tag{V.8}$$

where,

*L*j: projective span length(m)
d: the maximum diameter of evaluated piping
d_1: diameter of specified piping
l: projective span length for diameter : *d* (m)

l_l: projective span length for diameter : d_l(m)

(4) In case of piping with tee as shown in Fig. V.4,

(a) When the diameter of a branched leg is larger than one half of diameter of the main piping, evaluation of allowable displacement needs to meet the following conditions:

$$Ds(12) \leq Da(12), Ds(1b) \leq Da(1b), and Ds(2b) \leq Da(2b)$$

where,

Lj(12): projective span length of (L_1+L_2)
Lj(1b): projective span length of (L_1+L_b)
Lj(2b): projective span length of (L_2+L_b)
Da(12): allowable displacement of (L_1+L_2)
Da(1b): allowable displacement of (L_1+L_b)
$D\ a$(2b): allowable displacement of (L_2+L_b)
Ds(12): relative displacement between *S1* and *S2*
Ds(1b): relative displacement between *S1* and *Sb*
Ds(2b): relative displacement between *S2* and *Sb*

In Fig. V.4, piping supports, *S1*, *S2* and *Sb* support for perpendicular directions to the axes of piping. Moreover, L_1, L_2 and L_b are calculated according to items (1) to (4).

(b) When the diameter of a branched leg is smaller than one half of diameter of the main piping, evaluation of allowable displacement needs to meet the following conditions,

$$Ds(12) \leq Da(12)$$
$$and\left(\frac{Ds(1b)+Ds(2b)}{2+20\left(\frac{L_{12}}{L}\right)}\right)^4 \leq Da(Tb) \qquad \text{(V.9)}$$

where,

Da(Tb): allowable displacement of the third leg of tee
L: allowable span length of the main piping
L_{12}: span length of the main piping.

V.7. CALCULATION OF RELATIVE DISPLACEMENT

The displacement of support to be used for an evaluation against seismic motion is the relative displacement between two supporting structures of piping.

Relative displacement of evaluated span length: Ds is calculated by the following equation.

$$Ds = D_1 + D_2 \tag{V.10}$$

where,

Ds:	relative displacement of evaluated span length
D_1:	displacement of support 1 under seismic event
D_2:	displacement of support 2 under seismic event

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[V.2] SHIBATA, H., On anti-earthquake design procedure of equipment and pipings in near future, in: Proc. Sixth Int. Conf. Structural Mechanics in Reactor Technology, Paris, (1981), paper K 13/1.

[V.3] SHIBATA, H., On a method of evaluation of anti-earthquake design code of industrial facilities, Proc. Seventh World Conf. on Earthquake Engineering, Istanbul, Turkey (1980), paper 4/613.

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Appendix VI

DETAILS OF CONNECTING CABLE AND PIPING BETWEEN COMPONENTS

Some suggested good practices detailing connecting cable and piping between components are given in Figures VI.1.–VI.3.

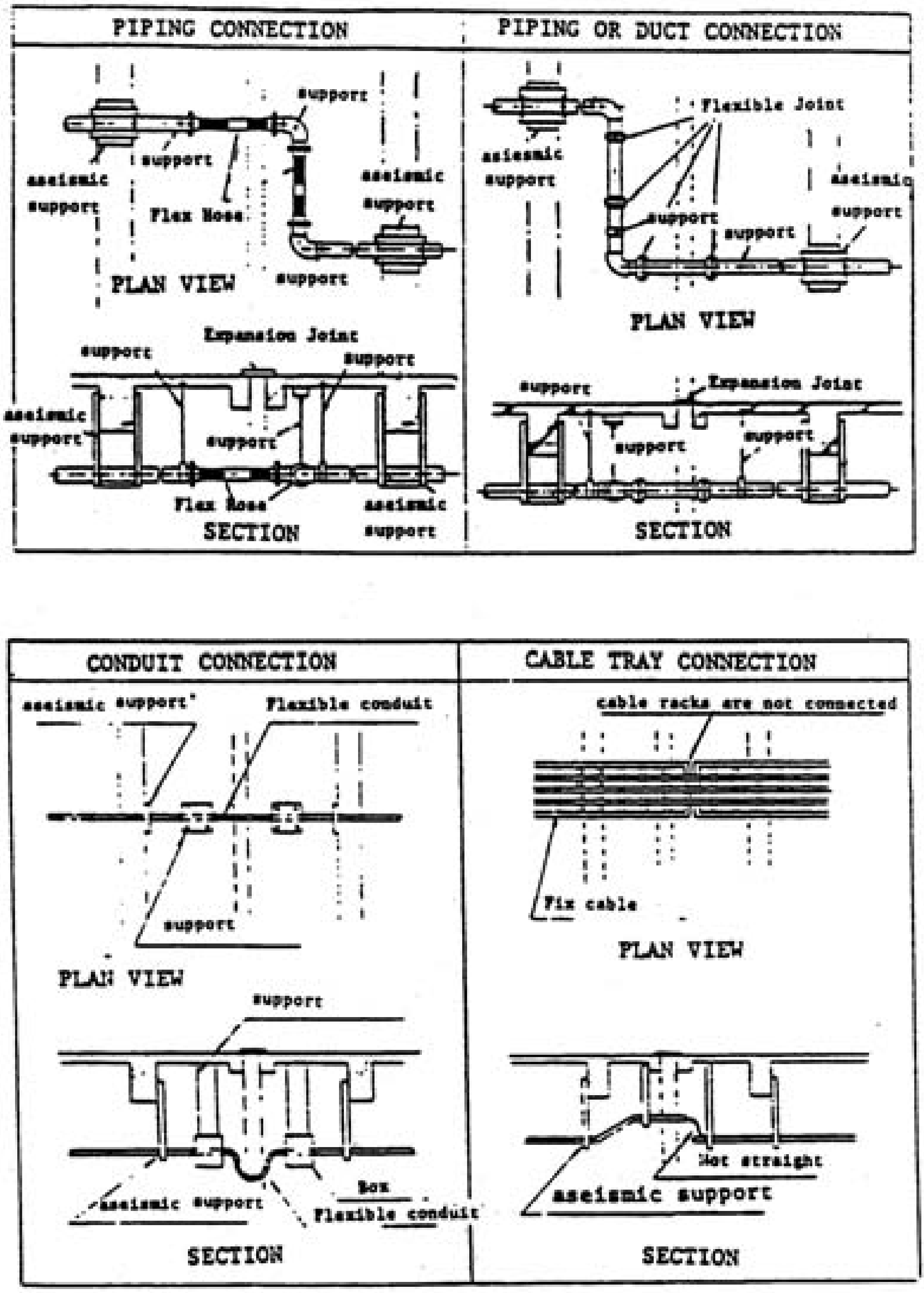
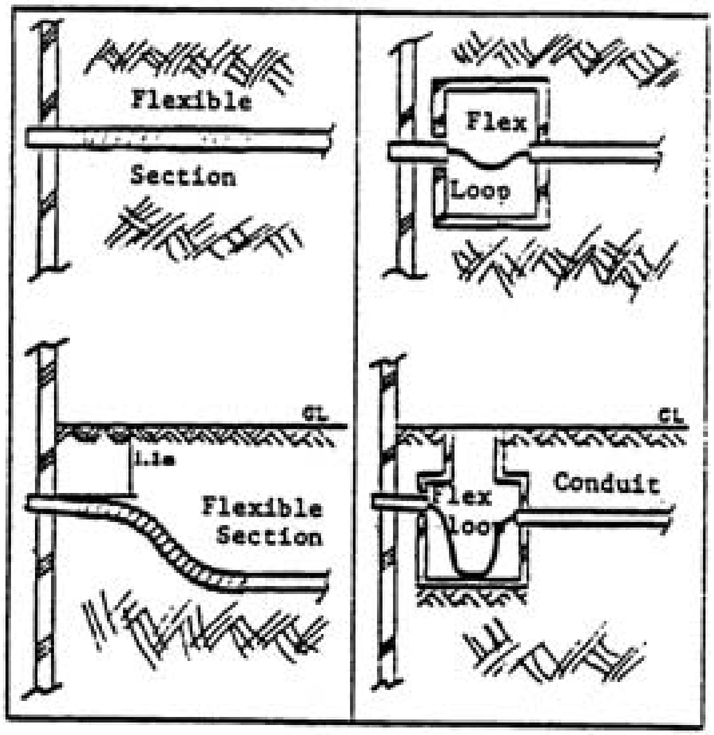
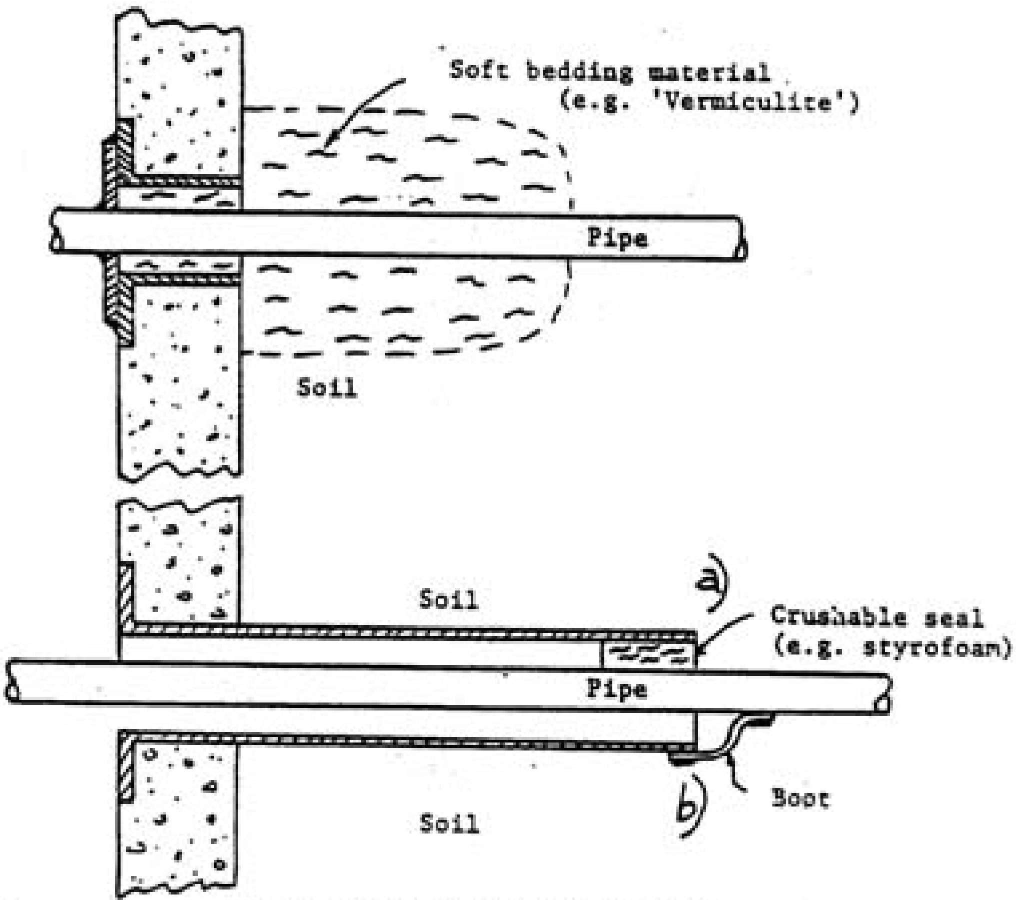


FIG. VI.1. Connection between buildings.



Underground Cable Penetrations



Underground Piping Connections

FIG. VI.2. Connections for buried cables and pipes.

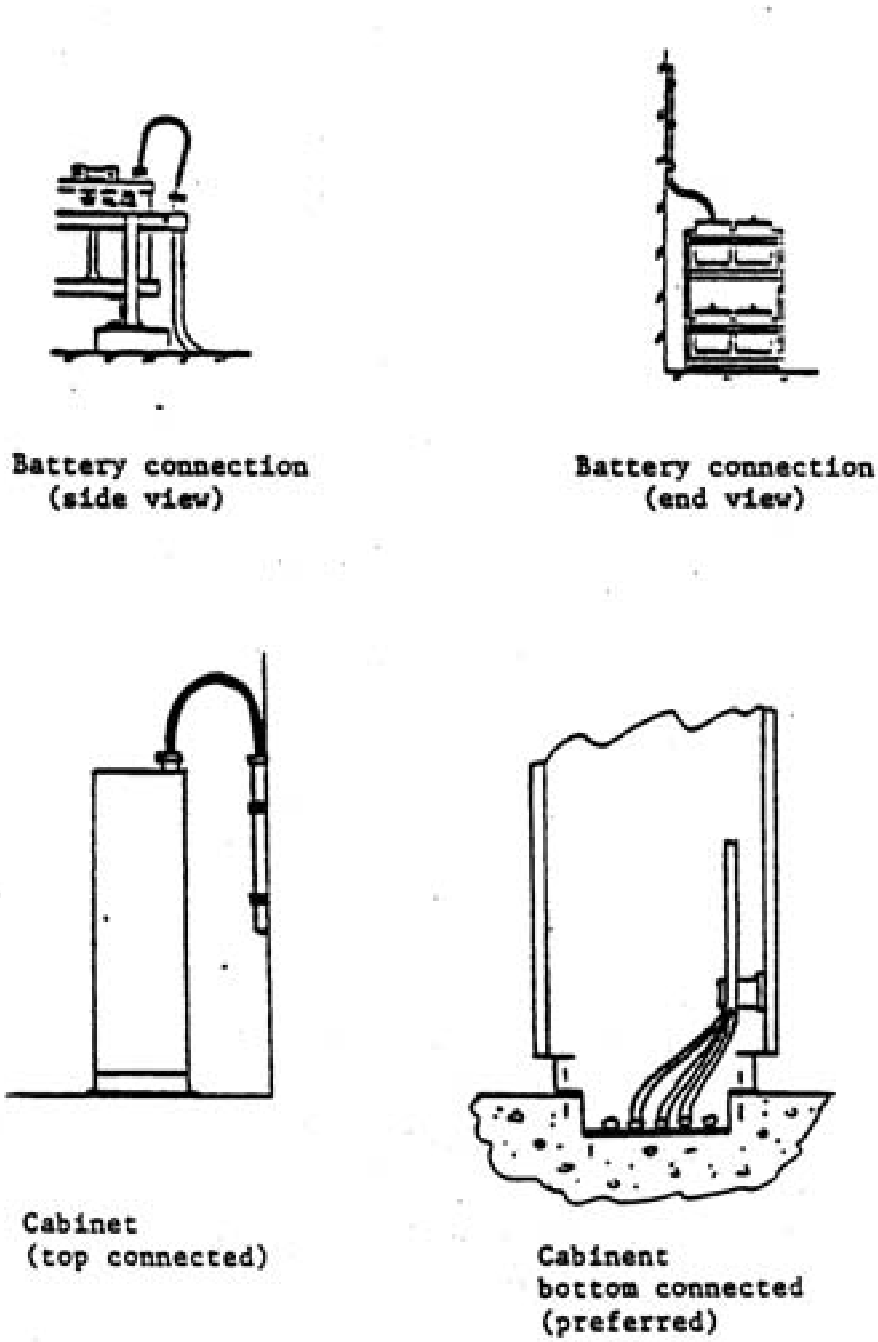


FIG. VI.3. Connections to equipment.

Appendix VII

SEISMIC SCRAM SYSTEMS

VII.1. ISSUES AFFECTING THE DECISION TO INSTALL AUTOMATIC SEISMIC TRIP SYSTEM (ASTS) IN A POWER PLANT

The following issues play a significant role in the decision for an installation of an automatic seismic trip system (ASTS) in an NPP:

- level, frequency and duration of earthquake activity at the facility site;
- seismic capacity of facilities structures, equipment and distribution systems;
- safety considerations related to spurious scrams;
- expected time of seismic scram and comparison of this time to the expected time for reaching the strong motion part of the earthquake time history;
- economic impact of shutting down the facility to the country; or to the research.
- issues related to public acceptance of the seismic safety of the facility;
- level of operator confidence and reliability.

It is suggested to take these issues into consideration prior to deciding the installation of an ASTS.

VII.2. ELEMENTS FOR THE SETTING OF THE TRIGGER LEVEL FOR ASTS

Two trigger levels are generally used for ASTS. However for research facilities lower levels may be selected.

The first level is chosen to be close to the SL-1 level, as defined in [VII.1]. Significant structures, equipment and distribution systems of the NPP are generally expected not to fail at levels lower than the SL-1 earthquake.

The second level may be close to the SL-2 level [VII.1]. At this level, it may be possible to have the failure of some parts of the core internals and control rod insertion mechanism within a few seconds after the seismic scram signal. In cases where such problems are encountered, alternate ways of scramming the reactor may be considered.

It needs to be noted that earthquake causing the triggering of the second level settings would have the potential for large scale destruction in the site vicinity, including loss of offsite power and disruption in the normal cooling water supply.

Settings lower than SL-1 may be selected for interim periods in cases where seismic capacity assessment and upgrading works for the NPP are ongoing.

Considering the inherent seismic resistance of facility structures, equipment and distribution systems, lower bound setting level of seismic scram would be suggested.

VII.3. ELEMENTS TO BE CONSIDERED IN THE DESIGN OF ASTS

ASTS sensors and associated circuitry need to be designed to withstand the seismic excitation and to prevent any malfunctioning induced by seismic loads.

Typically free field and foundation level are chosen for locating the sensors.

The ASTS needs to be considered as an element in the safety analysis of the facility. Accordingly, the system needs to be treated as part of the safety protection system of the plant and needs to conform to all relevant requirements.

VII.4. GUIDELINES FOR NUCLEAR PLANT RESPONSE TO AN EARTHQUAKE

Plant response to an earthquake needs to be managed regardless of the decision to install an ASTS. The control room operator needs to record the occurrence of an earthquake by any information source and needs to follow the plant response monitoring the seismic instrumentation.

Later, an evaluation of recorded earthquake motion will need to be compared with the design basis of structures, systems and components, a walk-down evaluation of the damage experienced at the facility will need to be carried out and an evaluation of the readiness of the plant to resume operation following the earthquake occurrence will need to be completed.

In relation to this concern, specific post-event procedures need to be developed, particularly for the identification of the items to be inspected, for the evaluation of the damage related quantities and for the involvement of the regulatory body in the decision for resuming operation [VII.2–VII.5].

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Consultants Meetings

Vienna, Austria: 9–13 November 1998
Vienna, Austria: 1–5 March 1999
Vienna, Austria: 10–14 July 2000