IAEA Safety Standards

for protecting people and the environment

Geotechnical Aspects of Site Evaluation and Foundations for Nuclear Power Plants

Safety Guide

No. NS-G-3.6

IAEA

International Atomic Energy Agency

IAEA SAFETY RELATED PUBLICATIONS

IAEA SAFETY STANDARDS

Under the terms of Article III of its Statute, the IAEA is authorized to establish or adopt standards of safety for protection of health and minimization of danger to life and property, and to provide for the application of these standards.

The publications by means of which the IAEA establishes standards are issued in the **IAEA Safety Standards Series**. This series covers nuclear safety, radiation safety, transport safety and waste safety, and also general safety (i.e. all these areas of safety). The publication categories in the series are **Safety Fundamentals, Safety Requirements** and **Safety Guides**.

Safety standards are coded according to their coverage: nuclear safety (NS), radiation safety (RS), transport safety (TS), waste safety (WS) and general safety (GS).

Information on the IAEA's safety standards programme is available at the IAEA Internet site

http://www-ns.iaea.org/standards/

The site provides the texts in English of published and draft safety standards. The texts of safety standards issued in Arabic, Chinese, French, Russian and Spanish, the IAEA Safety Glossary and a status report for safety standards under development are also available. For further information, please contact the IAEA at P.O. Box 100, A-1400 Vienna, Austria.

All users of IAEA safety standards are invited to inform the IAEA of experience in their use (e.g. as a basis for national regulations, for safety reviews and for training courses) for the purpose of ensuring that they continue to meet users' needs. Information may be provided via the IAEA Internet site or by post, as above, or by e-mail to Official.Mail@iaea.org.

OTHER SAFETY RELATED PUBLICATIONS

The IAEA provides for the application of the standards and, under the terms of Articles III and VIII.C of its Statute, makes available and fosters the exchange of information relating to peaceful nuclear activities and serves as an intermediary among its Member States for this purpose.

Reports on safety and protection in nuclear activities are issued in other publications series, in particular the **Safety Reports Series**. Safety Reports provide practical examples and detailed methods that can be used in support of the safety standards. Other IAEA series of safety related publications are the **Provision for the Application of Safety Standards Series**, the **Radiological Assessment Reports Series** and the International Nuclear Safety Group's **INSAG Series**. The IAEA also issues reports on radiological accidents and other special publications.

Safety related publications are also issued in the **Technical Reports Series**, the **IAEA-TECDOC Series**, the **Training Course Series** and the **IAEA Services Series**, and as **Practical Radiation Safety Manuals** and **Practical Radiation Technical Manuals**. Security related publications are issued in the **IAEA Nuclear Security Series**.

GEOTECHNICAL ASPECTS OF SITE EVALUATION AND FOUNDATIONS FOR NUCLEAR POWER PLANTS

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The Agency's Statute was approved on 23 October 1956 by the Conference on the Statute of the IAEA held at United Nations Headquarters, New York; it entered into force on 29 July 1957. The Headquarters of the Agency are situated in Vienna. Its principal objective is "to accelerate and enlarge the contribution of atomic energy to peace, health and prosperity throughout the world".

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SAFETY GUIDE

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FOREWORD

by Mohamed ElBaradei
Director General

The IAEA's Statute authorizes the Agency to establish safety standards to protect health and minimize danger to life and property — standards which the IAEA must use in its own operations, and which a State can apply by means of its regulatory provisions for nuclear and radiation safety. A comprehensive body of safety standards under regular review, together with the IAEA's assistance in their application, has become a key element in a global safety regime.

In the mid-1990s, a major overhaul of the IAEA's safety standards programme was initiated, with a revised oversight committee structure and a systematic approach to updating the entire corpus of standards. The new standards that have resulted are of a high calibre and reflect best practices in Member States. With the assistance of the Commission on Safety Standards, the IAEA is working to promote the global acceptance and use of its safety standards.

Safety standards are only effective, however, if they are properly applied in practice. The IAEA's safety services — which range in scope from engineering safety, operational safety, and radiation, transport and waste safety to regulatory matters and safety culture in organizations — assist Member States in applying the standards and appraise their effectiveness. These safety services enable valuable insights to be shared and I continue to urge all Member States to make use of them.

Regulating nuclear and radiation safety is a national responsibility, and many Member States have decided to adopt the IAEA's safety standards for use in their national regulations. For the Contracting Parties to the various international safety conventions, IAEA standards provide a consistent, reliable means of ensuring the effective fulfilment of obligations under the conventions. The standards are also applied by designers, manufacturers and operators around the world to enhance nuclear and radiation safety in power generation, medicine, industry, agriculture, research and education.

The IAEA takes seriously the enduring challenge for users and regulators everywhere: that of ensuring a high level of safety in the use of nuclear materials and radiation sources around the world. Their continuing utilization for the benefit of humankind must be managed in a safe manner, and the IAEA safety standards are designed to facilitate the achievement of that goal.

IAEA SAFETY STANDARDS

SAFETY THROUGH INTERNATIONAL STANDARDS

While safety is a national responsibility, international standards and approaches to safety promote consistency, help to provide assurance that nuclear and radiation related technologies are used safely, and facilitate international technical cooperation and trade.

The standards also provide support for States in meeting their international obligations. One general international obligation is that a State must not pursue activities that cause damage in another State. More specific obligations on Contracting States are set out in international safety related conventions. The internationally agreed IAEA safety standards provide the basis for States to demonstrate that they are meeting these obligations.

THE IAEA STANDARDS

The IAEA safety standards have a status derived from the IAEA's Statute, which authorizes the Agency to establish standards of safety for nuclear and radiation related facilities and activities and to provide for their application.

The safety standards reflect an international consensus on what constitutes a high level of safety for protecting people and the environment.

They are issued in the IAEA Safety Standards Series, which has three categories:

Safety Fundamentals

— Presenting the objectives, concepts and principles of protection and safety and providing the basis for the safety requirements.

Safety Requirements

— Establishing the requirements that must be met to ensure the protection of people and the environment, both now and in the future. The requirements, which are expressed as 'shall' statements, are governed by the objectives, concepts and principles of the Safety Fundamentals. If they are not met, measures must be taken to reach or restore the required level of safety. The Safety Requirements use regulatory language to enable them to be incorporated into national laws and regulations.

Safety Guides

— Providing recommendations and guidance on how to comply with the Safety Requirements. Recommendations in the Safety Guides are expressed as 'should' statements. It is recommended to take the measures stated or equivalent alternative measures. The Safety Guides present international good practices and increasingly they reflect best practices to help users striving to achieve high levels of safety. Each Safety Requirements publication is supplemented by a number of Safety Guides, which can be used in developing national regulatory guides.

The IAEA safety standards need to be complemented by industry standards and must be implemented within appropriate national regulatory infrastructures to be fully effective. The IAEA produces a wide range of technical publications to help States in developing these national standards and infrastructures.

MAIN USERS OF THE STANDARDS

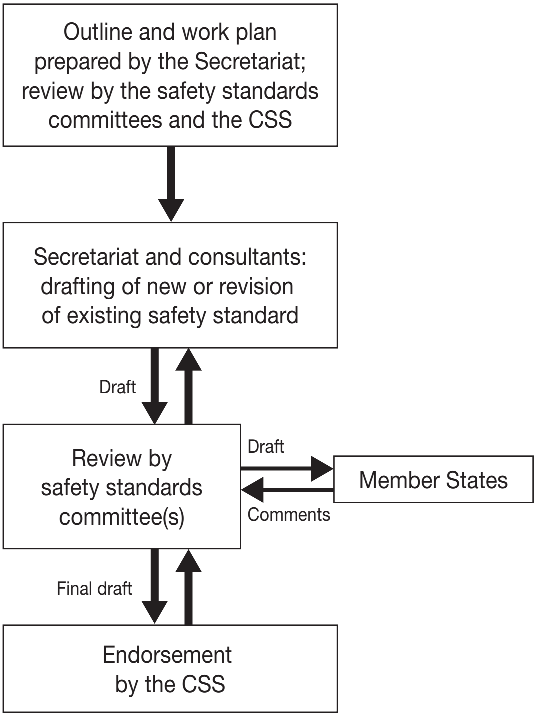
As well as by regulatory bodies and governmental departments, authorities and agencies, the standards are used by authorities and operating organizations in the nuclear industry; by organizations that design, manufacture and apply nuclear and radiation related technologies, including operating organizations of facilities of various types; by users and others involved with radiation and radioactive material in medicine, industry, agriculture, research and education; and by engineers, scientists, technicians and other specialists. The standards are used by the IAEA itself in its safety reviews and for developing education and training courses.

DEVELOPMENT PROCESS FOR THE STANDARDS

The preparation and review of safety standards involves the IAEA Secretariat and four safety standards committees for safety in the areas of nuclear safety (NUSSC), radiation safety (RASSC), the safety of radioactive waste (WASSC) and the safe transport of radioactive material (TRANSSC), and a Commission on Safety Standards (CSS), which oversees the entire safety standards programme. All IAEA Member States may nominate experts for the safety standards committees and may provide comments on draft standards. The membership of the CSS is appointed by the Director General and includes senior government officials having responsibility for establishing national standards.

For Safety Fundamentals and Safety Requirements, the drafts endorsed by the Commission are submitted to the IAEA Board of Governors for approval for publication. Safety Guides are published on the approval of the Director General.

Through this process the standards come to represent a consensus view of the IAEA's Member States. The findings of the United Nations Scientific Committee on the Effects of Atomic Radiation (UNSCEAR) and the recommendations of international expert bodies, notably the International Commission on Radiological Protection (ICRP), are taken into account in developing the standards. Some standards are developed in cooperation with other bodies in the United Nations system or other specialized agencies, including the Food and Agriculture Organization of the United Nations, the International



The process for developing a new safety standard or revising an existing one.

Labour Organization, the OECD Nuclear Energy Agency, the Pan American Health Organization and the World Health Organization.

The safety standards are kept up to date: five years after publication they are reviewed to determine whether revision is necessary.

APPLICATION AND SCOPE OF THE STANDARDS

The IAEA Statute makes the safety standards binding on the IAEA in relation to its own operations and on States in relation to operations assisted by the IAEA. Any State wishing to enter into an agreement with the IAEA concerning any form of Agency assistance is required to comply with the requirements of the safety standards that pertain to the activities covered by the agreement.

International conventions also contain similar requirements to those in the safety standards, and make them binding on contracting parties. The Safety Fundamentals were used as the basis for the development of the Convention on Nuclear Safety and the Joint Convention on the Safety of Spent Fuel Management and on the Safety of Radioactive Waste Management. The Safety

Requirements on Preparedness and Response for a Nuclear or Radiological Emergency reflect the obligations on States under the Convention on Early Notification of a Nuclear Accident and the Convention on Assistance in the Case of a Nuclear Accident or Radiological Emergency.

The safety standards, incorporated into national legislation and regulations and supplemented by international conventions and detailed national requirements, establish a basis for protecting people and the environment. However, there will also be special aspects of safety that need to be assessed case by case at the national level. For example, many of the safety standards, particularly those addressing planning or design aspects of safety, are intended to apply primarily to new facilities and activities. The requirements and recommendations specified in the IAEA safety standards might not be fully met at some facilities built to earlier standards. The way in which the safety standards are to be applied to such facilities is a decision for individual States.

INTERPRETATION OF THE TEXT

The safety standards use the form 'shall' in establishing international consensus requirements, responsibilities and obligations. Many requirements are not addressed to a specific party, the implication being that the appropriate party or parties should be responsible for fulfilling them. Recommendations are expressed as 'should' statements, indicating an international consensus that it is necessary to take the measures recommended (or equivalent alternative measures) for complying with the requirements.

Safety related terms are to be interpreted as stated in the IAEA Safety Glossary (http://www-ns.iaea.org/standards/safety-glossary.htm). Otherwise, words are used with the spellings and meanings assigned to them in the latest edition of The Concise Oxford Dictionary. For Safety Guides, the English version of the text is the authoritative version.

The background and context of each standard within the Safety Standards Series and its objective, scope and structure are explained in Section 1, Introduction, of each publication.

Material for which there is no appropriate place in the main text (e.g. material that is subsidiary to or separate from the main text, is included in support of statements in the main text, or describes methods of calculation, experimental procedures or limits and conditions) may be presented in appendices or annexes.

An appendix, if included, is considered to form an integral part of the standard. Material in an appendix has the same status as the main text and the IAEA assumes authorship of it. Annexes and footnotes to the main text, if included, are used to provide practical examples or additional information or explanation. An annex is not an integral part of the main text. Annex material published by the IAEA is not necessarily issued under its authorship; material published in standards that is under other authorship may be presented in annexes. Extraneous material presented in annexes is excerpted and adapted as necessary to be generally useful.

CONTENTS

1. INTRODUCTION

BACKGROUND

1.1. This Safety Guide, which supplements the Safety Requirements publication on Site Evaluation for Nuclear Installations, is issued under the IAEA's programme for establishing Safety Requirements and Safety Guides relating to land based nuclear installations.

1.2. The present Safety Guide supersedes a Safety Guide issued in 1986 as Safety Series No. 50-SG-S8, Safety Aspects of Foundations of Nuclear Power Plants. The revision involved principally an updating of the technical content in accordance with developments in geotechnical engineering and the feedback of experience, and reorganization of the text. In the revision process, it was decided to extend the scope to include earth structures, which were previously addressed in Safety Series No. 50-SG-D15, Seismic Design and Qualification for Nuclear Power Plants, which has now been superseded by Ref. [1].

OBJECTIVE

1.3. The purpose of this Safety Guide is to provide guidance on dealing with geotechnical engineering aspects that are important for the safety of nuclear power plants. Seismic aspects also play an important role in this field, and consequently the Safety Guide on Evaluation of Seismic Hazards for Nuclear Power Plants, Safety Standards Series No. NS-G-3.3 [2], which discusses the determination of seismic input motion, is referenced on several occasions. The present Safety Guide provides an interpretation of the Safety Requirements on Site Evaluation for Nuclear Installations [3] and guidance on how to implement them. It is intended for the use of safety assessors or regulators involved in the licensing process as well as the designers of nuclear power plants, and it provides them with guidance on the methods and procedures for analyses to support the assessment of the geotechnical aspects of the safety of nuclear power plants.

SCOPE

1.4. In the process of selection of a new site for a nuclear installation, a series of parameters are required to be considered, as established in Ref. [3]. These parameters usually play a prominent role, and the site finally selected is seldom an ideal one with regard to geotechnical conditions (practically, a site may be dropped from consideration on geotechnical grounds only if the conditions are very poor). This Safety Guide therefore provides guidance for dealing with the realistic possibility of a situation in which complex geotechnical conditions are faced.

1.5. This Safety Guide discusses the geotechnical engineering aspects of the subsurface conditions and not the geological aspects, except where these directly affect the foundation system. It discusses the programme of investigations that should be carried out to obtain an appropriate understanding of the subsurface conditions, which is necessary for determining whether the conditions are suitable for the construction of a nuclear power plant. It also provides a description of the geotechnical profiles and the parameters that are suitable for use in performing the geotechnical analyses that are required for the design of a nuclear power plant. It also discusses the monitoring of the geotechnical parameters at the site.

1.6. Methods of analysis appropriate for the safety assessment of the site are discussed here, particularly for the assessment of the effects of an earthquake on the site, including the determination of site specific response spectra and estimation of the liquefaction potential. The Safety Guide also discusses methods of analysis appropriate for the safety assessment of the effects of static and dynamic interaction between the soil and the structures, and of the consequences for the bearing capacities and for settlements. A more detailed description of methods for the analysis of soil–structure interactions is given in Ref. [1]. In this Safety Guide only the site dependent information and the methods of analysis are addressed.

1.7. This Safety Guide discusses foundation works, including the consequences for the geotechnical profiles and parameters, the possible improvement of foundation material and the appropriate choice of the foundation system according to the soil capacities.

1.8. Also discussed are earth structures, including natural slopes, and buried structures, the safety of which may need to be assessed. The Safety Guide discusses appropriate methods for the analysis of the behaviour of such structures under static and dynamic loads.

STRUCTURE

1.9. Section 2 concerns the programme of investigations, addressing the different stages of the programme and the sources of data; a special subsection is dedicated to the investigation of complex subsurface conditions. Section 3 covers the assessment of the site as it is before any construction and the relevant methods of analysis. Subsections are dedicated to site characterization from soft to stiff sites, relevant parameters for the description of the mechanical characteristics of the soil profiles, free field seismic response spectra and site specific response spectra, and the assessment of liquefaction potential. Section 4 focuses on considerations relating to the foundations; that is, to the site as it is modified by building construction. Foundation works are addressed first, followed by soil–structure interactions and their consequences for stability and for settlements. Sections 5 and 6 are dedicated to special structures. Section 5 addresses earth structures, with subsections on natural slopes, dykes and dams, embankments and cuts and fills, seawalls and similar structures. Section 6 addresses buried structures in a wide sense, with subsections on retaining walls, embedded structures, buried pipes and tunnels. Section 7 deals with the monitoring of geotechnical parameters.

2. SITE INVESTIGATION

INVESTIGATION PROGRAMME

2.1. Investigation of the subsurface conditions at a nuclear power plant site is important at all stages of the site evaluation process. The purpose of this investigation is to provide information or basic data for decisions on the nature and suitability of the subsurface materials. At each stage of the site evaluation, the investigation programme should provide the data necessary for an appropriate characterization of the subsurface. Detailed subsurface investigations should be performed in the later stages. The specific requirements will vary greatly from stage to stage.

2.2. The programme of investigation should cater to all stages of the site evaluation process. For a nuclear power plant, site evaluation typically involves the following stages:

— *Selection stage.* One or more preferred candidate sites are selected after investigation of a large region, rejection of unsuitable sites, and screening and comparison of the remaining sites.
— *Characterization stage.* This stage is further subdivided into:
  - Verification, in which the suitability of the site to host a nuclear power plant is verified mainly according to predefined site exclusion criteria;
  - Confirmation, in which the characteristics of the site necessary for the purposes of analysis and detailed design are determined.
— *Pre-operational stage.* Studies and investigations begun in the previous stages are continued after the start of construction and before the start of operation of the plant to complete and refine the assessment of site characteristics. The site data obtained allow a final assessment of the simulation models used in the ultimate design.
— *Operational stage.* Selected investigations are pursued over the lifetime of the plant.

2.3. The programme of investigation differs in the various stages, in that the data requirements vary greatly. Generally, the necessary data will yield geological and engineering related information for use in safety evaluations or analyses. These data can be classified as:

— Geological information (stratigraphical and structural);
— Descriptions of the extent and nature of subsurface materials;
— Characterizations of soil and rock (in terms of properties);
— Information on groundwater (the groundwater regime, locations and characteristics of the hydrological units, physical chemistry of the water).

2.4. The results of the investigations described in this section should be properly documented with reference to the particular site conditions (soil or rock), the stage of the site evaluation process concerned and the verification analysis required.

2.5. The various methods of investigation — that is, the use of current and historical documents, geophysical and geotechnical exploration in situ and laboratory testing — are applicable to all stages of the site evaluation process, but to varying extents. This section indicates the level of investigation necessary for the evaluation of the site in terms of the performance of subsurface materials and earthworks under the anticipated loading conditions (static and dynamic).

Selection stage

2.6. The purpose of an investigation at the site selection stage is to determine the suitability of sites. In this stage, geological, geomorphological and geotechnical aspects are considered and regions or areas are usually identified that are excluded from further consideration. Subsurface information for this stage is usually obtained from current and historical documents and by means of field reconnaissance, including geological and geomorphological surveys, and it is used in the following assessments:

— *Unacceptable subsurface conditions.* A site with geological conditions that could affect the safety of a nuclear power plant and that cannot be corrected by means of a geotechnical treatment or compensated for by constructive measures is unacceptable. Geological hazards such as surface faulting (see Ref. [3] under 'Potential for surface faulting at the site'), volcanic activity, landslides, permafrost, erosion processes, subsidence and collapse due to underground cavities (both natural and those deriving from human activities), or other causes should be identified and evaluated. The area of the investigation should be appropriate to the hazard under consideration.
— *Classification of sites.* The subsurface conditions at a site can be derived from the geological and geotechnical literature. A site may be classified as a rock site, a soft rock or stiff soil site, a soft soil site or a combination of these, and may be categorized as described in Section 3. The soil type is further divided into non-cohesive and cohesive soil. However, this rough classification may not apply for certain sites. For instance, quaternary formations may present complex interfaces between rock and clay that should be carefully investigated and monitored.
— *Groundwater regime.* The hydrogeological literature may allow an estimate to be made of the location of groundwater and the groundwater regime (see Ref. [4]).
— *Foundation conditions.* The type of soil, the depth to bedrock and the properties of the deposit may be determined. This permits the preliminary selection of acceptable foundation types.

2.7. On the basis of the above mentioned information on subsurface conditions, the potential or candidate sites can be ranked according to the suitability of the foundation. At this stage, inferences should also be made about geological hazards, seismic amplification effects, the liquefaction potential, the bearing capacity, potential settlement and swelling, soil–structure interactions and groundwater conditions. After this stage, sites are selected for further consideration on the basis of geotechnical considerations.

Verification stage

2.8. In the verification stage, it is assumed that broadly generalized layouts and building loads have been established. The following factors should be considered in the evaluation, to account for both normal conditions and extreme conditions such as earthquake and flood conditions:

— Geological hazards;
— Geological and subsurface conditions;
— Liquefaction potential;
— Feasible foundation types;
— Preliminary bearing capacity and other factors of foundation stability;
— Preliminary settlement ranges;
— Groundwater levels and regimes;
— Previous use of the site;
— Site preparation requirements.

In this stage, the investigation programme should cover the site as a whole as well as a smaller scale appropriate for layout considerations.

2.9. The following site investigation techniques and related points should be noted:

— *Rotary borehole drilling*. In this method of drilling, all cores are recovered to provide an overall definition of site conditions. This usually involves locating the borings along two intersecting lines with a common boring at the intersection; in addition to the extraction of cores or other samples for rock or soil qualification and laboratory testing, the boreholes can be used for the installation of instruments for long term in situ testing, including instruments for monitoring the groundwater regime. The possible effects of boreholes on the potable water regime should be investigated [4]. If necessary, test pits or test tunnels should be used to facilitate a direct examination of the subsurface conditions.
— *In situ testing*. According to the subsurface conditions, various types of simple in situ tests should be carried out to measure the mechanical properties of the foundation materials. These tests should also include various in situ loading tests and piezometric measurements of the groundwater.

— *Seismic refraction and reflection survey.* A seismic refraction and reflection survey should be conducted to provide continuous lateral and depth information for the evaluation of subsurface conditions. Interpretation of the survey results provides stratigraphic and structural geological information, information on the location of the groundwater table and an estimate of wave velocities at the site. The borings provide vertical stratigraphic confirmation for the survey.
— *Laboratory testing.* Limited laboratory testing consisting of index and classification tests should be conducted on rocks or soils. If cohesive soil samples were obtained during the drilling operation, appropriate consolidation and shear strength testing should be conducted on undisturbed samples to allow the estimation of soil strength and settlement.

2.10. In the field investigations, careful attention should be paid to identifying undesirable subsurface characteristics, such as cavity zones, swelling rocks and shales, the occurrence of gas pockets, zones of weakness or discontinuities in crystalline rocks, and potential slide planes predetermined by unstable subsurface layers.

Confirmation stage

2.11. The purpose of the site confirmation stage is to provide confirmation of the results obtained in the previous stages. A subsurface exploration and a laboratory testing programme should be conducted at the site using either a grid boring scheme or an alternative boring scheme suited to the site and the installation under consideration. The grid spacing may vary depending on the geometry of the subsurface characteristics. The uniform grid method is especially adaptable to a site with relatively uniform soil conditions. Where dissimilarities and discontinuities are present, the usual exploration process should be supplemented with borings at spacings small enough to permit detection of the features and their proper evaluation. The consequences of boring for the groundwater regime, and possibly for potable water, should be considered.

2.12. As a minimum, the following indicators of potential cavities and susceptibility to ground collapse should be considered:

— Sinks, sink ponds, caves and caverns;
— Sinking streams;
— Historical ground subsidence;

— Mines and signs of associated activities;
— Natural bridges;
— Surface depressions;
— Springs;
— Rock types such as limestone, dolomite, gypsum, anhydrite, halite, terra rossa soils, lavas, weakly cemented clastic rocks, coal or ores;
— Non-conformities in soluble rocks.

2.13. In this stage, preliminary plant characteristics such as the loads, the physical dimensions of the buildings, preliminary structural engineering criteria and the preferred plant layouts are known. The content of the in situ testing and laboratory testing programmes should be planned on the basis of both the preliminary plant characteristics and the geotechnical issues that were identified in the previous stage.

2.14. The necessary boring depths will vary with the site conditions, but the borings should be deep enough to be able to describe fully the site conditions that would affect structures and to confirm the soil and rock conditions determined in previous investigations. Where soils are very thick, and to enable the evaluation of potential deep instability at the site, the minimum boring depth for engineering purposes should be taken as the smaller of the following two values: (i) the depth at which the change in the vertical stress during or after construction is less than 10% of the in situ effective overburden stress, or (ii) the depth of one foundation diameter.

2.15. If the site is a rock site or if competent rock is encountered at a depth less than that recommended above, the borings should penetrate to the greatest depth at which discontinuities or zones of weakness or alteration could affect the stability of the foundation. For sites of weathered shale or soft rock, the boring depths should be the same as those for soil.

2.16. In this stage, sufficient in situ and laboratory testing should be conducted to allow the estimation of the bearing capacity, determination of settlements of structure and the site amplification of seismic waves, establishment of soil–structure interaction parameters (dynamic and static), evaluation of the liquefaction potential and evaluation of a site specific design response spectrum, if required. In addition to the boring programme described above, it may be necessary to include in the investigation programme several borings to establish the soil model for studies of dynamic soil–rock structure interactions. The borings required for site amplification studies may need to penetrate deeper than those required for normal purposes of geotechnical design.

2.17. If it has been found necessary to make improvements in the subsurface conditions, the improvements should be made at this stage and their effectiveness should be verified by in situ testing.

2.18. At this stage preliminary analyses should be carried out that cover the static stability, the response to dynamic loading, the liquefaction potential and the stability of slopes, embankments and dams. The analyses should be carried out on the basis of in situ exploration and data from laboratory testing.

2.19. The results of the investigations for this stage are usually combined with basic data obtained from the preceding phases in a detailed geotechnical report. This report should include the following items:

— Geological maps and profiles;
— Descriptions of geological factors and the site geology;
— An exploration programme and the basis thereof;
— Location plans and cross-sections for borings;
— Boring logs and test pit logs;
— The results of in situ testing;
— The results of laboratory testing;
— The results of geophysical surveys;
— Descriptions and results of analyses;
— Detailed descriptions of the groundwater regime and the physico-chemical properties of the groundwater.

2.20. The results from the site verification stage should provide the necessary information for establishing broad design parameters and conclusions relating to the site and its characteristics. The verification stage should be consistent with the final layout of buildings on the site. Any further geotechnical information required will be related directly to the individual buildings, structures and support facilities.

2.21. When the final layout of the buildings, structures and support facilities is known, a differentiation should be made between safety related and non-safety-related structures. The subsurface exploration and testing programme for the non-safety-related structures should follow standard practices. In general, at least one boring should be drilled at the location of every safety related structure. Where conditions are found to be variable, the boring spacing should be chosen to obtain a clear definition of changes in soil and rock properties.

Pre-operational stage

2.22. Investigations should be continued after the start of construction until the start of operation of the plant to complete and refine the assessment of site characteristics by incorporating geotechnical data that are newly obtained during the excavation and construction of the foundations. The outcrops of subsurface material should be carefully observed and mapped to compare them with the designed conditions to confirm the design. If necessary, in situ tests may additionally be carried out by utilizing the base excavation.

2.23. The data obtained on actual performance in settlements and deformations due to structural loads should be used to verify the predicted behaviour of the foundations. Since the construction sequence is generally long, these data should be used to revise the settlement models and the soil properties on the basis of actual performance.

Operational stage

2.24. During operation of the plant, the settlement of structures, as well as parameters such as the level of the water table, should be measured and compared with predictions to enable an updated safety assessment to be made. The choice of the parameters to be measured, the type of records to be obtained, the measurement intervals and in general all the activities of site evaluation in the operational stage should be described in a maintenance programme. This stage is also discussed in Section 7.

SOURCES OF DATA

2.25. The purpose of the investigations is to provide information or basic data to allow informed decisions to be made concerning the nature and suitability of the subsurface materials. The sources of data are:

— Historical and current documents;
— In situ exploration;
— Laboratory tests.

Historical and current documents

2.26. The investigations require an understanding of the general geology of the area of interest. This should be obtained by means of field reconnaissance and a review of available historical and current documents, such as:

— Topographic maps;
— Geological and engineering geological maps;
— Soil maps;
— Geological reports and other geological literature;
— Geophysical maps;
— Geotechnical reports and other geotechnical literature;
— Earth satellite imagery and aerial photographs;
— Water well reports and water supply reports;
— Oil and gas well records;
— Hydrogeological maps, hydrological and tidal data, flood records, and climate and rainfall records;
— Mining history, old mine plans and subsidence records;
— Seismic data and historical earthquake records;
— Contemporary accounts of landslides, floods, earthquakes, subsidence and other geological events of significance;
— Records of the performance of structures in the vicinity.

2.27. Other possible sources of information should be considered, such as individual observers, geology and engineering departments of colleges and universities, government geological surveys and engineering authorities, work done by other persons in the vicinity of the site, and observations made at quarries in operation.

In situ exploration

2.28. Two types of test, geophysical tests and geotechnical tests, are distinguished depending on the scale of the investigation, and tests of both types should be carried out.

2.29. The geophysical tests provide data or information that can be deduced by back analysis of the test results, but only in the domain of elastic deformation. These methods generally have a large coverage (in terms of depth and surface area) and provide only rough estimates of parameters (such as the thickness of the layers and parameters defining their mechanical properties) sufficient for the purposes of site evaluation. The surveys should include some or all of the different techniques shown in Table 1, according to the best practices under the circumstances, taking into account the subsurface conditions.

2.30. Geotechnical methods address the near field area (to a depth of at least one diameter of the reactor building base). There are many different techniques, using boreholes or working directly from ground level. In accordance with subsurface conditions, appropriate tests of those listed in Table 2 should be conducted.

TABLE 1. TECHNIQUES FOR GEOPHYSICAL INVESTIGATIONS OF SOIL AND ROCK SAMPLES

Type of test	Parameter measured	Types of problems	Observations
Seismic refraction/ reflection	Deformation time propagation	Site categorization	For surface investigation
Cross-hole seismic test	Dynamic elastic properties	Site categorization, soil–structure interaction	For deep investigations: one hole for emission, one hole for reception
Uphole/downhole seismic test	Dynamic elastic properties	Site categorization, soil–structure interaction	For deep investigations: one hole for both emission and reception
Nakamura method	Low level (ambient noise) vibrations	Site categorization, soil–structure interaction	
Electrical resistivity	Liquid table content	Internal erosion	Available for surface or deep investigation
Nuclear logging	Water content, density		Necessitates expensive logging techniques
Microgravimetry	Acceleration due to gravity	Sinkholes, heterogeneities	Complex subsurface
Georadar	Speed of propagation	Cavities	Complex subsurface
Magnetic techniques	Magnetic field intensity	Areas of humidity	Maintenance of dykes and dams

TABLE 2. TECHNIQUES FOR GEOTECHNICAL INVESTIGATIONS OF SOIL AND ROCK SAMPLES

Type of test	Type of materials	Parameter measured	Types of problems	Comments
Flat jack test	Rock	In situ normal stress	Deformability, convergence	Questionable results in rock with strongly time dependent properties
Hydraulic fracturing test	Rock	In situ stress state	Deformability, convergence	Affected by anisotropy of tensile strength
Direct shear stress test	Rock	Shear strength	Stability problems	Usually requires a sufficient number of tests for statistical control
Plate bearing tests	Clay, sand, gravel, rock	Reaction modulus	Compaction control; settlement	Used for excavations and embankments
Pressure meter test	Clay, sand, gravel, rock	Elastic modulus; compressibility	Settlement; bearing capacity	Needs a preliminary hole
Static penetrometer test	Clay, sand, gravel	Cone resistance; undrained cohesion; shear strength	Settlement; bearing capacity	Including cone penetrometer test
Dynamic penetrometer test	Clay, sand, gravel	Cone resistance; relative density	Liquefaction	Including standard penetration test
Vane shear test	Soft clay	Shear strength	Bearing capacity, slope stability	Not suitable for silt, sand or soils with appreciable amounts of gravel or shells
Pumping test	Clay, sand, gravel	Field permeability	Transmissivity of soil	Needs piezometers

Laboratory tests

2.31. Laboratory testing should be conducted on samples obtained by methods of direct exploration. The recovery of good undisturbed samples is important to the overall success of the laboratory testing. The treatment of samples after collection is as critical to their quality as the procedure used to obtain them. Handling, field storage and transport to the laboratory should be given careful attention. Sampling should be performed by means of pits, trenches or excavations and by in-hole methods. It may be necessary in certain circumstances to freeze 'cohesionless' soils in order to obtain undisturbed samples.

2.32. The purpose of laboratory testing is to supplement and confirm the in situ test data in order to characterize the soil and rock at the site fully and correctly, over the whole range of expected strains. The material damping ratio of the soil, for example, as well as other mechanical properties for large strains, are not easily obtainable by in situ tests. All phases of the site investigation and the associated field and laboratory testing should be carefully planned and carried out so that the properties of soil and rock can be realistically assessed in a timely manner.

2.33. The testing programme should identify and classify soil and rock samples. Their physical properties and engineering characteristics should be obtained from published data or by measurement. The laboratory tests should be directed towards the purposes shown in Table 3.

2.34. Site characterization parameters for use in the design profile should be carefully derived from the results of in situ and laboratory tests. Any discrepancies between the results of in situ and laboratory tests should be investigated and reconciled.

INVESTIGATIONS FOR COMPLEX SUBSURFACE CONDITIONS

2.35. The site investigation programme for nuclear power plants should include considerations for potential complex subsurface conditions. Such conditions encountered at a site could have serious implications for the integrity of the foundation of a nuclear power plant. Complex subsurface conditions include the potential for the occurrence of underground openings, of either natural or artificial origin, that could lead to a collapse. Consideration should also be

TABLE 3. TECHNIQUES FOR LABORATORY INVESTIGATIONS OF SOIL AND ROCK SAMPLES

Characteristics investigated	Type of soil	Test	Parameter measured	Purpose
Soil index and classification	Clayed soil	Atterberg limits	Water content (through liquidity and plasticity indexes)	Compressibility and plasticity
Physical and chemical properties of soils	All types	Dietrich–Frühling apparatus	Carbonates and sulphates	Soil classification
Physical and chemical properties of groundwater	All types		Salt content	Influence on permeability
Soil moisture–density relationships	All types	Proctor test, gammametry, ASTM[a] test (relative density)	Humid and dry densities, water content, saturation ratio, relative density	Settlement, consolidation, bearing capacity
Consolidation and permeability characteristics	All types	Oedometer	Oedometric, Young's modulus, consolidation coefficient	Settlement, consolidation
Shear strength and deformation capability of soil	All types	Shear test box, triaxial compression tests	Young's modulus, Poisson's ratio cohesion and fiction angle under drained and undrained conditions	Settlement, bearing capacity
Mechanical properties of rock	Rock	Shear test, biaxial or triaxial compression tests	Young's modulus and Poisson's ratio	Stability, strengthening
Dynamic characteristics of the soil	All types	Cyclic triaxial tests, resonant column	Dynamic Young's modulus, Poisson's ratio, internal damping, pore pressure	Site categorization, soil–structure interaction, liquefaction

[a] ASTM International, formerly known as the American Society for Testing and Materials (ASTM).

given to other ground conditions such as sinkholes and open joints that give rise to hazardous effects of other types such as piping and seepage.

2.36. The requirements for exploration, testing and analysis may vary depending on the conditions encountered and it is difficult to specify investigation programmes that cover all abnormal subsurface conditions. However, the basic elements of the investigation programme for complex subsurface conditions should include prediction, detection, evaluation and treatment.

Prediction of complex subsurface conditions

2.37. Prediction of the presence of cavities and subsurface discontinuities that could give rise to potential ground collapse and discontinuous geotechnical behaviour should be performed as an important step. Part of the Earth's surface is underlain by formations that have the potential for ground collapse as a result of solution processes or karstic phenomena.

2.38. Proper evaluation and understanding of the regional and site geology can provide indications of potential ground collapse. Soluble rocks are usually either sedimentary rocks that are appreciably soluble in water or in weakly acidic solutions (including carbonate types, mainly limestone and dolomites) or evaporates (of which halite, gypsum and anhydrite are the most common). The size of the cavities or underground solution is governed by both geological and environmental factors. The geological factors include the potential for buried channels, the stratigraphic sequence, the characteristics of the rock type and the properties of the rock mass. The environmental factors include surface water and groundwater hydrology, climate and climate change.

Detection of subsurface cavities

2.39. The subsurface exploration programme at a site should provide for the detection of subsurface cavities and should allow the extent of cavities to be evaluated. The possibility of the detection of areas susceptible to ground collapse should be considered in all aspects of the exploration programme. The conventional methods of site exploration are applicable, including hydraulic pressure tests, remote sensing, drilling, sampling, excavation, borehole logging and geophysical surveys. Standard methods of site investigation should be adopted to take into account possible complications caused by subsurface cavity systems.

2.40. If the presence of subsurface cavities is suspected at a site, the initial subsurface exploration programme to locate cavities may be based on probabilistic methods such as the theory of optimal search.

2.41. Some geophysical methods are useful in a reconnaissance mode for the detection of subsurface cavities, but not for delineating their depth, size or geometry. Such methods include surface electrical resistivity profiling, microgravimetry, seismic refraction surveys, seismic fan shooting and ground probing radar methods.

2.42. Geophysical methods that can be used as high resolution survey techniques in determining the depth, size and geometry of subsurface cavities include cross-hole seismic survey, cross-hole radar methods, electrical resistivity survey, acoustic resonance with a subsurface source, microgravimetry, seismic refraction, high resolution seismic reflection and ground probing radar methods. Several of these may be applied in conjunction with tomographic techniques.

2.43. Geophysical methods should be used with care and should usually be used in conjunction with drilling and sampling techniques that enhance their effectiveness. The result of an exploration programme to detect and define subsurface cavities should be a map showing the cavities and their relationships to the site structures.

2.44. It may not be possible or practicable, however, to detect and delineate every possible cavity or solution feature at the site. Consequently, a decision should be made on the largest possible undiscovered cavity that would be tolerable, on the basis of the effects of such cavities on the performance of important structures.

Evaluation and treatment of complex subsurface conditions

2.45. The greatest risk to the foundation safety of a nuclear power plant is from the existence of filled or open cavities and solution filled features at shallow depths (relative to the size) below the foundation of the structure. The compressibility and the erosion potential of the natural filling material should be evaluated to determine their impact on the bearing capacity, settlement and future erosion as a result of possible changes in the groundwater regime.

2.46. The stability of natural cavities below the foundation level should be considered. The size of the cavity, its depth, joint patterns, joint conditions, type of rock and bedding angles above the cavity are primary factors that influence the stability of the roof and the depth for consideration. An increase in the vertical pressures due to the structural loads could cause instability of the roof of the cavity. A site that is underlain by a potentially large and complex cavity system should be avoided since a realistic evaluation of the cavity system may be very difficult. In areas where the size and geometry of the cavity can be reliably determined, analytical techniques such as finite element analysis can be used for the evaluation of the stability of cavities.

2.47. For some sites where complex subsurface conditions are encountered below the foundation level, the results of the stability evaluation could indicate that ground treatment is required to ensure the safety of the structure. The general requirements for the improvement of foundation conditions for complex subsurface conditions are considered in Section 4.

3. SITE CONSIDERATIONS

SITE CATEGORIZATION

3.1. For the purpose of seismic response analyses, the following site categorization is used:

— Type 1 sites: $V_s > 1100$ m/s;
— Type 2 sites: 1100 m/s $> V_s > 300$ m/s;
— Type 3 sites: 300m/s $> V_s$;

where V_s is the best estimate shear wave velocity of the foundation medium just below the foundation level of the structure in the natural condition (i.e. before any site work), for very small strains. The site categorization is valid on the assumption that the shear wave velocity does not decrease significantly with depth; other than in this case, particular analyses should be carried out according to the best practices.

3.2. If the above mentioned site categorization is not valid, soil investigations should be carried out to determine the soil type for the site, or to provide comprehensive data for further analyses.

PARAMETERS OF THE PROFILES

3.3. A set of parameters should be determined in order to perform the geotechnical evaluation necessary for the construction of a nuclear power plant. The resulting set of parameters and data is called the profile. The profile may be defined as a geometrical and mechanical description of the subsurface materials in which the best estimates and ranges of variation for the characteristics of the foundation materials are determined and described in a way that is directly applicable to the subsequent analysis. The profile includes:

(1) The geometrical description, such as subsurface stratigraphic descriptions, lateral and vertical extents, number of layers and layer thicknesses;
(2) The physical and chemical properties of soil and rock and values used for classification;
(3) S and P wave velocities, stress–strain relationships, static and dynamic strength properties, consolidation, permeability and other mechanical properties obtained by in situ or laboratory tests;
(4) Characteristics of the groundwater table, the design level of the water tables and the maximum water level due to the maximum probable flood and other conditions.

3.4. As a result of the programme of in situ exploration and laboratory testing that is performed to obtain information on the relevant subsurface material properties and to aid in the definition of the subsurface model, many values of the geotechnical parameters are obtained. At this point, on the basis of the available information, a selection should be made of an appropriate set of representative parameters that are most suitable for use in the models for geotechnical analyses. In these analyses, the effects of uncertainties in the geotechnical parameters on the variability of the analytical results should be determined by means of parametric studies.

3.5. Even though conceptually the profile is unique to a particular site, various related design profiles for different purposes should be adopted to allow for different hypotheses in the analysis. Design profiles are presented in other sections for the assessment of the following:

— Site specific response spectrum;
— Liquefaction potential;
— Stresses in the foundation ground;
— Foundation stability;

— Soil–structure interaction;
— Settlements and heaves;
— Stability in earth structures;
— Earth pressure and deformation in buried structures.

FREE FIELD SEISMIC RESPONSE AND SITE SPECIFIC RESPONSE SPECTRA

3.6. For the purpose of the present Safety Guide, the seismic input level that should be considered is the SL-2 level[1], as defined in the Safety Requirements publication on Site Evaluation for Nuclear Installations [3], as specified in the Safety Guide on Evaluation of Seismic Hazards for Nuclear Power Plants [2] and as determined in accordance with Section 5 of Ref. [2].

3.7. A computation of site response under free field conditions should be carried out for sites other than Type 1 sites (see para. 3.1). This computation of site response may be needed for the assessment of settlement or liquefaction as well as for soil–structure interaction analyses. The site response computation may also be required for developing specific site response spectra. To carry out this computation, data on the following should be gathered:

— The input ground motion (derived by means of the procedures described in Ref. [2]);
— An appropriate model of the site, based on:
  - The geometrical description of the soil layers;
  - The velocities of the S and P waves in each layer;
  - The relative density of the soil in each layer;
  - The G–γ and η–γ curves which for each layer describe the apparent reduction of the shear modulus G and the internal damping ratio η of the soil with the shear strain γ;
— For those deep soil deposits in which wave velocities increase smoothly with depth, the change with depth of the aforementioned parameters.

[1] Seismic level 1 and seismic level 2 (SL-1 and SL-2) are levels of ground motion (representing the potential effects of earthquakes) considered in the design basis for a facility. SL-1 corresponds to a less severe, more likely earthquake than SL-2. In some States, SL-1 corresponds to a level with a probability of 10^{-2} per year of being exceeded, and SL-2 corresponds to a level with a probability of 10^{-4} per year of being exceeded.

3.8. Depending on engineering practices, the input ground motion may be representative of the ground surface motion either on the site or at a hard outcrop. For Type 3 sites, the input ground motion at a neighbouring hard outcrop (Type 1 site) should be provided; or, if this is not possible, at a neighbouring stiff soil outcrop (Type 2 site); or, if this is also not possible, at an appropriate subsurface level.

3.9. In the case of an input ground motion provided at surface level, a deconvolution computation of the input motion in free field conditions should be carried out as a preliminary stage of a consistent soil–structure interaction analysis for sites other than Type 1 sites (see para. 3.1). A high reduction in input ground motion should be carefully justified by means of parametric studies. Use at the foundation level of an input ground motion provided at the surface level instead of a deconvoluted input motion is a conservative practice and is acceptable.

3.10. If the input ground motion is not provided in a form suitable for geotechnical studies, an adequate input ground motion should be determined. This input motion should be chosen according to earthquake intensity, magnitude, epicentral distance, maximum acceleration, duration, frequency content and other parameters.

3.11. In order to compute the site response, the following model is acceptable:

— A viscoelastic soil system overlying a viscoelastic half-space;
— A horizontally layered system;
— Materials that dissipate energy by internal damping;
— Vertically propagating body waves (shear and compression waves).

Non-linear effects may be approximated by equivalent linear methods. The equivalent linear model(s) of soil constitutive relationships should be consistent with the strain level induced in the soil profile by the response to the input ground motion. This generally leads to an iterative process.

3.12. Uncertainties in the mechanical properties of the site materials should 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 $(1 + C_v)$ and the best estimate value divided by $(1 + C_v)$, where C_v is defined as the coefficient of variation. The minimum value of C_v is 0.5. Attention should be paid to the fact that a given soil profile cannot be assumed without an assessment to be conservative for all the items under consideration;

that is, a conservative profile for deconvolution may not be conservative for the site response analysis.

3.13. When the site is in the near field of a seismic source, the site response model should be carefully determined so that the frequency content of the input motion generated by the earthquake mechanism may be appropriately taken into account.

3.14. In the case of a Type 3 site, site specific response spectra should be determined; they should be at least representative of the response of the profile at the surface level.

LIQUEFACTION POTENTIAL

Design profile for liquefaction potential

3.15. The assessment of the liquefaction potential is mentioned in Ref. [2]. Soils susceptible to liquefaction are normally non-cohesive soils such as sand and gravel containing a small proportion of silts and clays and occurring in loosely deposited conditions below the water table.

3.16. For soils susceptible to liquefaction, the information on the design profile that is needed to evaluate the liquefaction potential is as follows:

(1) *Groundwater regime.* Data from measurements made with piezometers installed at the site should be used to establish an appropriate water level for use in liquefaction analysis. The groundwater regime reflects the seasonal variations in the water level. Appropriate conservative values may be assumed for the analysis, supported by such data as are or as become available. Data from measurements made in inspection wells may be used to establish the permeability parameters.

(2) *Grain size distribution.* For non-cohesive soils, grain size distributions should be obtained by means of sieving tests of soils sampled from different points of the site and at different depths. The fines content read off from the grain size distributions and the associated plasticity are significant considerations in judging the liquefaction resistance on the basis of the standard penetration test (SPT) blow counts or the cone penetration test (CPT) records.

(3) *Standard penetration tests.* SPT blow-counts at different locations should be plotted against depth, preferably on a chart of the same scale. From

these SPT values, undrained cyclic strength can be evaluated on the basis of empirical relationships. In the grain size tests attention should be paid to the percentage of fines content, which has a significant effect in these relationships. Even a soil with a fines content of more than 30% still tends sometimes to liquefy. In such cases the plasticity index of the fine soils should be measured so that their susceptibility to liquefaction can be properly judged on the basis of that value.

(4) *Cone penetration tests.* The CPT for penetration resistance has an advantage over the SPT in that it can give a very detailed profile of stratification, allowing a better judgement to be made on the extent of liquefiable soil. Even if the soil cannot be sampled in a CPT, soil types can be estimated on the basis of the ratio between the friction measured at the friction sleeve provided above the cone and the cone resistance. In a CPT the penetrability decreases with increasing soil density, which limits its use to rather loose sand only. For some site conditions, the combination of an SPT and a CPT may be more appropriate.

(5) *Relative density.* The in situ relative density of cohesionless soils is sometimes evaluated on the basis of the SPT blow counts because this functions as a convenient index to make a rough evaluation of the undrained cyclic strength or to determine the degree of instability of the soils once the pore pressure is 100% built up. In laboratory tests the relative density of soil samples is directly determined on the basis of the minimum and maximum densities of sand, for which a standardized testing method is available.

(6) *Undrained cyclic strength.* The undrained cyclic shear strength of the subsurface materials may be evaluated more directly by means of cyclic loading tests in the laboratory for undisturbed or remoulded samples. In most engineering practice the cyclic triaxial test is usually employed to evaluate the undrained cyclic strength. Correction factors are applied to the cyclic strength values measured in the triaxial test to allow for approximation to actual field conditions. The number of cycles required to attain specified failure conditions (e.g. initial liquefaction or percentage of axial strain) under a given cyclic stress amplitude is evaluated. The level of cyclic stress is varied and other samples are tested. In this test, the quality of the undisturbed samples may have significant effects on the liquefaction potential. An experimental curve that shows the relationship between cyclic stresses and the number of uniform cycles required to cause liquefaction failure is then prepared.

A similar curve can be obtained for remoulded samples having various relative densities and consolidation pressures for relatively younger soils that are not so influenced by cementation or pre-straining effects. The value of the undrained cyclic strength thus obtained is normalized by the consolidation stress in normal engineering practice, which gives the stress ratio. The in situ consolidation stress should be chosen appropriately because the stress ratio tends to decrease with increasing confining stress for medium dense to dense sand.

(7) *Strain dependence of soil properties.* G–γ and η–γ curves for each layer are needed to describe the apparent reduction in shear modulus and the damping ratio of the soil versus the shear strain.

(8) *Other soil properties.* Other properties may need to be known according to the types of sophisticated analysis. Some of the properties can be investigated by additional laboratory tests such as undrained monotonic loading shear tests and consolidation tests.

(9) *Past liquefaction history.* In addition to the determination of the parameters of the design profile for liquefaction analysis and the characterization of the cyclic strength of the subsurface materials by laboratory testing, data on liquefaction that has occurred at the site or in the vicinity of the site in the past should be collected and carefully studied. A detailed investigation programme and a specific liquefaction analysis at such locations should be performed.

3.17. As a result of the collection of data and the conduct of tests, values of the following design profile parameters needed for the evaluation of the liquefaction potential should be specified:

— The thicknesses and variation of the subsurface layers;
— The average relative density and its variation for each layer;
— The lateral extent of each layer;
— The water level to be associated with the reference ground motion for liquefaction analyses;
— The stress ratio versus the number of loading cycle curves for different types of soil;
— The correction factors to account for deviation of the laboratory conditions from the actual field conditions;
— The number of equivalent uniform cycles considered representative of the reference ground motion at the site;
— Other soil parameters used for numerical analysis;
— The failure criteria for liquefaction.

Methods of evaluation of the liquefaction potential

3.18. Three approaches for evaluating a liquefaction potential can be used, depending on the subsurface conditions and the level of the risk of liquefaction.

— An empirical approach, which is based on actual performance during past earthquakes and in which evaluations may readily be made from the SPT or CPT data;
— A conventional analytical approach;
— A sophisticated analytical approach.

Empirical approach

3.19. In the empirical approach the liquefaction potential is evaluated by using charts correlating the stress ratio with the SPT or CPT penetration resistance, which were empirically established on the basis of past liquefaction case histories. The earthquake magnitude and the fines content should be properly chosen on these charts because the results of the evaluation are highly dependent on these parameters.

Conventional analytical approach

3.20. The conventional analytical approach comprises the following steps:

— Establishment of the cyclic strength characteristics of the foundation material in each layer. The failure criterion is defined, with account taken of a number of factors, which may include relative density, number of cycles of stress, confining stresses and the heterogeneity of the soil (correction factors to convert laboratory results to field conditions are determined).
— The choice of a set of appropriate accelerograms.
— Calculation for each layer of the stresses induced by the accelerograms. These stress histories are transformed into numbers of equivalent uniform cycles.
— Determination of the liquefaction potential by comparing in each layer the cyclic strength characteristic with the computed equivalent cycles.

3.21. The most severe earthquake used for the analysis of structures, systems and components may not necessarily be the same as the most severe earthquake used in considering the liquefaction of foundation materials. A distant seismic event with a long duration may produce a large number of significant cycles with low acceleration at the site and these may be critical for liquefaction.

Sophisticated analytical approaches

3.22. In sophisticated analytical approaches a constitutive model of soil is incorporated into the non-linear step by step analysis to evaluate directly the buildup of pore pressure and the dynamic ground response. In most cases the effective stress analysis is carried out because it can simulate time dependent changes in pore pressure and their effects on changes in the properties of soil. In this sophisticated analysis, the liquefaction potential can be directly assessed according to chosen seismic input motions in terms of the buildup of pressure or the development of strain. However, the results may be quite variable owing to different input motions, constitutive models and other parameters, and the final assessment should be made in consideration of the extent of variability.

3.23. Safety factors are determined from a comparison of the analytical results of the above mentioned analyses with:

— The results from the empirical approach;
— The lower bound solution obtained by applying an analytical approach.

3.24. It is generally possible to compute a lower bound solution in the analytical approach by using conservative assumptions for the design profile parameters. For loose sands, a slight increase in the seismic stresses could bring the soil into an unstable condition, with possible large deformations, while in medium to dense sands even a large increase in seismic stresses would generate only limited deformation despite 100% pore pressure buildup.

3.25. Acceptable safety factors cannot be specified a priori but should be specified on a case by case basis by using the results derived as described above. They should also be selected in such a way that dynamically induced strain or residual strain does not impair the performance of the foundation.

4. CONSIDERATIONS FOR THE FOUNDATIONS

FOUNDATION WORK

Preliminary foundation work

4.1. This section addresses the geotechnical aspects of preliminary foundation work. For the purposes of this Safety Guide, preliminary foundation work is defined as those geotechnical activities conducted prior to the placement of the concrete foundations. These activities directly affect the performance of the foundation under the anticipated loading conditions and are therefore critical to safety. They may include:

- Prototype testing (including test fills and verification of techniques for improving foundation material);
- Excavations for foundations or foundation systems;
- Dewatering and its control;
- Rock removal;
- Mapping of excavations;
- Improvement of foundation materials (including such items as modification of material and drainage);
- Placement of structural backfill;
- Placement of mud mats or any type of protective layer.

4.2. The earthwork aspects of these activities should include testing requirements for proper control and documentation of construction. The testing should be conducted in both the field and the laboratory and throughout the construction period.

Improvement of foundation conditions

4.3. The improvement of foundation conditions is meant here in its widest sense and includes modification of the mechanical behaviour of the foundation material (such as by soil compaction), the total replacement of loose or soft material by an improved material, or the use of an added material to improve the static and/or dynamic behaviour. Another approach is the use of deep foundations, as described in the following.

4.4. Improvement of the foundation conditions should be carried out if:

- The foundation material is not capable of carrying the building loads without unacceptable deformation (settlements);
- There are cavities that can lead to subsidence, as discussed in Section 2;
- There are heterogeneities, on the scale of the building size, that can lead to tilting and/or unacceptable differential settlements.

4.5. When improvement of the foundation conditions is required, the following tasks should be performed:

- Determination of the existing in situ profile;
- Determination of the required profile for foundation material;
- Selection of the particular technology by which improvements in the foundation are to be made (over-excavation and compacted backfill, rock removal, densification by various methods, solidification by cement or permanent dewatering);
- Carrying out a prototype testing programme to verify experimentally the effectiveness of the methods proposed to improve the subsurface conditions;
- After the proposed technology has been verified, preparation of the specifications for field operations;
- At the completion of the improvement programme, carrying out an investigation to determine whether the specifications were met;
- Incorporation of any improvement in foundation material into the design profiles used in the assessments.

Choice of foundation system and construction

4.6. Two systems of foundations are available for transmitting the superstructure loads to the soil: shallow foundations and deep foundations. Shallow foundations are used when the distribution of the load is sufficiently uniform and the upper layers of the soil are sufficiently competent. In the case of weak soil conditions, deep foundations are used to transfer the loads to stiffer soil layers at depth. Owing to the complexity of the design, shallow foundations are usually considered first, the option of deep foundations being considered as a last resort.

4.7. The following criteria should be applied in the choice of the foundation system:

- The forces due to the structures should be transmitted to the soil with no unacceptable deformation;

— The soil deformations induced by the SL-2 input motion should be compatible with the design requirements of the structure;
— The risks associated with the uncertainties in the evaluation of the seismic response should be considered in the design and construction of the foundation system;
— The risks associated with possibly ‘aggressive’ underground water should be taken into account;
— One single type of foundation should be used for each structure;
— The choice of the type of foundation should depend on the type of building (basemat should be used for the nuclear island because it provides homogeneous settlements under static and dynamic loads and because it provides a barrier between the environment and the buildings).

4.8. The analyses and the design profile should represent the behaviour of the structures under the anticipated loading conditions and hence the analysis of the foundation systems and structures should represent the as-built conditions.

SOIL–STRUCTURE INTERACTION

Static analysis

Input parameters

4.9. The distribution of contact pressures beneath the foundations and the stresses induced in the subsurface materials are derived from the analysis of the static soil–structure interaction. In addition to the elastic and geometric parameters of the structures, the following parameters of the subsurface materials should be included in the design profile to allow the foundation contact pressure to be computed:

— Elastic moduli and Poisson’s ratio of the soil and their variation with depth and with strain level;
— Subgrade reaction;
— Unit weight of the subsurface materials;
— Groundwater regime.

4.10. Additionally, if the subsurface materials are soils or soft rock, information on the stress history of the subsurface materials should be obtained to predict settlements and heaves, and to assess the hazard of gross foundation (shear) failure. For computing this stress history, the following should be obtained as a minimum:

— The geological stress history and the resulting preconsolidation stress and the overconsolidation ratio;
— The loading–unloading history in operations such as dewatering, excavation, backfilling and building construction, as well as the geometry of the disturbed spaces;
— The parameters required for the establishment and application of the constitutive law applicable to the subsurface materials and their variation with depth;
— The geometry and stiffness of the foundation mats as well as of the superstructure of the buildings.

Computer codes are available that carry out the computations and allow for the non-linear behaviour of the soil. Assessments of settlement are treated in more detail later in this section.

Methods of analysis

4.11. The most widely used type of foundation for nuclear power plants is the mat type. The design of the foundation mat should be analysed for types of structural stiffness behaviour that may be relevant (i.e. the infinitely rigid foundation, the flexible foundation or the actual structural stiffness). The stiffness of the superstructure should be taken into account if it is needed in this evaluation. To compute the distribution of contact pressure under the mat foundation, the subsurface foundation material can be modelled by the finite element technique (continuum representation) or by representing it as a series of springs whose stiffness corresponds to the coefficient of subgrade reaction (lumped representation).

4.12. For the two extreme conditions of infinitely stiff and infinitely flexible foundations (in the case of distributed load on soil), solutions are available in the literature. For intermediate conditions, which generally occur in reality, numerical solutions using computer codes are usually used. Consideration should be given to the condition in which the stiffness of the structures changes as the construction proceeds. Additionally, if the subsurface materials exhibit non-linear behaviour when subjected to unloading and reloading during excavation, dewatering and backfilling, this should also be considered.

4.13. For structures located close together, the possible effects of impacts of adjacent structures on the response of the foundation soil should be evaluated. In this case, a three dimensional analysis should be considered.

Dynamic analysis

Basic elements of the analysis of dynamic soil–structure interaction

4.14. The objective of the analysis of dynamic soil–structure interaction is to determine the dynamic response of the structure, with account taken of the effects of the coupling between the structure and the supporting foundation medium, when the combined system is subjected to externally applied dynamic loads or earthquake related ground motions.

4.15. For structures subjected to externally applied dynamic loads, such as wind, blast or forced excitation of vibration, the solution of the dynamic response of the soil–structure system includes the following three basic steps:

(1) Determining the dynamic properties of the structure (i.e. the structural modelling step);
(2) Determining the force displacement relationships for the foundation medium (i.e. the foundation impedance step);
(3) Determining the dynamic response of the coupled soil–structure system to the applied load (i.e. the analysis of the interaction response step).

4.16. For structures subjected to earthquake related ground motions, the solution of the dynamic response of the coupled soil–structure system requires, in addition to the steps described here, the determination of the ground motion input to the system. The determination of the ground motion input consists of two parts:

(1) Definition of the free field motion (i.e. the site response problem (see Section 3));
(2) Determination of the scattering (modification) of the free field motion due to the presence of the structure and the excavations.

4.17. In general soil–structure interaction analysis should be performed for sites with conditions of Type 2 or Type 3 foundation material (see Section 3). A fixed base support may be assumed in the modelling of plant structures for the seismic response analysis for Type 1 sites.

Steps for the analysis of seismic soil–structure interaction

4.18. A complete analysis of seismic soil–structure interaction should include the following steps:

— Site response analysis;
— Foundation scattering analysis;
— Foundation impedance analysis;
— Structural modelling;
— Analysis of the coupled system interaction response.

Input parameters

4.19. The following information should be available in the design profile to perform the analyses of the seismic soil–structure interaction:

— Best estimate value for body wave (compression and shear) velocity profiles with a range of variation as determined by in situ measurement techniques.
— Number and thickness of layers above the viscoelastic half-space. Layering is selected in such a way that each layer has uniform characteristics (i.e. the same soil type and the same shear wave velocity).
— The initial conditions of the subsurface materials represented by the shear wave velocity (or shear modulus) at small strain and by Poisson's ratio. These values are determined for each foundation layer of the model.
— Non-linear soil behaviour, which should be taken into account by making use of the equivalent linear material properties. The design parameters required for the equivalent linear method are the shear modulus and the damping versus shear strain relationships for each of the subsurface layers.
— The water level to be used in performing an analysis using the reference ground motion.
— The total unit weight of the materials of each layer.
— The depth of the embedment into the subsurface.
— The dimensions and geometry of the foundation.
— The stiffness of the foundation mat.
— The mass, stiffness and damping of the superstructure.

Methods of analysis

4.20. Analyses of soil–structure interaction should be performed to investigate the following effects:

— The effects of the foundation soil conditions on the dynamic response of the structure;

— The effects of buried structures (e.g. scattering effects);
— The effects of dynamic pressures and deformations on the buried structures;
— The uplift of the foundation;
— The effects of structure–soil–structure interactions.

4.21. The effects on these analyses of uncertainties in the design profile parameters for the foundation material should be considered. The effect of this variation is to produce a range of results that would envelop the response of the soil–structure interaction system, accounting for the uncertainties. An approach similar to that described in para. 3.12 should be used.

4.22. The contributions of damping of different types (material damping such as viscosity damping and hysteretic damping as well as radiation damping) should be considered. For soil–structure systems that consist of components (foundation system, structures and substructures) with different damping characteristics, modelling may be done by using composite modal damping. Maximum limits of damping values are generally used, but this will depend on the models and methods of analysis selected.

4.23. Several methods are available for representing the foundation medium in the analysis of soil–structure interaction. The four main methods used are the lumped parameter soil spring method, the 3-D continuum half-space substructuring method, the 3-D finite element substructuring method and the 2-D axisymmetric finite element direct (one step) method.

4.24. These methods for the analysis of soil–structure interaction have implicit and explicit assumptions and mathematical models embodied in them which give them different capabilities and limitations, and thus differing applicability. The analytical method to be used for each site condition should therefore be carefully chosen.

4.25. In analyses of soil–structure interactions, consideration should be given to the effects of soil layering, embedment, strain dependent soil properties, the level of the groundwater table and backfill conditions.

4.26. Since both the foundation soil and the structures exhibit three dimensional dynamic characteristics, the structure–soil–structure interaction problem is a three dimensional phenomenon. To represent adequately the characteristics of both the foundation soil and the structures of the nuclear power plant, a three dimensional analysis should therefore be performed.

STABILITY

4.27. The assessment of foundation stability should be carried out under static (i.e. permanent) loads and under a combination of static loads and dynamic loads induced by earthquake input (the vertical component of the seismic acceleration should be considered acting upwards or downwards). The assessment should include the consideration of bearing capacity, overturning and sliding.

Input parameters

4.28. The information required to perform a stability analysis includes:

(1) Geometrical data for the foundation;
(2) The loads on the foundation and the load combinations to be considered;
(3) The soil conditions, including the level of the water table and the following mechanical characteristics:

- — unit weight,
- — unit weight of backfill material,
- — cohesion,
- — angle of effective shearing resistance,
- — angle of shearing resistance between soil and structure; this angle should be less than or equal to the angle of effective shearing resistance for cast-in-place foundations and should be less than or equal to two thirds of the angle of effective shearing resistance for precast foundations.

4.29. The cyclic seismic forces generated in the foundation material by the earthquake input should be computed by an appropriate dynamic method to derive the maximum of these forces, and to estimate the number of equivalent loading cycles if this is necessary for the assessment of bearing capacity. These forces could be converted to static equivalent forces for the assessment of stability.

4.30. This method should also be applied to the analysis of uplifting and overturning and to the computation of lateral loads on subsurface walls and retaining walls. The equivalent static forces should be derived according to the item under consideration.

4.31. The water level should be assumed to be equal to the maximum water level due to the maximum probable flood for static loading. The groundwater level is assumed to be the mean level for the determination of the bearing capacity under SL-2 seismic loading.

Bearing capacity

4.32. Classically used procedures of soil mechanics for computing the ultimate load bearing capacity are acceptable if the subsurface material is relatively uniform. The analysis of elastic–plastic equilibrium can be performed for the plane strain and the axially symmetric cases. The foremost difficulty is the selection of a mathematical model of soil behaviour or its constitutive (stress–strain–time) relationship. The available solutions are generally limited to those developed for the rigid–plastic solid of the classical theory of plasticity. This solid is assumed to exhibit no deformation prior to shear failure and a plastic flow at constant stress after failure. These solutions are acceptable provided that the actual situation under consideration satisfies the assumptions associated with the method. In the case of heterogeneous subsurface conditions, the ultimate bearing capacity should be determined by the sliding surface method.

4.33. In the case of cohesive soils, both short term and long term bearing capacities should be assessed.

4.34. If the subsurface material exhibits considerable heterogeneity, anisotropy or discontinuity, the sliding surface method should be used instead of the bearing capacity formulas. In this method, potential sliding surfaces with smaller safety factors for sliding are predetermined for the subsurface material and analysed in a conventional slip surface analysis for behaviour under the initial static load and equivalent seismic load. If the calculated safety factor is lower than acceptable, further analysis should be performed. A dynamic analysis using acceleration time histories under the initial static load may be carried out. In all these analyses, the vertical seismic force should be taken into account in a conservative manner.

Safety factors

4.35. The potential for failure of the bearing capacity of the subsurface materials for a nuclear power plant under static loading should be low so that there are high margins of safety under static loading (this is generally the case). These margins should be sufficient to meet SL-2 seismic loading conditions with reasonable safety margins.

4.36. If a required safety factor is achieved on the basis of a conservative assumption, no further analysis is generally required. It should be noted that acceptable safety factors depend on the method of analysis and on other considerations. In the conventional bearing capacity method, the safety factor should not be lower than 3.0 under static loads, and should not be lower than 1.5 under combinations of loads that involve SL-2 seismic input (the overturning effect). The safety factor for the sliding surface method should be larger than 2.0 for the conventional slip surface analysis under combinations of loads that involve SL-2 seismic input. If the calculated safety factor is lower than acceptable, additional analysis should be performed.

4.37. Where fractured rock is present as foundation material, a local safety factor should also be included. The local safety factor is defined as the ratio of the strength to the working stress at each point where there might be yielding or local sliding along the existing fracture zones and weathered zones beneath the foundation. This factor indicates the extent of the yielding zones or the progressive failure of the material subjected to the design load. It is useful in determining the position and extent of the improvements that may be required in foundation materials and in choosing an appropriate technique for the improvements. If, under combinations of loads that involve the SL-2 seismic input, this safety factor is lower than 1 in an area sufficiently large that it would affect the performance of the structure, foundation conditions should be improved. However, the macroscopic stability should be judged on the factors of safety for bearing capacity and sliding.

Overturning

4.38. Under certain combinations of ground motion, groundwater level and geometrical configuration of the building, conventional computing procedures may give rise to a potential uplift. This does not mean that the foundation would necessarily lift up but rather that conventional procedures to compute the structural response may not be applicable under these circumstances. In the event that the estimated surface area of the uplift of the foundation is larger than 30% of the total surface of the foundation, a more sophisticated method should be used in the analysis of the dynamic soil–structure interaction. The estimated uplift of the foundation should be limited to a value that is acceptable in consideration of the bearing capacity of the soil and the functional requirements.

4.39. The uplift condition should be taken into account in the analysis of the bearing capacity of the foundation material.

Sliding

4.40. The possibility of sliding of the structure beneath the foundation should be investigated.

4.41. In the case of an embedded foundation, active pressure of the soil should be regarded as an additional horizontal load while the possible additional capacity of the foundation should be limited according to the at-rest value of the soil pressure.

4.42. The sliding safety evaluation of the foundation of the nuclear power plant should include not only an assessment of the balance of forces between the resistance and the design load, but also a comparison of the displacements (evaluated by appropriate methods, such as the finite element method or the boundary element method) during and after the SL-2 input motion with the acceptable value.

SETTLEMENTS AND HEAVES

Static analysis

4.43. An assessment of settlement under static loads should be performed. The possibility of differential settlements or heaves between the buildings of a nuclear power plant should be investigated because of the presence of pipes, conduits and tunnels providing connections between the facilities. Settlements and heaves are also important in connection with deformation of the foundation, which could lead to overstressing of buildings and interference with the operation of machinery such as pumps and turbines if they are not isolated from their supports.

4.44. Short and long term settlements (occurring during the operating lifetime of the plant) should be estimated.

4.45. Time dependent settlements may be computed by applying the classical theory of consolidation and other sophisticated non-linear analyses. In saturated soils, the following three components should be considered:

- Settlement without drainage, due to shear, for fully saturated soil;
- Settlement caused by consolidation;
- Settlement caused by creep.

4.46. The following actions are necessary to evaluate long term settlement:

— The anticipated loading history of the subsurface materials should be specified (excavation sequence, dewatering process, backfilling, construction process).
— The following parameters should be considered: preconsolidation pressure, coefficients of consolidation, the initial Young's modulus, Poisson's ratio and other parameters that define a particular constitutive relationship; their values should be determined for the entire profile of interest.
— For each layer a model should be chosen in accordance with data from laboratory and in situ testing.
— These models should be assessed and improved by means of the interpretation of measurements for settlement and heave made during excavation, dewatering, backfilling and construction.
— The models should be corrected by means of the comparison of their predictions with observations so that any necessary adjustments can be introduced for their use in future predictions.

Dynamic analysis

4.47. A conservative assessment of differential and total settlement should be performed for the design of the foundations for buildings, interconnecting structures between adjacent buildings and foundations for machinery.

4.48. If no structure–soil–structure interaction analysis was carried out, a soil–structure interaction analysis should be performed building by building and the individual displacements of the buildings should be combined to obtain the dynamic part of the differential displacement. Both horizontal and vertical components and their combinations should be considered.

4.49. For soft soil sites, the residual settlement after an earthquake should be assessed by the best available means.

EFFECTS OF INDUCED VIBRATIONS

4.50. Foundations for structures subjected to vibrations or with vibration loads should be designed to ensure that vibrations would not cause excessive settlement. For this purpose, precautions should be taken to ensure that resonance would not occur between the frequency of the pulsating load and a critical frequency in the foundation–ground system. If these precautions are not relevant, the vibration source should be isolated from the supporting structure and from the soil by means of springs or systems of springs and dampers.

5. EARTH STRUCTURES

GENERAL CONCEPT

5.1. The design of earth structures and buried structures that are relevant to the safety of the nuclear power plant should be consistent with the design of the plant itself. In particular, the design of the plant against external hazards should be in accordance with the events that are selected in the design; these events, and the associated loads, should be listed in the contractual terms of reference relating to the earth structures or the buried structures; and the list of events should be supplemented by specific events, if any, that could challenge the safety of these structures. For instance:

— *With regard to consistency:* The level of seismic safety attained through the design of safety related dykes and dams should be consistent with that of the main facilities of the nuclear power plant;
— *With regard to specific events:* In relation to the stability of slopes, heavy rains should be considered, the return period of which should be consistent with those of the meteorological events selected for consideration in the design of the plant.

NATURAL SLOPES

5.2. The stability of natural slopes surrounding the important facilities of a nuclear power plant should be investigated with regard to the safety of the plant. The safety evaluation will depend largely on the separation distance and the features of a slope. If a slope is judged to be distant enough from important facilities that its failed debris would never reach safety related structures, no countermeasure would be necessary. Potentially hazardous slopes should therefore be differentiated in terms of such factors as the distance, the slope angle, the height, the geology, and the water content and other geotechnical conditions of the material of the slope.

5.3. The external effects of earthquakes and of heavy rainfalls should be considered in the safety evaluation for assessing the potential hazards of natural slopes.

5.4. If a slope is judged to be potentially hazardous, a stability analysis should be performed by some appropriate means. A conventional sliding surface analysis is usually performed to evaluate a safety factor for sliding failure.

5.5. The seismic effect is usually considered as an equivalent static inertia force by means of a seismic coefficient. To evaluate the equivalent static force, the seismic amplification in the slope should be reproduced if necessary. The peak ground acceleration should be used in the estimate of inertial forces; however, some lower value, if justified by an additional study, may be used instead. The safety factor should be equal to or more than 1.5. If the safety factor is not large enough, a dynamic response analysis should be performed on the basis of a design seismic motion. If necessary, the residual deformation should be evaluated to judge the ultimate safety in cases for which the safety factor is close to unity.

5.6. If the safety factor thus evaluated is low enough to indicate a potential for a major sliding failure, suitable countermeasures for stabilizing and strengthening the slope or for preventing any debris from reaching the safety related plant structures should be designed and implemented. Otherwise, the plot plan of the plant site should be altered.

DYKES AND DAMS

5.7. The term dyke should be used for structures running along water courses and the term earth dam should be used only for a structure higher than 15 m, which is, in some cases, necessary to create a water reservoir upstream from a nuclear power plant. For designing dams and dykes appropriately, reference should be made to appropriate design manuals.

5.8. Before construction, in addition to classical geophysical and geotechnical tests, special attention should be paid to the permeability of the site close to the areas of the foundations. This permeability should be monitored throughout the operating lifetime of the plant.

5.9. In addition to the usual failure modes, consideration should also be given in the design of these earth structures to all the possible failure modes that are dependent on the following two parameters: the pore pressure inside the embankment and the internal erosion that is caused by water flows inside the embankment.

5.10. The design requirements for dykes and dams, in relation to the consequences of their failure for plant safety (e.g. the loss of cooling water for the plant), should be consistent with the design requirements for the plant itself, especially for the evaluation of natural hazards (earthquake, rainfall or the return period for flooding).

5.11. In addition to the usual methods of engineering design, a specific analysis should be performed to compute the relevant parameters of the structures (e.g. displacements, pore pressures), the values of which should be compared with those measured in situ at the different stages of construction.

5.12. Surveillance (periodic inspection), the monitoring of dams and dykes, and maintenance work should be permanent during construction and during operation to prevent possible damage such as the internal erosion of dykes.

SEA WALLS, BREAKWATERS AND REVETMENTS

5.13. Sea walls, breakwaters and revetments are civil engineering structures for protecting important facilities of a nuclear power plant against the wave action of an ocean or a lake during storms and tsunamis. These structures should be properly designed to prevent soil erosion, flooding and structural failures which may jeopardize the safety of important facilities.

5.14. The external effects of waves, tsunamis and earthquakes should be considered in assessing the potential failures of sea walls, breakwaters and revetments. The dynamic effects of waves should be evaluated with account taken of the maximum static water level derived from flood hazard evaluation, as described in Ref. [5].

5.15. The stability of sea walls, breakwaters and revetments should be properly evaluated in relation to the sustainability of the above mentioned protective functions as well as the effects of their possible failure. The methods of evaluation are similar to those for the sliding failure of slopes as previously mentioned. In performing this evaluation, the material properties of sea walls, breakwaters, revetments and backfill materials, which may include concrete blocks, rubble and other large pieces, should be properly estimated. Sandy soils, whose potential for liquefaction may need to be evaluated, may be encountered at the foot of these structures.

5.16. The consequences of the failure of these structures (owing to their side effects) for safety related ducts, pipes and other underground facilities passing near or through the facilities of the nuclear power plant should be given appropriate consideration. If hazardous effects are expected, appropriate countermeasures should be taken to protect the facility or otherwise the site layout should be reconsidered.

6. BURIED STRUCTURES

RETAINING WALLS

6.1. Retaining walls can be classified into two groups:

— Gravity walls in which the weight of the wall and possibly that of the retained soil play an important part in its stability;
— Embedded walls, such as sheet walls, the stability of which depends on the passive pressure of soil and/or on anchors.

Frequently, a retaining wall is a combination of both types.

6.2. The input parameters are similar to those introduced for assessing the stability of foundations, generally supplemented by geometrical data for the soil behind the retaining wall, particularly the slope of the surface. Special care should be taken in determining the level of the water table. Data should be provided for soil to a depth consistent with the analyses that are carried out for assessing stability.

6.3. For the assessment of stability, the pressure of the earth behind the wall may be the active pressure. If some requirement limits the admissible displacement of the wall, the pressure of the earth should be the at-rest pressure.

6.4. The active pressure of the Earth due to earthquakes should be evaluated by means of considering an artificial gravity inclined in the unfavourable direction. The vertical component of the seismic acceleration should be considered as acting upward or downward. The passive pressure of the earth is likewise considered so as to produce the more unfavourable effects.

6.5. In stability analysis, the failure modes that involve sliding surfaces as well as the failure modes that involve the retaining capacity of the wall should be addressed. The associated safety factors are, respectively, those of the natural slopes and those of the bearing capacities of the foundations.

6.6. It should be ensured that the soil behind the foundation is not susceptible to liquefaction under SL-2 earthquake conditions (see footnote 1).

EMBEDDED STRUCTURES

6.7. Embedded structures are buildings with foundations deep enough that the interaction of the underground walls with the surrounding ground is significant. Two consequences of such embedment should be taken into account:

— The underground walls act as retaining walls; this point has already been covered.
— This situation has consequences for the building itself, which are considered in this section.

6.8. The input parameters for the assessment of embedded structures are similar to those for foundations and retaining walls, and information on them should be obtained accordingly. Supplementary information should be obtained on the safety and serviceability requirements for the underground walls, particularly in relation to leaktightness, that have to be met under different loading cases. For this purpose, the possible cracking of concrete (and thus the need to limit the stresses in reinforced bars and concrete) should be taken into account in the design of the foundation and special attention should be paid to the design of the construction joints of buildings. For further consideration of the containment, see Ref. [6].

6.9. The challenging effects of groundwater on both the stability and the leaktightness of embedded structures should be taken into account in the design. In any case, drainage should be incorporated for any foundation beneath the level of the water table, or alternatively the hydrostatic pressure should be taken into account. At coastal sites, the possible adverse effects of varying levels of groundwater salinity on the foundation material and isolation material should be considered.

6.10. A building can be regarded as embedded only if the backfill has been properly compacted or if other appropriate measures have been taken. In such a case, the effects of embedment on the impedance of the foundation and on the soil–structure interaction should be taken into account. If the building is not mechanically embedded, only the consequences of the depth of the foundation should be taken into account, disregarding interaction effects of the soil with the underground walls.

6.11. Recommendations on the stability analysis of a building are given in Section 4 on foundations. Even for mechanically embedded foundations, friction between soil and walls should be disregarded for the stability analysis under seismic loads.

BURIED PIPES, CONDUITS AND TUNNELS

Site investigation programme

6.12. The layout of buried pipes or conduits should be considered in the site investigation programme. Adequately spaced boreholes and/or test pits should be dug along the pipe route. Special consideration should be given to identifying areas of discontinuities or changes in the foundation material along the route of the pipe.

6.13. The investigation boreholes or test pits should be dug to a depth that will depend on the stratigraphy of the foundation material below the pipe, but they should extend to a competent soil layer below the foundation level structure.

6.14. An assessment of the potential effects of any corrosive environmental agents on the piping material should be included in the site investigation programme.

Considerations of construction

6.15. Buried piping should be placed at a depth sufficient to prevent damage due to surface loading (e.g. traffic loads), or alternatively should be designed to resist the surface loads that are expected to act on the pipes.

6.16. The piping should be placed on well compacted granular material over competent foundation material such that no damage or distortion of the piping, due to settlement or to liquefaction of the foundation material, can occur. Techniques of foundation improvement may be used for weak subsurface conditions.

Considerations of design

6.17. Safety related buried systems and tunnels should be designed to resist the effects of earthquakes.

6.18. Long, buried piping systems are primarily subjected to relative displacement induced strains rather than inertial effects. These strains are induced primarily by the passage of seismic waves and by differential displacement between a building attachment point (anchor point) and the ground surrounding the buried piping. The following seismic induced loadings should be considered for long buried piping, conduits and tunnels:

— Strains induced by the passage of seismic waves;
— Differential displacements in zones of different materials;
— Deformation and shaking of the ground or anchor points relative to the ground;
— Ground failures such as liquefaction, landslides and settlements.

Considerations of analysis

6.19. In the analysis of the effects on the piping system due to earthquake ground shaking, the following two types of loading should be considered:

— Relative deformations imposed by seismic waves travelling through the surrounding soil or by differential deformations between the soil and anchor points;
— Lateral earth pressures acting on the cross-section of the structural element.

6.20. Unless it is otherwise justified, it may be assumed that sections of a long, linear buried pipe remote from anchor points, sharp bends or intersections move with the surrounding soil and that there is no movement of the buried structure relative to the surrounding soil. In this case the maximum axial strain can be estimated by ignoring friction between the piping and the surrounding soil. If there is a possibility of slippage between the pipe and the surrounding soil, the axial strain for straight sections remote from anchor points, sharp bends or intersections should be estimated with account taken of the friction.

6.21. An estimate of these axial strains will depend on the wave type that results in the maximum ground differential displacements. The wave types that should be considered are compression waves, shear waves and surface waves.

6.22. In addition to computing the forces and strains in the buried pipes due to wave propagation effects, the forces and strains due to the maximum relative movement between anchor points (such as a building attachment point) and the adjacent soil, which occurs as a result of the dynamic response of the anchor point, should also be calculated. In calculating maximum forces and strains in the buried piping, the motion of adjacent anchor points should be considered in a conservative manner.

6.23. In the analysis of tunnels the stresses and deformations due to all expected loads, including earthquake motions, should be considered. Stresses can be assessed empirically or numerically such as by the finite element method.

6.24. For deep tunnels and shafts, hoop stresses and strains will also develop owing to travelling seismic waves, and these hoop strains should be considered in the design.

7. MONITORING OF GEOTECHNICAL PARAMETERS

PURPOSE OF MONITORING GEOTECHNICAL PARAMETERS

7.1. Subsurface exploration, in situ testing and laboratory testing should provide values of parameters and information on site characteristics suitable for predicting the performance of foundation systems under the envisaged loading conditions. The use of these parameters allows criteria for foundation design to be established for the performance of the foundation materials and structures under anticipated loadings. In order to verify the performance of the foundation and earth structures, their actual field behaviour should be monitored from the beginning of siting activities through construction to operation.

7.2. The monitoring of actual loads and deformations permits a field check to be made of the predicted behaviour of the foundations and earth structures. Since the construction sequence is generally over the long term, the monitoring data allow the settlement models to be revised on the basis of actual performance. Predictions of long term performance can therefore be made with reasonable confidence.

GUIDELINES FOR MONITORING

7.3. Construction phases usually consist of excavation, backfilling and building construction. The behaviour of the soil should be monitored during these phases. During the excavation and backfilling phase, deformation of subsurface material (heave and settlement, lateral displacements) should be monitored and load evaluations should be made. Monitoring should be continued throughout the lifetime of the plant.

7.4. The groundwater regime under buildings and in adjoining areas should be monitored to verify the conditions outlined in the design assumptions, especially if deep drainage systems or permanent dewatering systems are installed.

7.5. Deflection and displacements and relevant parameters of safety related structures, including retaining structures and earth structures, should be monitored.

7.6. The seismic behaviour of the site and the subsurface material should be monitored. The need for instrumentation to monitor the in situ pressure of pore water for liquefaction studies should also be considered.

7.7. The monitoring devices should be carefully chosen so that the monitoring system provides the expected information for the lifetime of the installation. The choice of devices should be informed by the feedback of experience. In deciding on the number of devices to be used, their expected failure rate should be taken into account.

MONITORING DEVICES

7.8. The following monitoring devices should be used to observe the behaviour of the foundation and related materials (Table 4). Other devices can be used for monitoring soil and buildings (e.g. extensometers, load and pressure cells), depending on the particularities of the site, the requirements and the type of plant.

TABLE 4. MONITORING DEVICES

Type of device	Principle	Location	Parameter measured	Purpose
Piezometers	Hydraulic pressure	Bore holes	Pore pressure, water table	Monitoring of water table
Global positioning system	Aiming by satellite	Site	Topography of the site	Site evaluation
Settlement monuments	Topographic aiming	Ground surface	Displacements, settlements	Settlement of structures
Gammagraphy, photogrammetry	Superposition of picture	Ground surface	Deformation of topography	Deformation of structures
In situ settlement plates	Topography	Ground surface	Displacements	Settlement of structures
Inclinometers, tiltmeters	Mechanic	Bore-holes	Verticality	Stability of slopes
Seismometers	Accelerometers, triggers	Free field, buildings	Accelerations time histories	Operability of plants; seismic behaviour of structures; floor response spectra.
Hydraulic devices	Hydraulic U-tube, Glötzl cells	On basemat and beneath	Deformations and stresses of the basemat	Behaviour of the soil–structure system

REFERENCES

[1] INTERNATIONAL ATOMIC ENERGY AGENCY, Seismic Design and Qualification for Nuclear Power Plants, Safety Standards Series No. NS-G-1.6, IAEA, Vienna (2003).

[2] INTERNATIONAL ATOMIC ENERGY AGENCY, Evaluation of Seismic Hazards for Nuclear Power Plants, Safety Standards Series No. NS-G-3.3, IAEA, Vienna (2002).

[3] INTERNATIONAL ATOMIC ENERGY AGENCY, Site Evaluation for Nuclear Installations, Safety Standards Series No. NS-R-3, IAEA, Vienna (2003).

[4] INTERNATIONAL ATOMIC ENERGY AGENCY, Dispersion of Radioactive Material in Air and Water and Consideration of Population Distribution in Site Evaluation for Nuclear Power Plants, Safety Standards Series No. NS-G-3.2, IAEA, Vienna (2002).

[5] INTERNATIONAL ATOMIC ENERGY AGENCY, Flood Hazard for Nuclear Power Plants on Coastal and River Sites, Safety Standards Series No. NS-G-3.5, IAEA, Vienna (2003).

[6] INTERNATIONAL ATOMIC ENERGY AGENCY, Design of Reactor Containment Systems for Nuclear Power Plants, Safety Standards Series No. NS-G-1.10, IAEA, Vienna (2004).

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