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Geotechnical Aspects in Siting and Design of Nuclear Installations

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CONTENTS

1. INTRODUCTION

BACKGROUND

1.1. Requirements for site evaluation for nuclear installations are established in IAEA Safety Standards Series No. SSR-1, Site Evaluation for Nuclear Installations [1]. This Safety Guide provides recommendations on geotechnical characteristics and the evaluation of geotechnical hazards as part of such site evaluation.

1.2. Seismic aspects also play an important role in this field, and relevant recommendations are provided in IAEA Safety Standards Series No. SSG-9 (Rev. 1), Seismic Hazards in Site Evaluation for Nuclear Installations [2].

1.3. This Safety Guide supersedes IAEA Safety Series No. NS-G-3.6, Geotechnical Aspects of Site Evaluation and Foundations for Nuclear Power Plants[1]. The revision ensures consistency with the requirements established in SSR-1 [1], while incorporating the latest knowledge, experience, and lessons learned from significant geotechnical events in Member States. This Safety Guide explicitly expands the scope to include nuclear installations other than large nuclear power plants and presents recommendations for applying a graded approach to geotechnical site investigations and activities for other types of nuclear installation.

OBJECTIVE

1.4. The objective of this Safety Guide is to provide recommendations on dealing with geotechnical engineering aspects that are important to the safety of nuclear installations, such as site investigation planning, evaluation of geotechnical hazards, considerations for design and analyses, monitoring of geotechnical parameters, and the application of a graded approach to geotechnical evaluations for nuclear installations with limited risk. These recommendations are intended to meet the requirements established in SSR-1 [1], in particular Requirements 21 and 22.

1.5. This Safety Guide is intended for use by operating organizations, licensees and regulatory bodies involved in the licensing of nuclear installations as well as designers and technical support organizations of such installations.

SCOPE

1.6. In this Safety Guide, 'geotechnical aspects' refer to those aspects of geotechnical site investigation, evaluation, engineering design and safety assessment related to the subsurface materials

[1] INTERNATIONAL ATOMIC ENERGY AGENCY, Geotechnical Aspects of Site Evaluation and Foundations for Nuclear Power Plants, IAEA Safety Standards Series No. NS-G-3.6, IAEA, Vienna (2004).

at nuclear installation sites.

1.7. This Safety Guide provides recommendations on the geotechnical aspects necessary for the establishment of parameters used in the site evaluation and the development of the design basis for nuclear installations. It covers the programme of site investigation that should be performed to obtain appropriate understanding of the subsurface conditions, which is necessary for determining whether the conditions are suitable for foundation and construction of a nuclear installation. It provides recommendations specific to the characteristics of the geotechnical profiles (foundation ground types) and the parameters that are suitable for use in performing the geotechnical analyses for the design of a nuclear installation. It also addresses the approach to monitoring of geotechnical parameters, the application of a graded approach and the application of a management system.

1.8. This Safety Guide provides recommendations on the methods of analysis appropriate for the safety assessment of a site for a nuclear installation, particularly for the assessment of the effects of an earthquake on site, including the determination of site specific response spectra and estimation of the liquefaction potential. This Safety Guide also provides recommendations on methods of analysis for the safety assessment of the effects of static and dynamic interaction between soil and structures, and of the consequences on the soil bearing capacity and for settlements. A more detailed description of methods for the analysis of soil structure interaction is given in SSG-9 (Rev. 1) [2]. In this Safety Guide only the site dependent information and the methods of analysis are addressed.

1.9. This Safety Guide also considers foundation works, including consequences for the geotechnical profiles and parameters, the possible improvement techniques of foundation material and the appropriate choice of the foundation system in accordance with the soil capacity. Earth structures, natural slopes and buried structures, the safety of which need to be assessed in the site safety assessment are also considered. The Safety Guide provides recommendations on appropriate methods for the analysis of the behaviour of such structures under static and dynamic loads.

1.10. This Safety Guide provides recommendations on methodologies for the development of the design basis of nuclear installations. The collected data and interpreted information from site investigations, (considering their variability and the analysis methodologies described in this Safety Guide) are appropriate for use in the evaluation of structural response to both design basis and beyond design basis events. The acceptance criteria for the assessment of beyond design basis external events may be relaxed provided they are consistent with the provisions for beyond design basis external hazards described in IAEA Safety Standard Series Nos SSG-67, Seismic Design for Nuclear Installations [3], and SSG-68, Design of Nuclear Installations Against External Events Excluding Earthquakes [4]. Furthermore, these evaluations need to consider the potential for cliff-edge effects and provide adequate margin to protect items ultimately necessary to prevent an early radioactive release or a large radioactive release.

STRUCTURE

1.11. Section 2 provides recommendations on geotechnical site investigation, addressing different stages of the programme, sources of data, special considerations for investigation of complex subsurface conditions, and site considerations for nuclear installations. Section 3 provides recommendations on geotechnical hazards, including undesirable soil conditions, natural slopes and liquefaction. Section 4 provides recommendations on the considerations for design and evaluation of dykes and dams, sea and retaining walls, foundations, earth and buried structures, embedded structures and buried pipes, conduits and tunnels. Section 5 provides recommendations on monitoring of geotechnical parameters. Section 6 provides recommendations on applying a graded approach to geotechnical aspects for nuclear installations other than nuclear power plants. Section 7 provides recommendations on the application of a management system, with a focus on quality management for geotechnical investigations, testing, verification, record keeping and monitoring.

2. GEOTECHNICAL SITE CONSIDERATIONS FOR NUCLEAR INSTALLATIONS

GEOTECHNICAL INVESTIGATION PROGRAMME FOR SITING OF NUCLEAR INSTALLATIONS

2.1. Requirement 21 of SSR-1 [1] states:

> "**The geotechnical characteristics and geological features of subsurface materials shall be investigated, and a soil and rock profile for the site that considers the variability and uncertainty in subsurface materials shall be derived.**"

2.2. Investigations of the subsurface conditions at potential sites for a nuclear installation should be performed at all stages of the site evaluation process (see paras 2.7–2.26). The purpose of such investigation is to obtain information and basic data on the physical and mechanical properties of the subsurface materials, to be used when making decisions about the suitability of the site for a nuclear installation.

2.3. The geotechnical investigation programme for a nuclear installation should provide the data necessary for an appropriate characterization of the subsurface at each stage of the site evaluation. The various methods of investigation - that is, the use of current and historical documents, geophysical and geotechnical exploration, in situ and laboratory testing - are typically applicable to all stages of the site evaluation process, but will vary from stage to stage, as necessary. In general, the investigations should become more detailed in character when approaching the later stages of the investigation programme. Furthermore, some analysis specific considerations may apply only to datasets used as input data in soil and rock characterization and analysis.

2.4. The long term impact of investigative drilling on the geological environment and aquifers should be considered. Relevant precautions should be taken to minimize the long term impact. All boreholes not needed for monitoring purposes (see Section 5) should be filled and sealed with suitable materials.

2.5. Generally, data related to geophysical, geological, geotechnical and engineering information should be collected for use in safety evaluations or analyses. The data is typically grouped as follows:

(a) Composition of the subsurface (rock and soil types);
(b) Characterization of soil and rock (in terms of physical, chemical and geomechanical properties), including applicable classifications (such as those used in engineering geology);
(c) Spatial information about the continuity, extent and geometrical arrangement of the subsurface materials (stratigraphy and geologic structure geometry);
(d) Spatial information and properties about discontinuities and/or other features in the subsurface (e.g. faults, fracture zones, cavities) that could affect the suitability (e.g. in terms of mechanical stability or hydrogeology) of the site, including applicable classifications (e.g. those used in engineering geology);
(e) Hydrogeological, hydrological, and hydrochemical information (e.g. groundwater regime, hydrostratigraphical and hydrogeological model, groundwater table, groundwater chemicals, quality of the groundwater, connections between groundwater and surface water).

2.6. The results of the investigations should be clearly documented with reference to the particular site conditions (e.g. soil or rock), the stage of the site evaluation process concerned, and the verification analysis needed. The detail of this documentation should be sufficient to support the safety justification, evaluations, analyses and to support independent peer reviews and review and assessment by the regulatory body.

Selection stage

2.7. The purpose of an investigation at the site selection stage should be to determine the preliminary suitability of sites (see para 2.3 of SSG-35 [5]). In this stage, geological, geomorphological, geotechnical and hydrogeological aspects are considered, and some regions or areas may be excluded from further consideration. Subsurface information at this stage is usually obtained from current and historical documents (see paras 2.30 and 2.31) and by means of field reconnaissance, including geological, geophysical and geomorphological surveys (see para. 2.32), and this information is used in the following considerations:

(a) Unacceptable subsurface conditions. A site with geological conditions that could challenge the safety of a nuclear installation and that cannot be corrected by means of geotechnical treatment or compensated for by construction or design measures is unacceptable, and, consequently, such conditions are considered as exclusion criteria. The potential for geotechnical hazards associated with faulting, ground motion, uneven bedrock movements, liquefaction, flooding, volcanic activity, landslides, permafrost, swelling, erosion processes, subsidence and collapse due to underground cavities (both natural and those deriving from human activities) or other causes is required to be identified and evaluated in accordance with Requirements 21 and 22 of SSR-1 [1].

The scope and extent of the investigation should be sufficient to estimate the hazard under consideration with a level of uncertainty that can enable the application of the relevant exclusion criteria.

(b) Classification of the site. The site should be classified for the purposes of seismic response analysis, using the seismic velocities ($V_{s,30}$) as criteria (see para. 2.43). If such site classification is not applicable, the subsurface conditions at a site can be derived from geological and geotechnical literature, and the site may be classified into one of three main categories: a rock site, a soil site or a combination of rock and soil. If applicable, the hardness (soft, medium, or hard) of the rock at a rock site should be further classified. If applicable, the stiffness (soft, medium, or stiff) of the soil at a soil site should be further classified. However, this rough classification might not apply to certain sites. For instance, quaternary formations or intensive bedrock fracturing and alteration may introduce complex interfaces and ambiguity in defining the contacts between the different subsurface materials.

(c) Groundwater regime. If there is a lack of detailed data, at this stage the hydrogeological literature may allow a preliminary estimation of presence and level of groundwater, potential groundwater–surface water interactions and the groundwater regime (see para. 5.26 of SSR-1 [1] and IAEA Safety Standards Series No. NS-G-3.2, Dispersion of Radioactive Material in Air and Water and Consideration of Population Distribution in Site Evaluation for Nuclear Power Plants [6]).

(d) Foundation conditions. The type of soil and/or bedrock, their lateral extent and the depth to bedrock or load bearing stratum and the properties of the bedrock and/or soil should be determined, as a minimum set of information. This enables the preliminary selection of suitable foundation types.

2.8. On the basis of the above mentioned information on subsurface conditions, candidate sites can be ranked in accordance with the suitability of foundation works. In addition to the assessment of the potential geotechnical hazards (see para. 2.7), inferences can be made about seismic amplification effects, bearing capacity, potential settlement and swelling, and soil–structure interactions. After this stage, sites with unacceptable subsurface conditions for which there are no generally practicable engineering solutions should be excluded, and sites with acceptable subsurface conditions would be retained for further consideration.

Characterization stage: Verification

2.9. In the verification stage, it is assumed that the generalized layout and foundation loads are established and the primary geotechnical and geological characteristics of the site are known (based on the site selection stage investigations). In addition to the features stated in para. 2.5, the following factors should be considered in the evaluation, to account for both normal conditions, geohazards and other extreme conditions:

(a) Spatial information about continuity, extent and geometrical arrangement of the subsurface materials and discontinuities (stratigraphy and geologic structure), with reference to the site layout;

(b) Identification of other undesirable subsurface characteristics (see paras 2.27–2.28 and Section 3), such as cavity zones, swelling rocks and shales, collapsing soils or soluble rocks, the occurrence of gas pockets, and potential displacement planes determined by unstable or mechanically weak subsurface layers;

(c) Liquefaction potential;

(d) Feasible foundation types;

(e) Preliminary bearing capacity and other factors of foundation stability;

(f) Preliminary settlement ranges;

(g) Shoring needs for deep excavations;

(h) Dewatering requirements;

(i) Excavation difficulty;

(j) Prior use of the site;

(k) Site preparation requirements.

2.10. In the verification stage, the investigation programme should cover the site as a whole, but should also be conducted on a smaller scale appropriate for the layout of the nuclear installation. The investigation programme should take into account site characteristics (e.g. compositional and structural heterogeneity within the subsurface materials) and their variability, available from the earlier stages of investigation, and the overall planned layout. The site geotechnical investigation phase should be carefully planned to ensure that it is structured, complete and sufficient to satisfy all stakeholders' expectations and to address any uncertainties. The following site investigation techniques and related points should be considered:

(a) Geophysical investigations, such as seismic refraction and/or reflection surveys. These should be conducted to provide continuous lateral and depth information for the evaluation of subsurface conditions. Geological constraints should be considered in interpretation of the survey results. The results should provide stratigraphic and structural geological information, information on the location of the groundwater table, and an estimate of wave velocities at the site. The geophysical investigations should be designed to optimally reflect the site characteristics and their spatial variability; drilling, coring and sounding should be used to complement the subsurface geophysical data (e.g. stratigraphic information) as well as to constrain and validate the interpretations of the geophysical datasets.

(b) Rotary borehole drilling, coring or sounding. These techniques are used to define the overall site conditions, and to collect basic information about the subsurface materials. The method selected should be justifiable by the site conditions. Borehole drilling and coring involve extraction of cores or other samples for rock or soil qualification and laboratory testing. Sounding measures the resistance offered by the soil and is used for determination of the soil profile. The recovered information typically includes rock and/or soil units and their stratigraphic order, the attitude and shape of the boundaries between the subsurface units (bedding, contact), the depth of the bedrock or load bearing stratum, and the presence and attitude of the structural elements (bedding, foliation, fractures, faults) within the subsurface materials. The investigations should be conducted along at least two intersecting lines that are oriented to capture the expected variation within the subsurface and have a common investigation hole at the line intersection. These investigations should be used to determine and map the soil profiles. Borehole numbers and depths should be sufficient to verify the site suitability with no unacceptable subsurface conditions. In addition to the extraction of cores or other samples for rock or soil examination and laboratory testing, the investigation holes can be used for the installation of instruments for long term in situ testing, stress monitoring, and monitoring the groundwater regime. The possible effects of boreholes on the potable water regime should be investigated [6]. If necessary, test pits or test tunnels should be used to facilitate a direct examination of the subsurface conditions.

(c) In situ testing. In accordance with the subsurface conditions, in situ tests should be performed to measure the mechanical properties of the foundation materials. These tests should include in situ loading tests and piezometric measurements of the groundwater.

(d) Laboratory testing. Laboratory testing consisting of index and classification tests sufficient to characterize the geomechanical properties of the strata should be conducted on rocks or soils. For cohesive and granular soil samples obtained during the drilling/coring operation, appropriate consolidation and shear strength testing should be conducted on the undisturbed samples (see para. 2.35) to allow the estimation of soil strength, stiffness, stress-strain responses and consolidation properties.

2.11. 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. Therefore, the preliminary characteristics of the nuclear installation, such as loads, physical dimensions of the buildings, preliminary structural engineering criteria and the preferred plant layouts should be known at the beginning of the confirmation stage.

Characterization stage: Confirmation

2.12. The purpose of the site confirmation stage is to confirm the results obtained in the previous stages and ensure that the spatial and thematic coverage of the site characterization data and interpretations is sufficient for the purposes of final layout planning. The results of the site confirmation stage should address geotechnical parameter variability and uncertainty and provide sufficient geotechnical parameters for detailed design of the nuclear installation and its safety assessment.

2.13. The content of the site characterization, in situ testing and laboratory testing programmes conducted in the confirmation stage should be planned on the basis of both the preliminary characteristics of the nuclear installation and the geotechnical characteristics of the site as identified in the previous stage. The plan should reflect the necessary information needed for detailed design of the installation. It is advisable that data validation and necessary validation is undertaken timely, to enable additional or repeat testing if it is deemed necessary. The results of these investigations should be used in evaluating the suitability of the preliminary layout and modifying it, as necessary. If planned layouts are changed and new locations are chosen additional testing and investigations should be performed if necessary. The final confirmations should be consistent with the known geotechnical characteristics of the site and the final layout of buildings on the site, including the final safety classification of buildings (see para. 2.23).

2.14. In addition to refining the investigations conducted in the earlier stages (see paras 2.5, 2.7 and 2.9), the investigations should address the following:

(a) Detailed scrutiny of the potential for undesirable subsurface conditions such as cavities, fracture systems and faults (see Section 3);
(b) A revised estimation of the bearing capacity of the soil and bedrock underlying the nuclear installation;
(c) A determination of the settlement of structures and the site amplification of seismic waves;
(d) Establishment of soil and soil–structure interaction parameters (dynamic and static);
(e) Engineering assessments of the liquefaction triggering and consequences;
(f) Evaluation of a site specific design response spectrum (if needed).

2.15. A subsurface investigation and laboratory testing programme should be conducted at the site using a drilling scheme that is suited to the planned layout of the nuclear installation, in order to adequately characterize the geotechnical conditions of the site. At sites of relatively uniform soil and bedrock conditions, a uniform grid method can be applied. In other cases, the grid spacing and orientation should be defined based on the extent, heterogeneity and geologic structure of the subsurface units and discontinuities. Where heterogeneity and discontinuities are present, the usual investigation process should be supplemented with investigation holes at adequate spacings and depths to permit detection of the geological and geotechnical features and their proper evaluation.

2.16. It may also be necessary to include complementary drilling in the investigation programme to either establish the soil model for studies of dynamic soil–rock structure interactions, or to further delimit any undesirable subsurface conditions (see paras 2.27–2.28).

2.17. In the confirmation stage, the subsurface investigation campaign should include sufficient in situ and laboratory testing to address the goals defined in para. 2.14.

2.18. The necessary drilling depths depend on site conditions: drilling should be deep enough to allow the site conditions that would affect the structures, systems and components of the nuclear installation to be fully ascertained, and to confirm the soil and rock conditions determined in previous investigations.

2.19. For sites characterized by very thick soils, drilling should enable the evaluation of potential deep instability at the site, and the potential effects associated with sloping sites.

2.20. If competent rock is exposed on the surface or encountered at a shallow depth, drilling should, at a minimum, penetrate to the greatest depth at which discontinuities or zones of weakness or alteration could affect the stability of the foundation. If such a depth cannot be unequivocally determined (e.g. due to a large depth continuity of steeply dipping weakness zones), drilling should enable the discontinuities or zones of weakness or alteration to be adequately characterized so that justified evaluations of their significance for the nuclear installation can be made.

2.21. For sites of weathered shale or soft rock, drilling may need to penetrate deeper than that needed for the normal purposes of geotechnical design in order to facilitate site amplification studies.

2.22. The consequences of drilling on the groundwater regime, and possibly on potable water, should be considered.

2.23. The distinction between the structures, systems and components important to safety and other items should be considered when defining the detail of the site investigations. The subsurface investigation and testing programme for structures that are not safety related should follow relevant local, national or international codes and standards and with proven engineering practices. Depending on the site characteristics, drillings may be necessary at the planned location of buildings not important to safety. At least one investigation hole should be drilled at the planned location of every safety related structure[2]. Where conditions are found to be variable, the number and spacing of drillings should be chosen to obtain a clear definition of changes in soil and rock properties.

Pre-operational stage

2.24. Geotechnical investigations, studies and monitoring should be continued after the start of construction of the nuclear installation and until the start of operation of the installation to complete and refine the assessment of site characteristics by incorporating geological and geotechnical data that are newly obtained during the excavation and construction of the foundations. As subsurface material is exposed during and after foundation excavations, it should be carefully observed and mapped for comparison with the assumed design conditions and confirmed with the design itself. Deformation features (e.g. faults, potential soft zones or soft interbeds in rocks, lateral compositional changes, materials susceptible to volume change, other features of engineering significance) discovered during construction should be carefully assessed to ensure the safety objectives are not compromised[3]. If necessary, in situ tests may also be performed in the base of the excavation. The existing ground model should be validated and verified or should be revised to reflect any new information.

2.25. 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

[2] Some Member States define a minimum of 3 investigation holes for every safety related structure [7].
[3] Additional information about the significance of such findings can be found in Ref. [8].

generally long, these monitoring data should be used to revise the settlement models and the soil properties on the basis of actual performance, if needed.

Operational stage

2.26. Selected geotechnical investigations and monitoring of geotechnical parameters are pursued over the lifetime of the installation to confirm the conditions, to demonstrate the continued validity of the design basis, safety assessment and periodic reviews and to potentially support future reassessment if necessary. During the operation of a nuclear installation, the settlement of structures, displacements, and deformation of foundations and associated safety related items, as well as parameters such as the level of the water table and its seasonal fluctuations, should be monitored 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 the site evaluation in the operational stage should be described in a maintenance and monitoring programme and assessed as part of the periodic safety review. Recommendations on this stage are provided in Section 5.

Investigations for undesirable subsurface conditions

2.27. The geotechnical site investigation programme for a nuclear installation should consider the potential presence of particularly undesirable subsurface conditions, i.e. which could have serious implications for the integrity of the foundation of the installation due to ground instability and/or collapse, bedrock block movements and changes in groundwater conditions. In investigating such undesirable subsurface conditions, the following should be considered:

(a) Potential cavities and susceptibility to ground collapse:
   - (i) Underground void spaces, of either natural or artificial origin;
   - (ii) Sinkholes and open joints that give rise to hazardous effects of other types such as piping and seepage;
   - (iii) Sinks, sink ponds, caves, cavity zones and caverns;
   - (iv) Gas pockets;
   - (v) Evidence of solution or karstic phenomena;
   - (vi) Sinking streams;
   - (vii) Historical ground subsidence;
   - (viii) Mines and signs of associated activities;
   - (ix) Natural bridges;
   - (x) Surface depressions;
   - (xi) Springs;
   - (xii) Rocks, soil types or minerals characterized by mechanical weakness and/or tendency towards dissolution or collapse, such as limestone, dolomite, gypsum, anhydrite, halite, terra rossa soils, lavas, weakly cemented clastic rocks, coal or ores;
   - (xiii) Non-conformities in soluble rocks;

(xiv) Altered bedrock.

(b) Features causing additional bedrock instability:

(i) Swelling rocks and shales;

(ii) Potential displacement planes determined by unstable or mechanically weak subsurface layers;

(iii) Faults and fracture zones, and associated complex fracture systems.

2.28. The detection of most types of undesirable subsurface conditions is expected to result from the standard site characterization activities (see paras 2.1–2.28). However, the criteria for exploration, testing and analysis for some of the undesirable conditions might be difficult to specify to ensure that investigation programmes cover all abnormal subsurface conditions. For this reason, the recommendations in Section 3 of this Safety Guide should be followed to address any undesirable subsurface conditions for which the potential for their occurrence has been indicated during the standard site characterization. Investigation programmes for complex subsurface conditions should include prediction, detection evaluation and treatment.

SOURCES OF GEOTECHNICAL DATA FOR SITING OF NUCLEAR INSTALLATIONS

2.29. The purpose of the geotechnical investigations is to gather information to allow informed decisions to be made concerning the nature and suitability of the subsurface materials. The sources of data are as follows:

(a) Historical and current documents and data sets;

(b) In situ investigations and tests;

(c) Laboratory tests.

Historical and current documents and data sets

2.30. The geotechnical investigations will necessitate 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. The site review should include references to internationally acknowledged scientific literature within the corresponding discipline and ensure an adequate interpretation and evaluation of the available data. The appropriate documents used in site review may include the following:

(a) Geological reports and other relevant literature;

(b) Geotechnical reports and other relevant literature;

(c) Satellite imagery and aerial photographs;

(d) Digital elevation models (light detection and ranging (LiDAR) or other)

(e) Three dimensional models of the subsurface;

(f) Topographic maps;

(g) Geological maps and cross-sections, including soil and bedrock;

(h) Engineering geological maps and cross-sections;

(i) Geophysical maps and cross-sections;

(j) Hydrogeological maps, hydrological and tidal data, flood records, and climate and rainfall records;

(k) Water well reports and water supply reports;

(l) Oil and gas well records;

(m) Mining history, old mine plans and subsidence records;

(n) Indications for mineral resources, record of exploration history;

(o) Seismic observational (instrumental) data and historical earthquake and paleoseismic records, and relevant seismological studies;

(p) Contemporary accounts of landslides, floods, earthquakes, subsidence, slow bedrock movements and other geological events of significance;

(q) Records of the performance of structures and facilities in the vicinity.

2.31. Other possible sources of information should be also considered, such as observations, reports, publications, theses, and models available from 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 investigations and tests

2.32. Two types of test — geophysical tests and geotechnical tests — are available for soils and rocks. While both types of test should be performed, their extent can vary based on the scale and goal of the investigation.

2.33. Geophysical tests provide estimates of continuation and consistency of stratigraphy and they allow data or information to be derived by back analysis of the test results, but only in the domain of elastic deformation. These tests generally have a large spatial coverage (in terms of depth and surface area) and provide 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 tests should include some of the different techniques shown in Table 1, in accordance with best practices taking into account the subsurface conditions. Geophysical tests can be verified or complimented by the subsequent in-situ tests. Complimentary data sets may be combined to provide a robust characterisation and understanding of ground conditions.

2.34. Geotechnical tests address the near field area (to a depth of at least two times the shorter dimension of the structure's base or to a depth where the change in the vertical stress during or after construction due to applied loads are less than 10% of the effective in situ overburden stress). If competent rock is encountered at lesser depths, boring should penetrate to the greatest depth where discontinuities or zones of weakness or alteration can affect foundations and should penetrate at least 6 m into sound rock. The tests can be performed by many different techniques, such as using boreholes or working directly from ground level. A list of some techniques for geotechnical investigations of soil and rock samples are shown in Table 2. The appropriate tests should be selected, taking into account the subsurface conditions, and conducted. In some cases (e.g. when developing seismic site response characteristics), geotechnical testing of samples taken deeper in the soil profile is needed.

Laboratory tests

2.35. Laboratory testing should be conducted on samples obtained using methods of direct investigations. 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; therefore, sampling should be done in accordance with established procedures and practices with respect to quality requirements. 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, and the effects of this potential disturbance should be considered.

2.36. 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 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 implemented so that the properties of soil and rock can be realistically assessed with an uncertainty level compatible with the accuracy requested by the design assessment phase.

2.37. The testing programme should identify and classify soil and rock samples that adequately represent the geological and geotechnical composition and properties within, and their variation across the site. Their physical properties and engineering characteristics should be obtained from published data or by measurement. The laboratory tests should be conducted in conditions adequately representing the conditions of the site. A list of some techniques and their purposes is shown in Table 3.

2.38. Site characterization parameters for use in the design profile should be carefully derived from the results of in situ tests (see paras 2.32–2.34) and laboratory tests. Any discrepancies between the results of in situ tests and laboratory tests should be investigated and reconciled.

Reporting

2.39. The results of the geotechnical investigations and the resulting site characterization should be documented in a detailed geotechnical report. This report should be compiled at the end of the confirmation stage and updated during the pre-operational and operational stages. In some circumstances, such as a large ground investigation, it may be beneficial to have separate reports with constrained scopes. The report(s) should include the following items:

(a) A description of the investigation programme and its basis;
(b) The layout of the planned buildings;

(c) Descriptions of the site geomorphology, including digital elevation models or other topographic data;
(d) The results and interpretations of geophysical surveys, including maps and cross-sections;
(e) Spatial information about the conducted drillings, including drilling-based cross-sections;
(f) Geological maps and profiles;
(g) Engineering geological classifications, maps and profiles;
(h) Drilling logs and test pit logs;
(i) The results of in situ testing;
(j) The results of laboratory testing;
(k) Descriptions and results of laboratory analyses;
(l) Descriptions of the groundwater regime and the physicochemical, physical and chemical properties of the groundwater;
(m) Data collection should include a documentation of the magnitudes and sources of uncertainties.

GEOTECHNICAL CONSIDERATIONS FOR SITING OF NUCLEAR INSTALLATIONS

Parameters of the geotechnical profiles

2.40. 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, should result in a distribution of values of the geotechnical parameters. 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. In these parametric studies, the state dependency (e.g. density, stress, strain and stiffness) of the responses should be considered.

2.41. A set of parameters should be determined in order to perform the geotechnical evaluation necessary for the construction of a nuclear installation. 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 should include the following:

(a) The geometrical description, (e.g. subsurface stratigraphic descriptions, lateral and vertical extents, number of layers and layer thicknesses);
(b) The physical and chemical properties of soil and rock and the parameters used for classification;
(c) Primary or pressure (P) and secondary or shear (S) wave velocities, stress–strain relationships, static and dynamic strength properties, strain-dependent modulus degradation and damping relationships, consolidation, permeability and other mechanical properties obtained by in situ tests and/or laboratory tests;
(d) 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 (e.g. runoff inundation or erosion, depth to groundwater, spring or groundwater discharge within or near the site).

2.42. Even though conceptually the profile is unique to a particular site, various related design profiles for different uses or assessments should be adopted to allow for different hypotheses in the analysis. These include design profiles for the assessment of the following:

(a) Site specific response spectra;
(b) Liquefaction engineering;
(c) Stresses in the foundation ground;
(d) Foundation stability;
(e) Soil–structure interaction;
(f) Settlements and heaves;
(g) Stability in earth structures;
(h) Earth pressure and deformations or displacements in buried structures.

Seismic site categorization

2.43. For the purpose of seismic site response analyses, the following categorization can be used:

- Type 1 sites: $V_{s,30m} > 1100$ m/s;
- Type 2 sites: 1100 m/s > $V_{s,30m} > 360$ m/s;
- Type 3 sites: 360m/s > $V_{s,30m}$.[4]

This site categorization is based on the assumption that the shear wave velocity smoothly increases with depth. If this assumption is not fulfilled (i.e. V_s decreases or abruptly increases with depth in the upper 30 metres or if there is a strong impedance contrast at any depth), specific analyses including site response assessments should be performed in accordance with best practices, independent of the site type.

If this site categorization is not applicable, soil investigations should be performed to determine the soil type for the site, or to provide comprehensive data for further analyses.

2.44. Independent of the site type, if the value of $V_{s,30m}$ adopted as part of probabilistic seismic hazard assessments is not in conformance with the V_s profile of the site, then site response analyses (incorporating a suitable deconvolution scheme as applicable to the approach used) should be performed.

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[4] $V_{s,30m}$ is estimated using the equation below, from the representative small-strain (< 10^{-4} %) shear wave velocity profile of the site in its natural conditions before the execution of site works:

$$V_{s,30m} = \sum_{i=1}^{n} \frac{30m}{\Delta t_i}$$

where $\Delta t_i = \frac{\Delta H_i}{V_{s,i}}$, and ΔH_i and Δt_i are the thickness of the i^{th} layer in the upper 30 m and the travel time of the shear wave in this layer, respectively.

Free field seismic response and site specific response spectra

2.45. The seismic input level that should be considered is the SL-2 level, as specified in SSG-67 [3].

2.46. Seismic site response assessments under free field conditions should be performed for Type 2 sites and Type 3 sites (see para. 2.43) or when site specific conditions differ from the ground motion model reference conditions. Site response assessments provide input parameters for the assessment of cyclically induced displacements and deformations (including those for soil liquefaction engineering) as well as for soil–structure interaction analyses. Additionally, the site response assessments should provide site-specific response spectra. At a minimum, data on the following should be gathered:

(a) The input ground motion (derived by means of the procedures described in SSG-9 (Rev. 1) [2]).

(b) An appropriate model of the site, based on:

  (i) The geometrical description of the soil layers;

  (ii) The velocities of the S and P waves in each layer;

  (iii) The relative density and density of soils in each layer;

  (iv) Strain-dependent modulus degradation and damping relationships (i.e. (G versus γ) and (ξ versus γ) curves), which describe the apparent reduction in shear modulus G, and corollary increase in internal damping ratio ξ of the soil layers with increasing shear strain γ levels.

(c) For those deep soil deposits in which wave velocities increase smoothly with depth, the change of the aforementioned parameters with increasing confining stress and/or depth.

2.47. Depending on engineering practices, the seismic scenario-compatible outcrop motions recorded at a reference site (e.g. a site with a reference $V_{s,30m}$ value) should be selected from available ground motion databases (databases that include strong motion recordings and associated metadata). These outcrop input motions should be chosen in accordance with the event type, magnitude, distance to the seismic source, and directivity effects, which govern the intensity, frequency content, duration and other relevant seismic parameters. If necessary, these records should be scaled in intensity or duration, or modified in spectrum to match the target seismic scenario, while maintaining consistency with the ground shaking characteristics. Synthetic records can be also tailored based on a combination of Fourier amplitude spectra and random vibration theory.

2.48. In the case of an input ground motion provided as a free field outcrop motion, a deconvolution of the outcropping input motion to a within motion should be performed. As part of deconvolution assessments, a reduction in the within motion intensity levels as compared to those of the outcrop should be carefully reconsidered and justified by means of parametric studies.

2.49. There are alternative methods to assess the idealized layered soil–rock systems, including wave mechanics, finite element, finite difference, discrete element, and hybrid methods. To assess the site response, models with the following properties are acceptable:

(a) A viscoelastic soil system overlying a viscoelastic half space;

(b) A horizontally layered system;

(c) Materials that dissipate energy by internal damping;

(d) Vertically propagating body waves (shear and compression waves).

2.50. 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. If non-linear models are used, the strain dependent modulus degradation and damping responses should be captured as part of the constitutive model that is implemented.

2.51. Uncertainties in the mechanical and dynamic properties of the site materials should be considered through parametric studies. A single set of soil profile parameters should not be assumed conservative for all the considered scenarios (a conservative profile for deconvolution might not be conservative for the site response analysis).

2.52. 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 affected by the earthquake mechanism may be appropriately assessed considering the directivity effects. For these cases, time histories should be selected to include pulse like motions in the ensemble of input motions.

2.53. In seismic response analyses of Type 3 sites, significant deamplification in acceleration levels may be observed. In such cases, assessments supported by engineering judgment based on parametric studies should be considered.

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

Type of test	Parameter	Area of application	Remarks
Seismic refraction and reflection	Deformation propagation time	Site categorization	For surface investigations and vertical sections. Most suitable if the velocity increases with depth and rock surface is regular
Cross-hole seismic test	Dynamic elastic properties (shear wave and compression wave velocities)	Site categorization, obtaining of velocities for particular strata, dynamic properties, rock mass quality. Results used for site response and soil–structure interaction, liquefaction triggering assessment, foundations	For deep investigations, one hole emission, one hole for reception
Uphole and downhole seismic test	Dynamic elastic properties (shear wave and compression wave velocities)	Site categorization, obtaining of velocities for particular strata, dynamic properties, rock mass quality. Results used for site response and soil–structure interaction, liquefaction triggering assessment, foundations	For deep investigations, measurements only need a single hole
Nakamura method	Low level (ambient noise) vibrations	Site categorization, site response and soil–structure interaction, liquefaction triggering assessment, foundations	Horizontal to vertical spectral ratio (HVSR), passive seismic method to determine resonant characteristics of a site
Electrical resistivity	Electrical resistance or conductivity Liquid table content	Internal erosion, location of saltwater boundaries, clean granular and clay strata, rock depth, and underground mines by measured anomalies	Available for deep or surface investigation
Nuclear logging	Water content, density	Settlements, liquefaction, foundations	Necessitates expensive logging techniques
Microgravimetry	Residual anomaly (µGals) Acceleration due to gravity	Sinkholes, heterogeneities, including faults, domes, intrusions, cavities, buried valleys by measured anomalies.	Undesirable subsurface features
Ground penetrating radar (GPR)	Reflections of electromagnetic radiation	Cavities, deformation zones, open and water-filled fractures	Undesirable subsurface features
Magnetic techniques	Magnetic field intensity	Site categorization, areas of humidity	Identification of surface lineaments, maintenance of dykes and dams
Spectral analysis of surface waves (SASW)	Dispersive character of seismic surface waves	Site characterization, subsurface composition and structure	Used to determine the variation in shear wave velocities with depth within layered systems
Microtremor array measurement (MAM)	Dispersive character of seismic surface waves	Site characterization, subsurface composition and structure	Like Seismic analysis of surface waves but uses passive sources and seismic noise
Multichannel analysis of surface waves (MASW)	Surface wave geophysical method; shear wave velocity variations below the surveyed area	Site characterization, subsurface composition and structure	Uses various types of seismic source

* Note: This table is non exhaustive.

TABLE 2. TECHNIQUES FOR IN-SITU GEOTECHNICAL INVESTIGATIONS OF SOIL AND ROCK SAMPLES*

Type of test	Type of material	Parameter	Area of application	Remarks
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, foundations	Usually needs a sufficient number of tests for statistical control
Plate bearing tests	Clay, sand, gravel, rock	Reaction modulus	Compaction control, settlement, foundations	Excavations and embankments
Pressure meter test	Clay, sand, gravel, rock	Elastic modulus, compressibility	Settlement, bearing capacity	Needs a preliminary hole
Pumping test	Clay, sand, gravel	Field permeability	Transmissivity of soil, settlement	Needs piezometers
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
Static cone penetration test	Clay, sand, gravel	Cone resistance, undrained cohesion, shear strength	Settlement, bearing capacity	Including cone penetration test
Cone penetration test (CPT)	For all but very strong soils	Side friction and point resistance, shear wave velocity, pore water pressure, relative density	Detailed information of stratigraphy, shear strength, liquefaction, site response, soil–structure interaction, foundations	No samples recovered Applicable in fine and coarse soils with an average diameter of grain less than 20mm
Seismic cone penetration test (SCPT)	For all but very strong soils	Measurement of small strain velocities	Detailed information of stratigraphy, soil velocity, site response, soil–structure interaction, foundations	No samples recovered
Active gamma cone penetration test (GCPT)	Clean sands	Density	In situ soil density	No samples recovered
Standard penetration test (SPT)	Soils and soft rock; not suitable for boulders or hard rocks	Blow counts	Detailed information of stratigraphy, site response, soil–structure interaction, foundations, settlement	Applicable in fine and coarse soils with an average diameter of grain less than 20mm
Gamma–gamma borehole probe	Rock and soil	Density	Continuous measure of density	
Rock coring	Rock	Measure rock quality designation (RQD) used for various empirical correlations	Rock integrity, slope stability, foundations	Can be further used in classification of rock masses (Q-value)
Overcoring test	Rock	In-situ stress state	Deformability, convergence	Difficult to implement in highly fractured rock
Dilatometer or Goodman Jack	Rock/Soil	Measures E in lateral direction	Settlement, foundations	
Dynamic cone penetration test	Clay, sand, gravel	Cone resistance, relative density	Liquefaction, settlement, foundations	Including standard penetration test

Type of test	Type of material	Parameter	Area of application	Remarks
Large penetration test (LPT); Becker penetration test (BPT)	Gravelly soil	Cone resistance, relative density	Liquefaction, settlement, foundations	

* Note: This table is non exhaustive.

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

Type of test	**Type of material**	**Parameter**	**Characteristics investigated**	**Purpose**
Fall cone test, Casagrande test	Clayed soil	Water content (through liquidity and plasticity indices)	Soil index and classification	Atterberg limits Compressibility and plasticity
Sieve, hydrometer	Coarse grained soil mixtures	Grain size characteristics; percentage of fines and their consistency limits, mean grain size, uniformity coefficient, minimum and maximum void ratio, particle angularity, sphericity and specific gravity	Index properties	Liquefaction, settlement, foundations
Dietrich-Frühling apparatus	All soils	Carbonates and sulphates	Physical and chemical properties of soils	Soil classification
Physical and chemical analysis of soil	All soils	Salt content	Physical and chemical properties of soils	Influence on permeability
Proctor test, gammametry, American Society of Testing and Materials test (relative density)	All soils	Humid and dry densities, water content, saturation ratio, relative density	Consolidation, bearing capacity	Settlement, consolidation, bearing capacity
Oedometer	All soils	Oedometric, Young’s modulus, consolidation coefficient	Consolidation, permeability characteristics	Settlement, consolidation
Shear test box, triaxial compression test	All soils	Young’s modulus, Poisson’s ratio, cohesion and friction angle, under drained and undrained conditions	Shear strength, deformation capability of soil	Bearing capacity
Chevron bend; Brazilian test	Rock	Mode I fracture toughness	Mechanical properties	Rock mechanical characterization
Punch-through-shear (PTS) test	Rock	Mode II fracture toughness	Mechanical properties	Rock mechanical characterization
Cyclic simple shear, torsional shear and triaxial test	All soil	Undrained cyclic shear strength, dynamic Young’s modulus, Poisson’s ratio, internal damping, pore pressure, G–γ and η-γ curves	Dynamic characteristics of soils	Liquefaction, settlement, site response, soil–structure interaction, foundations
Uniaxial / triaxial compression test	Rock	Young’s modulus, Poisson’s ratio, unconfined compression strength and cohesion friction parameters of intact rock	Mechanical properties	Rock mechanical characterization
Point load test	Rock	Unconfined compression strength of intact rock	Mechanical properties	Rock mechanical characterization
Direct / indirect tensile strength test	Rock	Tensile strength of intact rock	Mechanical properties	Rock mechanical characterization

* Note: This table is non exhaustive.

3. GEOTECHNICAL HAZARDS IN SITE EVALUATION FOR NUCLEAR INSTALLATIONS

3.1. Requirement 22 of SSR-1 [1] states:

> "**Geotechnical hazards and geological hazards, including slope instability, collapse, subsidence or uplift, and soil liquefaction, and their effect on the safety of the nuclear installation, shall be evaluated.**"

UNDESIRABLE SOIL CONDITIONS AT NUCLEAR INSTALLATION SITES

Prediction of undesirable subsurface conditions

3.2. Potential undesirable subsurface conditions should be investigated. Understanding of the regional and site geology can provide indications of potential ground collapse. This should include a consideration of soluble rocks (i.e. which are usually either sedimentary rocks (including carbonate types, mainly limestone and dolomites) that are appreciably soluble in water or in weakly acidic solutions or evaporites (of which halite, gypsum and anhydrite are the most common)). The current size and future evolution of the size of cavities or underground solutions are governed by geological factors and environmental factors, both of which should be considered. 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.

3.3. The mechanical stability of the bedrock is governed by the stress state, the properties of the rock mass and the discontinuities transecting the rock mass at all depths of interest. As the discontinuities might define complex patterns and networks, their occurrence, orientation, and properties should be investigated. Prediction of the future evolution of discontinuities should involve a review of the deformation history of the site and its wider surroundings, with specific focus on the presence of deformation zones (e.g. faults, shear zones) and their character. The review should include consideration of the potential for slow movements between juxtaposed bedrock blocks, due to glacial rebound, tectonism, groundwater extraction, and other industrial activities. Capable faults are required to be identified and evaluated (see Requirement 15 of SSR-1 [1]).

Detection of undesirable subsurface conditions

3.4. The investigation programme at a site, as outlined in Section 2, 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 and the potentially resulting complications 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.

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

3.6. Some geophysical methods are useful in reconnaissance for the detection of geophysical anomalies which could correspond to potential subsurface cavities. Such methods include surface electrical resistivity profiling, microgravimetry, low resolution seismic refraction surveys, seismic fan shooting and ground penetrating radar methods. If detected, geophysical anomalies should be confirmed by drilling (and remote visual inspections if necessary) to determine the depth, size and geometry of the cavities.

3.7. Geophysical methods that can be used as preferred 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, high resolution seismic refraction, high resolution seismic reflection, surface wave method, ground penetrating radar methods and suspension P-S logging. Several of these should be applied in conjunction with tomographic techniques, for cross validation.

3.8. Geophysical investigations should be carefully planned and typically implemented in conjunction with drilling and sampling techniques that enhance their effectiveness. The result of an investigation programme to detect and define subsurface cavities and their potential patterns, should be a map or a cross-section showing the cavities and their relationships to the structures, systems and components on the site.

3.9. It might not be possible or practicable 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 potential effects of such cavities on the performance of structures, systems and components important to safety.

3.10. Detection of significant mechanical discontinuities in the rock mass should follow the standard site investigation procedures.

3.11. Evaluation of the significance of bedrock discontinuities should involve characterization of the geometry, size, topological relationships and mechanical properties of the discontinuities. This should enable an understanding of how these discontinuities are arranged into fault and fracture systems and networks. Such understanding is necessary in evaluating their potential to cause slow movements of bedrock blocks and faulting, including slip along the main slip surface of the fault, as well as secondary displacements in fractures spatially associated with the faults.

Evaluation and treatment of undesirable subsurface conditions

3.12. The greatest risk to the foundation safety of a nuclear installation, from a geotechnical perspective, is from the existence of filled or open cavities, solution filled features at shallow depths (relative to the size) and mechanical discontinuities below the foundation of the structures, systems and components at the site. The compressibility and the erosion potential of the natural filling material should be evaluated to determine its impact on the bearing capacity, settlement and future erosion as a result of possible changes in the groundwater regime.

3.13. The stability of natural cavities and mechanical discontinuities below the foundation level should be considered. The size of the cavity, its depth, the patterns and properties of the associated mechanical discontinuities, type of rock and bedding inclinations above the cavity are primary factors that influence the stability of the roof and the depth for consideration. Changes in the vertical pressures due to structural loads or seismic events could cause instability of the roof of the cavity. A site that is underlain by a potentially large and complex cavity system should be excluded, since a realistic evaluation of the hazard posed by the cavity system might be very difficult. In areas where the size and geometry of the cavity can be reliably determined, analytical techniques such as finite element analysis and finite difference analysis should be used for the evaluation of the stability of cavities.

3.14. For some sites where complex subsurface conditions are encountered below the foundation level, the results of the stability evaluation should indicate the need for ground treatment to ensure the safety of the structure. Further recommendations on the improvement of foundation conditions in the case of complex subsurface conditions are provided in Section 4.

Improvement of surface conditions and subsurface conditions

3.15. If it has been found necessary to make improvements in the subsurface conditions due to the risk of slope failure or other unfavourable soil or ground conditions, the improvements (e.g. jet grouting, ground cementation) should be designed and conducted during the ongoing stage of site characterization and/or site preparation and construction, and their effectiveness should be verified by in situ testing (see also paras 3.48–3.50 and 4.16–4.19).

3.16. In areas subjected to slow differential movements of bedrock blocks (e.g. due to unevenly distributed glacial rebound), engineering countermeasures should be considered. In such cases a layer of crushed rock can be used as a mitigation technique, and the movements should be monitored and assessed against well established and defined limits for maximum allowed movements.

NATURAL SLOPES ON OR NEAR SITES FOR NUCLEAR INSTALLATIONS

3.17. A natural slope is composed of rocks and/or soils. In rock slopes, the existence of weak parts, like weak layers, lithological contacts and discontinuities such as joints and faults, play an important role in their stability. In soil slopes and weak parts in rock slopes, an increase of pore water pressure caused by heavy rainfall or earthquakes should be evaluated if the water table level is within the slope.

Slope stability

3.18. Slope stability assessment will depend largely on the separation distance from the nuclear installation and site and the features of the slope. Potentially hazardous slopes should be identified and evaluated in terms of such factors as distance from the site or installation, orientation, slope angle, height, geology, groundwater level as well as any of their changes over time (i.e. additional units at the same site, settlements within the slope, glacial rebound, groundwater changes and/or climate change). If a slope is determined to be distant enough that it would not affect any safety related structures, systems and components, emergency planning zones or other important site features, no further measures are necessary.

3.19. The stability of slopes surrounding structures, systems and components that are important to the safety of a nuclear installation should be assessed with regard to the safety of the installation. In particular, the effects of earthquakes (e.g. ground motion, liquefaction, landslides, tsunamis) as well as the effects of heavy rainfall, flash floods and thawing permafrost should be considered in the assessment of slope stability.

3.20. For pseudo-static slope stability calculations, the methodology is based on the consideration of seismic effects as equivalent static inertial forces by means of seismic coefficients. To determine the equivalent static inertial forces, the seismic amplification in the slope should be based on a seismic loading distribution along the vertical direction of the slope. Peak ground acceleration can be used for the initial estimation of the inertial forces; however, a lower value may be acceptable, if justified by additional calculations and studies. In slope stability calculations, the resulting safety factor calculated based on the pseudo-static equilibrium should be at least 1.1.[5]

3.21. If the resulting safety factor is not greater than the specified minimum (regulatory expectation), a dynamic response analysis should be performed based on the design seismic ground motion to evaluate the seismic effects more precisely. If necessary, the permanent displacements (residual deformation) should be evaluated to assess safety and stability in cases where the safety factor is close to unity. For sites on (or surrounded by) natural slopes, these evaluations are important for beyond design basis external events, and the results should be considered with respect to cliff edge effects for nuclear installations.

3.22. If natural slopes are credited as barriers against floods or tsunamis, the influence of ground erosion and related changes of material properties and slope geometry should be taken into account in the safety assessments and evaluations.

3.23. If a slope is deemed to be potentially unstable, a stability analysis should be performed. The stability analysis should consider factors such as slope angle, height, water content, groundwater level, reduced soil strength under seismic loadings, other geotechnical conditions of the material of the slope, as well as the potential uncertainties associated with these factors due to the variability of the slope material (e.g. primary stratification of the sediments; see Section 2).

3.24. A conventional sliding surface analysis is usually performed to evaluate a safety factor against

[5] Different national regulations and practices may specify a minimum safety factor as high as 1.5.

sliding failure. This method is based on a simple equilibrium of force and is valid for an external load like gravity. However, for loads such as those generated by an earthquake, an additional evaluation should be conducted to determine the exact location of the expected sliding surface if it is different from the sliding surface determined using the minimum safety factor that considers only gravity and the residual strength of the slope. A three dimensional slope stability analysis might be needed to more realistically evaluate the stability of the slope and the impact of the portion of the failed slope portion.

3.25. If the evaluation results in a safety factor that is low enough to indicate a potential for a major sliding failure, suitable measures for stabilizing and strengthening the slope and/or for preventing any debris from reaching structures, systems and components important to safety should be designed and implemented. Otherwise, the layout of the nuclear installation site should be modified.

Measures for prevention and mitigation of slope failure

3.26. If a natural slope is assessed as not sufficiently safe (i.e. by a safety factor and/or any other criteria (e.g. residual displacements)) measures for prevention and mitigation of slope failure should be considered, such as the removal of the whole or a part of the natural slope. If removal is deemed unreasonable, strengthening measures should be considered, such as lowering the slope angle, soil nailing, rock bolting, grouting, anchors, piles and/or retaining walls.

3.27. There are two different mechanisms to strengthen a slope with anchors: one provides extra confining pressure to increase the strength of the slope material by a pretension of the anchor; the other uses the strength of anchors to hold a sliding block after sliding is initiated. There is a large variety of possible measures; therefore, reference should be made to appropriate design manuals in determining the best option for a specific scenario. The approach selected should be supported by a quantitative comparison of the various options, and should be agreed with the regulatory body.

3.28. Measures should also be considered to prevent any debris from reaching structures, systems or components important to safety. For example, a protective wall can be designed to stop the debris after an external event of a certain severity that may exceed the stability of the slope. The wall should be designed with consideration of the maximum and minimum size of the falling debris that is estimated to reach the wall. The design should ensure that the wall will withstand the loads of the debris and its impact, as well as the earth pressure to be retained.

SOIL LIQUEFACTION ON SITES FOR NUCLEAR INSTALLATIONS

3.29. The basis of liquefaction engineering assessments for a nuclear installation site should be established, and acceptable performance levels should be defined. Soil liquefaction should be fully described using definitions of the soil behaviour and loading conditions (e.g. flow liquefaction versus cyclic softening, soil response to shear, controlling stresses, onset of threshold strain levels, excess pore pressure ratio).

3.30. The most critical seismic design scenario adopted for liquefaction assessments might not necessarily be the same as that used for the assessment of overlying structural systems. A distant but larger magnitude seismic event with a lower intensity but longer duration (producing a larger number of equivalent stress cycles) may be more significant for liquefaction response.

3.31. The necessary data for the liquefaction engineering assessment should be collected. The following list presents relevant types of data:

(a) Historical performance data. Available data for soils of identical or similar properties to those at the site should be compiled and studied. Additionally, if available, the cyclic performance of the site during and after historical earthquake events should be documented.

(b) Soil profile. A detailed representative soil profile indicating the stratigraphic characteristics of each layer with special emphasis on their spatial variabilities, should be developed.

(c) Groundwater regime. Piezometric and/or borehole water level data should be used to define the phreatic surface. The seasonal fluctuations in the phreatic surface should be conservatively considered in the assessments. Additionally, borehole pump and/or cone penetration test with pore water pressure measurement (commonly referred to as CPTu) data can be used to determine the permeability parameters.

(d) Index properties. For coarse grained soil mixtures, sieve and sedimentation/laser diffraction or hydrometer tests should be performed on soil samples to assess grain size characteristics. Samples should be collected to accurately represent the spatial variability of the site soil conditions. In addition to the percentage of fines and their consistency limits, mean grain size, uniformity coefficient, relative density and specific gravity are additional important properties that are useful for liquefaction engineering assessments.

(e) Standard penetration tests. There exists significant variability in the equipment used, and procedures and protocols adopted, for standard penetration testing. To minimize this variability, such testing should be performed in conformance with standardized testing methodologies (e.g. those developed by the International Organization for Standardization (ISO) and American Society for Testing Materials (ASTM)). Additionally, to enable the execution of test corrections, the equipment details (e.g. sampler type and dimensions, hammer type, cathead-rope-pulley system details (for none-automatic hammers), rod type, rod length, coupling type and dimensions, anvil-hammer, anvil-rod inclinations) should be fully documented. Either a calibrated standard penetration test hammer system should be used or direct stress wave energy measurements should be performed in situ in conformance with standardized testing methodologies (e.g. ISO, ASTM). The field blow counts, N, should be corrected to consider the variability in the procedures followed, equipment used, and stress states. Considering the spatial variability of standard penetration test blow counts, either deterministic or probabilistic representative blow counts should be determined. When gravelly soil layers are present, large penetration test, Becker penetration test or shear wave measurement results should be used for the assessments.

(f) Cone penetration tests. The cone penetration test has an advantage over the standard penetration test in that it provides a continuous soil profile, allowing better judgment about the extent of liquefiable soil layers. However, unless customized systems are used, conventional cone penetration testing equipment does not allow soil sampling, so soil classification should be developed on the basis of sleeve friction and cone tip resistance data. Additionally, penetrability decreases with increasing soil density and grain size, which might limit its use in gravelly and/or cemented sandy soils. Under these circumstances standard penetration tests and cone penetration tests should be performed either jointly or in combination with boring. For reliable assessments, calibrated cone penetration test equipment and sensors should be used.

(g) Shear wave velocity (V_s) measurements. Such measurements are a complimentary tool for liquefaction triggering assessments. There are different V_s measurement techniques with different levels of accuracy. Downhole and cross-hole measurements include drilling of boreholes and sampling. Non-invasive surface measurement techniques (e.g. seismic analysis of surface waves or multichannel analysis of surface waves) can also be considered but only provide a mean V_s value per layer of the soil profile. Seismic cone penetration test systems may also be considered to measure V_s and unite cone penetration test and V_s based assessments. When possible, multiple independent field test data-based methods should be used to reduce the epistemic uncertainty in liquefaction triggering predictions.

(h) Relative density. The in situ relative density of cohesionless soils should be evaluated on the basis of the standard penetration test blow counts and/or cone penetration test cone tip resistances from justified correlations compared to estimates from undisturbed sampling. Conversely, clean soil samples (fines content < 5%) at the target relative density can be directly reconstituted in the laboratory, after estimating the minimum and maximum void ratios, for which standardized testing methods are available.

(i) Undrained cyclic shear strength. The undrained cyclic shear strength of soils may be evaluated directly by means of cyclic loading tests performed in the laboratory on undisturbed (frozen) or reconstituted soil samples. Cyclic simple shear, torsional shear and triaxial tests along with centrifuge models are commonly employed in engineering practice to evaluate the undrained cyclic response of soils. The quality of the undisturbed samples or the method of sample preparation (i.e. reconstitution) for laboratory tests significantly affects liquefaction response, and should therefore be considered in interpretation of the assessment results. An alternative to laboratory based assessments are case history based semi-empirical methods for the evaluation of liquefaction resistance, and post-liquefaction (residual) shear strengths, which are presented as functions of effective confining stress and penetration resistances.

(j) Strain dependence of soil properties. For advanced dynamic analysis, G–γ and η-γ curves for each soil layer are needed to describe the apparent degradation in shear modulus and increase in damping ratio with increasing shear strain levels, respectively.

(k) Additional soil properties. Additional parameters (e.g. Poisson's ratio, critical state soil parameters) may be needed as part of more advanced assessments.

(l) Seismic design parameters. A minimum of moment magnitude and peak ground acceleration data pair in deterministic seismic hazard assessments. Alternatively, peak ground acceleration levels deaggregated for moment magnitude bins, or peak ground acceleration levels corrected to a reference magnitude (duration) event as part of probabilistic seismic hazard assessments, is needed.

(m) Ground motion duration. The number of equivalent uniform stress cycles corresponding to the magnitude of the seismic design event is needed. The magnitude of the seismic event can be commonly used to assess the duration of seismic demand on the premise that ground motion duration can be correlated as a first approximation to the number of cycles of the earthquake.

(n) Cyclic stress ratio. The induced cyclic stress ratio at the depth of interest — which can be estimated by seismic site response analyses or by simplified procedures using site-response based soil mass participation factors — should be evaluated.

(o) Laboratory based cyclic resistance. For laboratory based assessments, the cyclic stress ratio versus the number of equivalent stress cycle curves that correspond to the triggering of liquefaction should be developed.

(p) Laboratory–field condition corrections. A set of correction factors to account for the differences between laboratory conditions and field conditions should be developed and justified.

(q) Additional seismic parameters. These parameters (e.g. design basis time histories) may be needed for more complex assessments.

3.32. Liquefaction engineering assessments should include, at a minimum, the following engineering evaluation steps:

(1) Liquefaction susceptibility and triggering;

(2) Post-liquefaction residual strength and overall post-liquefaction stability;

(3) Liquefaction induced deformations and displacements;

(4) Consequences of induced deformations and displacements;

(5) Engineered mitigation (if necessary).

Liquefaction susceptibility

3.33. As part of susceptibility assessments, fully saturated clean sands, clean gravels, and clean sand–gravel mixtures should be considered as susceptible to liquefaction. Clean sands or gravels are defined as soils with a fines content of less than 5%. The lateral extent of the susceptible soil layers should be confirmed and studied in the overall stratigraphical context.

3.34. The mixtures of sands and/or gravels with fines should also be assessed for susceptibility. For the susceptibility assessment of fine grained soil mixtures, grain size, grain distribution, consistency limits and experimentally assessed pore pressure generation data can be used. If soils are concluded to be susceptible, liquefaction triggering assessments should be performed.

Liquefaction triggering

3.35. As part of liquefaction triggering assessments, three different approaches are used:

(a) Case history based semi-empirical approaches (see para. 3.41);

(b) Analytical approaches (see para. 3.42);

(c) Advanced constitutive model based numerical approaches (see para. 3.43).

3.36. It is generally possible to compute a lower bound solution in all of the three approaches to liquefaction engineering assessments outlined in paras 3.41–3.43 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 might only generate limited deformations , even if pore pressure buildup is 100%. Hence, cliff edge effects should be considered.

3.37. For deterministic assessments, the safety factor against liquefaction triggering should be greater than the limit value considered for the calculation and should be consistent with the methods used (regulations or standardized codes). For probabilistic assessments, the frequency of liquefaction triggering should be established sufficiently low to satisfy performance targets.

3.38. Fulfilling the minimum safety factor or annual probability of liquefaction triggering might not guarantee an acceptable displacement or deformation performance. Thus, the rest of the liquefaction engineering assessments (see paras 3.44–3.50) should be performed independent of the liquefaction triggering evaluation outcome.

3.39. When liquefiable conditions exist within a soil layer, their volume should be estimated using resistance profiles measured in situ (e.g. beds, lenses, extended layers). The distribution of these liquefiable levels, their configurations, distances, hydraulic connections, permeability contrasts and proximity of the drainage boundaries should be considered in the liquefaction triggering assessment. If there are insufficient details in the data, the whole layer should be considered liquefiable.

3.40. Liquefaction triggering assessments should consider groundwater levels which should be defined from piezometric measurements and should account for groundwater fluctuations.

Case history based semi-empirical approaches

3.41. Semi-empirical approaches are based on deterministic or probabilistic assessment of liquefaction triggering case histories from historical events, where capacity versus demand terms were selected respectively, as an in situ test parameter (e.g. $SPT_{N1,60,cs}$,$CPT_{qc1n\text{-}cs}$, V_{s1}, BPT_{N}) versus normalized cyclic stress ratio. The use of these semi-empirical approaches involves the earthquake moment magnitude, fines content, non-linear shear mass participation factor and some basic soil parameters (e.g. unit weight, grain size, consistency limits).

Analytical approaches

3.42. The analytical approaches to liquefaction triggering assessments should comprise of the following steps:

(1) Choosing a set of representative accelerograms, consistent with the seismic design scenario, at the outcropping reference rock site.

(2) Deconvoluting or convoluting the outcropping reference rock motions to within motions, and estimating induced cyclic shear stress histories through a set of site response analyses.

(3) Converting the number of cycles of transient stress–time histories into equivalent uniform stress cycles.

(4) Developing the cyclic resistance ratio versus the number of equivalent uniform stress cycle curves through a set of cyclic laboratory tests.

(5) Assessing the liquefaction triggering response by comparing the induced cyclic stresses with the cyclic resistance corresponding to the number of equivalent uniform stress cycles estimated earlier.

Advanced constitutive model based numerical approaches

3.43. A validated and calibrated constitutive model, capable of modelling cyclic large strain response of fully saturated soils is incorporated into the non-linear, time step analysis to directly assess the buildup of pore pressure and the overall seismic response. As part of these assessments, effective stress-based, time-domain, coupled or decoupled analyses are usually performed to simulate strain and time dependent changes in soil stiffness and strength along with the buildup of pore pressure. The onset of liquefaction triggering can be directly identified under the cyclic loading defined by the set of input motions used. However, the results might vary considerably owing to the use of different input motions, different constitutive models, and/or a different set of constitutive model parameters. Advanced dynamic analyses necessitate calibration of many parameters which are difficult to identify in routine applications. The results should therefore be calibrated with case history based evaluations and should consider the uncertainties in the parameter used.

Post-liquefaction residual strength and overall post-liquefaction stability

3.44. If it is concluded that soils could liquefy during the design basis seismic event, post-liquefaction residual strength and overall post-liquefaction stability assessments should be performed, taking into consideration the uncertainties associated with the parameters and methodology used. Semi-empirical, analytical and calibrated constitutive model based assessments can also be used to assess post cyclic residual strength. Post-liquefaction stability assessments should include the applicable potential failure modes, including slope stability, bearing capacity, uplift, sliding and toppling, and others if relevant. These assessments should also consider earthquake aftershocks during transitional phases (pore water pressures have not dissipated) and all changes of soil states after the main shock (see para. 4.93), if applicable.

3.45. If post-liquefaction overall stability cannot be guaranteed, mitigation solutions should be engineered and implemented against soil liquefaction. In overall stability evaluations, an acceptable safety factor and/or displacement and deformation performance levels should be selected to comply with short term loading conditions.

Liquefaction induced deformations and displacements

3.46. When overall stability is achieved, cyclically induced deformations and displacements should be evaluated. Post-liquefaction differential settlements and their associated uncertainties should be assessed.

3.47. Consequences of induced deformations and displacements should also be assessed. The deformations and displacements should comply with acceptable performance criteria. Acceptable levels of performance with regard to preserving repairability, reducing overall damages, maintaining serviceability and/or minimizing out of service duration should be defined.

Liquefaction mitigation

3.48. If cyclically induced deformations and displacements do not fall within the acceptable performance levels described in para. 3.44, mitigation solutions should be engineered and implemented.

3.49. The engineering mitigation of the unacceptable liquefaction hazard should be performed on the basis of applicability, effectiveness, the ability to verify the reliability of the mitigation achieved, cost and other concerns (e.g. regulatory requirements, environmental issues).

3.50. Liquefaction engineering assessment procedures should also be followed for beyond design basis external events, where the seismic input level is selected for a return period exceeding SL-2. The performance of safety related structures, systems and components during and after beyond design basis external events should be evaluated against predefined acceptance criteria to avoid cliff edge effects.

4. GEOTECHNICAL CONSIDERATIONS FOR DESIGN AND EVALUATION OF SITES FOR NUCLEAR INSTALLATIONS

DYKES AND DAMS ON OR NEAR SITES FOR NUCLEAR INSTALLATIONS

4.1. The term dyke is used to describe a structure running along a water course and the term earth dam applies to a structure, used to create a water reservoir upstream from a nuclear installation.

4.2. Before construction, in addition to classical geophysical and geotechnical tests, special attention should be paid to the soil/rock permeability of the site close to the areas of the foundations. Soil and/or rock permeability should be monitored throughout the operating lifetime of the installation.

4.3. The potential hazard associated with the failure of upstream water control structures such as dams is required to be analysed (see para. 5.21 of SSR-1 [1]). The design of dykes and dams should consider all possible failure modes (including those that are dependent on pore pressure inside the embankment and on internal erosion caused by water seepage and flow inside the embankment).

4.4. The design requirements related to consequences of failure of dykes and dams that might impact the safety of the nuclear installation (e.g. due to the loss of cooling water), should be consistent with the design requirements for the installation itself, especially with regard to the evaluation of natural hazards (e.g. earthquakes, rainfall), including the return period for flooding.

4.5. In addition to the usual methods of engineering design, a specific analysis should be performed to evaluate 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. If the base ground is soil containing fines, the settlement caused by consolidation should be taken into account when setting the height of the water table in a design cross-section used for stability analysis (i.e. because there is a possibility that, after experiencing pore pressure accumulation by an earthquake or any other external loads or events, the borderline might move down and the body of dykes or dams and the dry side might sink down lower than the water table).

4.6. Surveillance (periodic inspection and monitoring of dams and dykes) and maintenance work should be performed continually during construction and operation of the nuclear installation (by third party or shared by dam operator/safety organization) to prevent and predict potential damage such as internal erosion of dykes. Dam safety review should be conducted periodically to demonstrate that the dam is safe, operated safely and maintained in a safe condition, and that surveillance is adequate to detect any developing safety problem.

SEA WALLS, BREAKWATERS AND REVETMENTS ON OR NEAR SITES FOR NUCLEAR INSTALLATIONS

4.7. Sea walls, breakwaters and revetments are civil engineering structures used for protecting nuclear installations against the wave action of an ocean or a lake during storms and tsunamis. These structures should be properly designed to withstand soil erosion, flooding and structural failures that might jeopardize safety related structures, systems and components at the nuclear installation.

4.8. The 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 consideration of the maximum static water level derived from flood hazard evaluations, as described in IAEA Safety Standards Series No SSG-18, Meteorological and Hydrological Hazards in Site Evaluation for Nuclear Installations [9].

4.9. The stability of sea walls, breakwaters and revetments should be properly evaluated in relation to the sustainability of their protective functions as well as the effects of their possible failure. The methods of evaluation are similar to those for retaining walls and for the sliding failure of slopes. 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 may be encountered at the foot of these structures and their potential for liquefaction may need to be evaluated, assessed and/or mitigated.

RETAINING WALLS ON SITES FOR NUCLEAR INSTALLATIONS

4.10. Retaining walls can be classified into two groups, as follows:

(a) Gravity walls, for which the weight of the wall and possibly that of the retained soil play an important part in the wall's stability;

(b) Embedded walls, such as sheet walls, the stability of which depends on the passive pressure of soil and/or anchors.

Frequently, a retaining wall is a combination of both types. For retaining walls, the input parameters are similar to those for assessing the stability of foundations and are generally supplemented by geometric 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. Sufficient data should be collected and provided for stability assessment (to a soil depth consistent with the analyses being performed).

4.11. For stability assessments, the pressure of the earth behind the wall may be considered as the active pressure. However, when the admissible displacement of the wall is limited, the pressure of the earth used in assessments and evaluations should be the at rest pressure.

4.12. For analysis of stability during an earthquake, the inertia forces of the retaining wall and surrounding ground, and the influences of liquefaction or accumulated pore pressure of the ground behind and under the retaining wall should be taken into consideration. Those influences that produce the more unfavourable conditions should be considered. If the more unfavourable conditions are not clear, a series of parametric studies or the most extreme conditions for both sides of the wall should be considered. For instance, in a pseudo-static evaluation based on seismic coefficients, the vertical component of the seismic acceleration should be considered as acting upward or downward.

4.13. 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 foundation. The results of failure modes evaluations might indicate that the movement of a retaining wall becomes larger, the vertical and lateral displacements of the backfill tend to increase and the effects reach further, especially when soil liquefaction occurs in the backfill and/or foundation soil.

FOUNDATIONS OF NUCLEAR INSTALLATIONS

Preliminary foundation work

4.14. Preliminary foundation work is 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 essential to safety, and should include the following, as appropriate:

(a) Prototype testing (including test fills and verification of techniques for improving foundation material);
(b) Excavations for foundations or foundation systems;
(c) Dewatering and its control;
(d) Rock removal (if rock is removed by blasting, controlled blasting techniques should be used to minimize blast-induced fractures below foundations);
(e) Improvement of foundation materials (including e.g. modification of material and drainage);
(f) Placement of structural backfill;
(g) Placement of mud mats or any type of protective layer.

4.15. Testing requirements of preliminary foundation work should be specified for proper control and documentation. Testing should include both field and laboratory tests and be performed throughout the construction period.

Improvement of foundation conditions

4.16. The improvement of foundation conditions is meant here in its widest sense and includes modification of the mechanical behaviour of the foundation material (e.g. by soil compaction), the total replacement of loose or soft material by an improved material (consistent with specified quality and performance criteria), or the use of an added material (of sufficient quality) to improve the static and/or dynamic behaviour. Another acceptable approach is the use of deep foundations.

4.17. Improvement of the foundation conditions should be performed if one or more of the following apply:

(a) The foundation material is not capable of carrying the structural loads without unacceptable deformation (settlements);

(b) There are cavities that can lead to subsidence, as discussed in Section 2;

(c) There are heterogeneities, on the scale of the building size, which can lead to tilting and/or unacceptable differential settlements;

(d) The in situ foundation material has shear wave velocities that might lead to unacceptable amplification of the rock input seismic ground motions;

(e) The in situ foundation material is susceptible to liquefaction.

4.18. When improvement of the foundation conditions is necessary, the following tasks should be performed:

(a) Characterization of the existing in situ profile and determination of relevant soil parameters pertinent to the selected ground improvement technology;

(b) Determination of the necessary profile of the foundation material;

(c) Selection of the particular technology (e.g. overexcavation and compacted backfill, rock removal, densification by various methods, solidification by cement or permanent dewatering) by which improvements in the foundation are to be made;

(d) Performance of a prototype testing programme to verify experimentally the effectiveness of the methods proposed to improve the subsurface conditions;

(e) Preparation of the specifications for field operations, after the proposed technology has been verified;

(f) Performance of an investigation at the completion of the improvement programme to determine whether the specifications were met;

(g) Incorporation of any improvement in foundation material into the design profiles used in the assessments.

4.19. Foundations should not be built on expansive or collapsing soils unless mitigating measures are implemented and it is demonstrated that these phenomena do not adversely impact foundation performance.

Choice of foundation system and construction

4.20. Two systems of foundations are available for transmitting the superstructure loads to the soil: shallow foundations and deep foundations. Shallow foundations should be 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 should be used to transfer the loads to stiffer soil layers at depth.

4.21. The following criteria should be applied in the choice of the foundation system for a nuclear installation:

(a) The forces due to the structures should be transmitted to the subgrade soil without any unacceptable deformation;

(b) The soil deformations induced by the SL-2 input motion should be compatible with the design requirements of the structure;

(c) 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;

(d) The risks associated with underground water should be taken into account;

(e) One single type of foundation should be used for each structure;

(f) The choice of the type of foundation should depend on the type of building, for example a continuous raft should be used under the nuclear island (either supported by piles or founded on competent ground) because it provides homogeneous settlements under static and dynamic loads and because it provides a barrier between the environment and the buildings.

4.22. 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.

Analysis and design of the foundation system

4.23. Foundation instability can develop due to inadequate bearing capacity and/or excessive settlements, sliding, and overturning; these conditions should be carefully considered, as they can occur due to static or dynamic loadings. Additionally, special consideration should be given to environmental and meteorological conditions and construction activities because they can lead to foundation damage.

Inputs to analysis and design of the foundation system

4.24. The soil and rock characterization should include classification, stiffness and strength, and hydrogeologic properties. Engineering properties should include index properties, density, shear strength, seismic wave velocity, elasticity moduli, compressibility, stress state and cyclic resistance. Some of these properties may be strain dependent; testing and reporting of these properties should cover the strain range expected from design analysis.

4.25. Rock property characterization should include rock type (i.e. sedimentary, igneous, volcanic or metamorphic), lithology (e.g. mineralogy, texture), overall geometry (e.g. strike and dip of bedding), discontinuities (e.g. joints, shear zones, fractures), weathering and depositional environment, field and laboratory measurements of engineering properties (e.g. mechanical, dynamic, hydraulic and geochemical properties), and rock mass conditions.

4.26. If the subsurface materials are soils or soft rock, information on the stress history of the subsurface materials should be obtained to predict settlement and heaves, and to assess the hazard of gross foundation (shear) failure. Additionally, for soft rocks (e.g. gypsum, chalk) and clay soil in saturated conditions, their creep under static loading should be assessed. For computing this stress history, the following should be obtained at a minimum:

(a) The geological stress history and the resulting pre-consolidation stress and overconsolidation ratio;

(b) The loading–unloading history in operations such as dewatering, excavation, backfilling and building construction, as well as the geometry of the disturbed spaces;

(c) The parameters for the establishment and application of the constitutive law applicable to the subsurface materials and their variation with depth including consolidation parameters. These parameters include the following:

   (i) Natural water content;

   (ii) Void ratio;

   (iii) Liquid limit and plasticity index;

   (iv) Compression and recompression indices;

   (v) Coefficient of secondary consolidation.

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

(a) The best estimate value for body wave (compression and shear) velocity profiles with a range of variation as determined by in situ measurement techniques. These values should be consistent with the strain levels anticipated from the design basis ground motions.

(b) The 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).

(c) 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.

(d) The non-linear soil behaviour, which should be taken into account by making use of equivalent linear or non-linear material properties. The design parameters for the equivalent linear method are the shear modulus and the damping versus shear strain relationships for each of the subsurface layers.

(e) The ground water level to be used in performing an analysis.

Soil–structure interaction

4.28. Soil–structure interaction (commonly referred to as SSI) denotes the phenomenon of coupling between a structure and its supporting medium (soil or rock) during an earthquake or during the application of dynamic loads directly to the structure. The response of a structure depends on the characteristics of the ground motion, the applied dynamic loading, the surrounding soil, and the structure itself.

4.29. For structures founded on hard rock or very stiff soils, the foundation motion during an earthquake is essentially the same as the ground motion at the foundation level in the free field. However, for softer media, the effects of soil–structure interaction should be evaluated since the foundation motion differs from that in the free field due to the interaction of the soil and structure during the seismic excitation.

4.30. The two general methods of analysis for soil–structure interaction are as follows:

(a) The direct method, which is based on analysing the combined soil–structure system in a single step, without invoking superposition. The direct method solves the soil–structure interaction problem in the time domain and the frequency domain. The direct method can be implemented as linear or non-linear time history analysis.

(b) The substructuring method, which solves the soil–structure interaction problem in the frequency domain explicitly invoking superposition. Time variations of the earthquake ground motion are treated through Fourier transform techniques applied to the input motion. The substructuring method can only be implemented as linear analysis.

Either method of analysis is acceptable, provided the physical characteristics of the supporting media or the structure can be modelled adequately.

4.31. The effects of soil–structure interaction should be considered for all safety-related nuclear structures, as follows:

(a) The effects may be minimal for structures supported by rock or rock like foundation material depending on the amplitude and frequency content of the earthquake ground motion, the dominant natural frequencies of the structure, and the stiffness of the supporting rock. In such cases, seismic analysis may be performed using a fixed-base model.

(b) In general soil–structure interaction analysis should be performed for sites with conditions of Type 2 or Type 3 foundation material (see para. 2.43). A fixed base support may be assumed in modelling of structures for seismic response analysis for Type 1 sites[6].

4.32. The objective of the analysis of dynamic soil–structure interaction should be 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.

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[6] Some States have additional requirements in the consideration of Type 1 structures as fixed-base (e.g. Type 1sites where the combination of earthquake input motions, rock conditions, and structure characteristics are demonstrated to behave as a fixed-base system).

4.33. Analyses of soil–structure interaction should investigate the following effects:

(a) The effects of the foundation soil condition on the dynamic response of the structure;

(b) The effects of buried structures (e.g. scattering effects);

(c) The effects of dynamic pressure and deformations on the buried structures;

(d) The global stability and potential uplift and sliding of the foundation;

(e) The effects of structure–soil–structure interactions.

4.34. The foundation should be designed to resist the forces developed and accommodate the movements imparted to the structure by the design ground motions or the dynamic loading applied to the structure. The dynamic nature of the forces, the expected ground motion, the design basis for strength and energy dissipation capacity of the structure, and the dynamic properties of the soil should be included in the determination of the foundation design criteria.

Direct method of soil–structure interaction analysis

4.35. In general, soil–structure interaction analysis by the direct method should consist of the following steps:

(1) Model the structure.

(2) Model the foundation: geometry, stiffness, and interface.

(3) Model the soil:

   (i) Determine soil material properties (linear, non-linear);

   (ii) Discretize the soil;

   (iii) Locate the bottom and lateral boundaries of the soil–structure model such that the structural response is not significantly affected by those boundaries.

(4) Establish input motion to be applied at the boundaries, compatible with the site response analyses.

(5) Perform soil–structure interaction analyses.

(6) Perform a second stage analysis for detailed structure response, as necessary.

4.36. The location and type of lateral and bottom boundaries should be selected so as not to significantly affect the structural response at points of interest. Soil discretization (elements or zones) should be established to adequately reproduce static and dynamic effects.

Substructuring methods of soil–structure interaction analysis

4.37. Substructuring methods can be classified into four types depending on how the soil and structure interface degrees of freedom are handled: (i) the rigid boundary method, where ‘rigid’ refers to the boundary between the foundation/partially embedded structure and the soil; (ii) the flexible boundary method; (iii) the flexible volume method; and (iv) the substructure subtraction method. Technical justifications should be provided to demonstrate the adequacy of soil–structure interaction analysis based on the substructure subtraction method.

4.38. In general, the seismic soil–structure interaction subproblems that the four types of substructuring method listed in para. 4.37 should solve are as follows:

(a) The site response problem. This step applies to all four methods.

(b) The structure model.

(c) The scattering problem, noting the following:

  (i) For the rigid boundary approach, the foundation input motion is developed by applying the constraints of rigid body motion to the free field particle motions developed in the site response problem.

  (ii) For the flexible boundary methods, foundation input motion is not a separate output of the complete soil–structure interaction analysis.

  (iii) For the simplified soil spring method, the foundation input motion may be assumed equal to the free field ground motion.

(d) Foundation impedances:

  (i) For the rigid boundary methods, foundation impedances may be developed on the basis of continuum mechanics, finite element methods, tables of data, or other methods. In general, complex-valued, frequency-dependent impedances are generated.

  (ii) For the simplified soil spring method, frequency-independent spring stiffnesses and dashpots are most often used. Care should be taken to ensure that the non-linear behaviour of the impedances (i.e. stiffness and damping components) is properly reproduced in the soil spring method.

4.39. Similar to the direct method of soil–structure interaction analysis (see paras 4.35 and 4.36), soil discretization (elements or zones) should be established to adequately reproduce static and dynamic effects. 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).

4.40. The effects on the analyses of uncertainties in the design profile parameters for the foundation material should be considered. These effects should produce a bounding 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. 2.51 should be used.

4.41. The foundation soil and the structures exhibit three dimensional dynamic characteristics; consequently, the 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 installation, a three dimensional analysis should therefore be performed.

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

4.43. Embedment effects should be considered in the soil–structure interaction analysis of structures supported by embedded foundations. The potential for reduced lateral soil support of the structure should be considered when accounting for embedment effects. Acceptable methods to account for partial soil-wall separation include the following:

(a) Calculating the seismic and static soil pressure to evaluate the extent of separation, and performing a soil–structure interaction analysis based on the reduced contact area or reduced soil stiffness adjacent to the walls.

(b) Assuming no connectivity between the structure and the lateral soil over the upper half of the embedment or 6 metres, whichever is less. Full connection between the structure and lateral soil elements may be assumed if adjacent structures founded at a higher elevation produce a surcharge equivalent to at least 6 metres of soil.

(c) Including the potential for separation and stiffness degradation in the constitutive model of the soils surrounding the foundation and their interfaces.

4.44. Structure–soil–structure interaction (commonly referred to as SSSI) is a three dimensional phenomenon which could be overemphasized by linear analysis. SSSI represents the coupling of the dynamic response between adjacent structures through the soil, where the vibrations of one of the structures may have an effect on another. Generally, structure–soil–structure interaction may be neglected for overall structural response. Exceptions are as follows:

(a) Seismic analysis of a relatively light structure in close proximity to a massive structure;

(b) The analysis of local effects due to the effect of one structure on another, such as increased pressure on walls of adjacent structures.

Structure–soil–structure interaction effects for these cases should be considered by either including all structures in the same soil–structure interaction model, or by computing the ground motion at the footprint of the light structure using the analysis of the heavy structure and modifying the input motion to include the effects induced by the heavy structure on the translational and rotational input motion to the light structure.

4.45. Simplifying assumptions in structure–soil–structure interaction should be carefully considered. Except for specific sites where significant inclined waves or surface waves may be induced by the soil configuration, the simplifying assumption of vertically propagating shear and compressional waves are considered acceptable for the soil–structure interaction analysis, provided that torsional effects due to non-vertically propagating waves are considered. A loading contribution due to accidental torsion may be included to take into account torsional effects. Accidental torsion is intended to address the effects of waves not propagating vertically, rotational components of ground motion, and distributions of mass and stiffness in the structure that differ from those assumed in the construction of the mathematical model.

(a) Seismic wave incoherency effects should be considered in the soil–structure interaction analysis. Seismic wave incoherency arises from the horizontal spatial variation of both horizontal and vertical ground motions. There are two sources of incoherency or horizontal spatial variation of ground motion random spatial variation (scattering of waves due to the heterogeneous nature of the soil or rock beneath the foundation and along the propagation paths of the incident wave fields), and wave passage effects (systematic spatial variation due to difference in arrival times of seismic waves across a foundation). Generally, the incoherency effects reduce the foundation translational motions and increase the rotational motions. The differences are larger at high frequencies and with larger foundation dimensions. The use of coherency models that represent spatial variation effects, as a function of frequency and separation distance, and soil–structure interaction formulations that implement such coherency models should be adequately justified.

Probabilistic analysis of soil–structure interaction

4.46. Where safety objectives and performance goals are defined probabilistically, probabilistic soil–structure interaction analysis may be used to determine the probability distributions of responses of interest (demands) and show that the design meets the acceptance criteria (see also paras 4.23 and 4.24 of SSR-1 [1]).

4.47. Probabilistic soil–structure interaction analysis should be performed with simulation approaches. The correlation between simulated parameters should be incorporated into the probabilistic models. A Monte Carlo approach can be used for systems that contain significant non-linear behaviour. For systems with essentially linear, or that include minor non-linear responses, either a Monte Carlo sampling approach or a more efficient stratified sampling approach such as Latin hypercube simulation may be used. Parameters significant to the seismic response should be treated as random variables.

4.48. For the soil–structure interaction response analysis, the input should consist of an ensemble of input motion sets. The ensemble is represented as N ground acceleration time series sets or as N response spectra sets. Each of the N input motion sets should consist of two horizontal components and one vertical component.

4.49. A set of N statistical response analysis simulations should be assembled, where each simulation is developed by sampling a random value from the previously identified parameters. N response analyses are performed, and the statistical properties of selected response quantities are evaluated. Given the computational demands of soil–structure interaction response analysis Latin hypercube simulation is generally used.

4.50. The effects of seismically induced soil–structure interaction effects related to foundation overturning and sliding, and potential differential displacement for single foundations and between safety related piping and conduits connected to the foundation or the superstructure should be considered.

Contact pressure beneath foundations

4.51. The distribution of contact pressure beneath the foundations and the stresses induced in the subsurface materials should be derived from the analysis of the static soil–structure interaction. In addition to the elastic and geometric parameters of the structures (e.g. geometry and stiffness of the foundation mats and of the superstructure of the buildings), the mechanical characteristics of the subsurface materials should be included in the design profile to allow the foundation contact pressure to be computed.

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

4.53. For the two extreme conditions of infinitely stiff and infinitely flexible foundations (in the case of distributed load on soil), general solutions are available in foundation design textbooks and design standards. 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 change as the construction proceeds. If the subsurface materials exhibit non-linear behaviour when subjected to unloading and reloading during excavation, dewatering and backfilling, this should also be considered.

4.54. 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.

Foundation stability, sliding and overturning

4.55. The assessment of foundation stability should be performed 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.

4.56. 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. These forces can be converted to equivalent static forces for the assessment of stability. The equivalent static forces should be derived in accordance with the item under consideration. These equivalent static forces may be applied to the analysis of uplifting and overturning and to the computation of lateral loads on subsurface walls and retaining walls. The use of a non-linear or linear time history approach to show stability for seismic loading should be considered.

4.57. In the case of an embedded foundation, active pressure of the soil should be regarded as an additional horizontal load.

4.58. For structures founded above the groundwater table level, the angle of shearing resistance between soil and structure 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.

4.59. If the sliding resistance is the sum of shear friction along the foundation and the soil lateral pressure (i.e. up to the full passive pressure capacity induced by embedment effects), a consistent lateral displacement criterion for activating the passive soil pressure should be used. This involves the use of a static (as opposed to dynamic) coefficient of friction consistent with the use of partial versus full passive pressure.

4.60. The sliding safety evaluation of the foundation of a nuclear installation 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 seismic input motion with the acceptable value.

4.61. For static loading, stability against sliding and overturning analysis should provide an adequate factor of safety against sliding and overturning[7]. The analysis should consider variations in loading during the life of the structure due to such factors as rise in groundwater level, removal or reduction in passive forces downslope (for any reason), increase in driving forces upslope (for any reason), liquefaction potential, or other factors.

4.62. For evaluation of overturning, a ground contact ratio, defined as the ratio of the minimum area of the foundation in contact with the soil to the total area of the foundation, may be used. The seismic response computed over the entire duration of the seismic ground motion should be considered to determine the minimum value of this ratio. If the defined minimum contact area[8] is not achieved then the non-linearity due to the foundation uplift should be assessed and, if found to be important, should be accounted for in the design.

4.63. Under certain combinations of ground motion, groundwater level and geometrical configuration of the building, conventional computing procedures might give rise to a potential uplift. This does not mean that the foundation will necessarily lift up, but rather that conventional procedures to compute the structural response might not be applicable under these circumstances. If the estimated surface area of the uplift of the foundation is larger than the defined contact area limit[8] (as a percentage 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.

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[7] Some States define the minimum factor of safety against sliding and overturning under dynamic loadings as 1.1, while other States define the minimum factor as 1.5. It should be noted that the acceptable safety factor depends on the method of analysis, definition of capacities, and other considerations.

[8] Differing definitions of minimum contact areas exist. Some States set a minimum value for the ground contact area ratio limit as low as of 70% corresponding to a 30% uplift, while other States set a minimum contact area ratio limit as high as 80% for overturning and 20% for uplift.

4.64. The evaluation of foundations should consider the effects of the bending moment and shear forces in the foundation induced by static and dynamic loads, buoyant load, potential foundation lift-off effect and embedment effect, as well as the effect of various sliding interfaces on selections of coefficient of friction (e.g. soil shear failure, concrete to soil, waterproofing to soil, and concrete basemat to concrete mudmat).

4.65. Uncertainties in dynamic foundation sliding and rocking responses should take into account variable friction coefficients, cohesion strength and other parameters to estimate behaviour and ensure the design meets the acceptance criteria of the regulatory body.

4.66. The sliding safety evaluation of the foundation should include a comparison of the displacements during and after seismic input motion with acceptable values.

Settlement and bearing capacity

4.67. Foundations should be evaluated to ensure adequate bearing and tolerable settlement of the underlaying soils. The evaluation should include assessment of geological materials extending to a sufficient depth within the zone of influence of foundations. Evaluation should consider uncertainties due to materials, models and loads.

4.68. Linear and/or non-linear methods may be used for settlement evaluation. Both total settlement and differential settlement due to elastic compression, consolidation, secondary compression, and dynamic settlement over the life of the nuclear installation should be considered.

4.69. An assessment of settlement under static loads should be performed. The possibility of differential settlements or heaves between the buildings of a nuclear installation because of connecting pipes, conduits and tunnels should be investigated. Settlements and heaves are also important with regard to 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.

4.70. Short and long term settlements occurring during the operating lifetime of the nuclear installation should be estimated.

4.71. 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:

(a) Undrained shear settlement;

(b) Settlement caused by consolidation;

(c) Settlement caused by creep.

4.72. The following actions should be taken to evaluate long term settlement:

(a) The anticipated loading history of the subsurface materials should be specified (e.g. excavation sequence, dewatering process, backfilling, construction process).

(b) For each layer, a model should be chosen in accordance with data from laboratory and in situ testing.

(c) The models should be assessed and improved by means of the interpretation of measurements for settlement and heave made during excavation, dewatering, backfilling and construction.

(d) 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.

4.73. 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.

4.74. Seismically induced settlements should be considered in settlement evaluations. Settlement effects from other potential vibratory sources should also be included in these evaluations, if appropriate. Other effects causing additional settlements (e.g. changes in ground elevation, adjacent excavations, hydrogeological conditions), should also be considered in the settlement evaluations, as appropriate.

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

4.76. The effects of the construction sequence and the installation of systems and components on settlement should be assessed.

4.77. For structures located on soils that may exhibit permanent seismically induced vertical or lateral deformations, the effects of the permanent deformations should be evaluated.

4.78. The method for computing the ultimate bearing capacity should be consistent with the assumptions associated with the soil conditions and the chosen approach. Classic soil mechanics methodologies 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 main 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. This solid is assumed to exhibit no deformation prior to shear failure and a plastic flow at constant stress after failure.

4.79. 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 performed. In all these analyses, the vertical seismic force should be taken into account in a conservative manner.

4.80. For cohesive soils, both short term and long term bearing capacities should be assessed.

4.81. Estimates of ultimate capacity should include dynamic effects and should not be based on standard relationships associated with general shear failure concepts appropriate for static load cases.

4.82. For cohesive soils or saturated cohesionless soils, earthquake induced strength degradation (associated with cyclic softening or excess pore water pressure generation) should be used in bearing capacity evaluations.

4.83. The water level should be assumed to be equal to the highest water level expected due to the maximum probable flood for static loading. The groundwater level should be assumed to be the mean level, due to the maximum probably flood, for the determination of the bearing capacity under seismic loading.

4.84. 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.

4.85. The potential for failure of the bearing capacity of the subsurface materials for a nuclear installation 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 seismic loading conditions with reasonable safety margins.

4.86. If a safety factor is achieved on the basis of a conservative assumption, no further analysis is generally necessary. Acceptable safety factors depend on the method of analysis and on other considerations. In the conventional bearing capacity method, the safety factor should be consistent with national and/or international codes and standards including combinations of loads that involve seismic input (the overturning effect). Reliability analysis, including load and resistance factor design approaches, may be used to demonstrate that an adequate margin is included in the design.

4.87. 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 needed in foundation materials and in choosing an appropriate technique for the improvements. If, under combinations of loads that involve the 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.

Heave effects on foundations

4.88. The effects of frost depth and frost heave should be considered in the analysis of shallow foundations.

4.89. In areas subjected to frost heave, spread footings and mats should be placed below frost depth or designed to have sufficient uplift resistance to overcome forces due to ground heave and frost jacking[9]. The structural design of foundation connections should be sufficient to transmit the loads due

[9] Frost-jacking is the frost heave process which involves upward displacement of an object embedded in freezing soil.

to frost heave and adfreeze[10].

4.90. Where shallow foundations are placed above the seasonal frost depth, they should be protected from frost heave effects using frost-protected shallow foundations[11].

4.91. The effects of heave due to excavation and unloading, expansive soils or rocks, and glacial rebound should be evaluated where applicable.

EARTH STRUCTURES AND BURIED STRUCTURES ON SITES FOR NUCLEAR INSTALLATIONS

4.92. The design of earth structures and buried structures that are important to safety at a nuclear installation should be consistent with the design of the installation itself. In particular, the external hazards against which those structures are designed should be consistent with the events that are selected in the design of the nuclear installation; these events and their associated loads should be listed in the contractual terms of reference related to the earth structures and buried structures. The list should be supplemented by the specific events that could challenge the safety of these structures.

4.93. The time, extent and duration of seismic aftershocks are unpredictable: consequently, changes of soil states after a main shock should be taken into account for aftershock safety assessments and evaluations. For example, degradation of soil rigidity and strength might result from decreased confining pressure caused by excess pore water pressure that could take considerable time to dissipate.

4.94. At sites that are expected to experience inundation caused by a flood or tsunami, potential ground erosion including changes in geometry and material properties should also be taken into account for evaluations according to the nature of the event (duration, peak flow, maximum water height). This holds in particular for considerations of phenomenon related to water flows leading to the failure of earth structures or soils foundations such as internal and external erosion, scouring.

4.95. Evaluations of the consequences of failure of earth structures and buried structures that are important to safety should be conducted with special consideration of their significance and purpose (e.g. a buried electrical conduit may fail due to breaches in watertightness).

4.96. The consequences of failure of safety related earth structures and buried structures, and any structures, systems and components dependent on them, should be evaluated against stability and/or deformation criteria.

4.97. The consequences of failure of earth structures and buried structures that are only indirectly related to the safety of the nuclear installation (i.e. that are not important to safety but could have an impact on the site or on structures, systems or components that are important to safety) should also be taken into consideration. To simplify evaluations of complex interactions with such structures, stability

[10] Adfreeze is the process by which two objects are bonded together by ice formed between them.

[11] A frost-protected shallow foundation is a foundation that does not extend below the design frost depth but is protected against the effects of frost using, for example, expanded polystyrene and extruded polystyrene.

analyses can be conservatively adapted provided the consequences remain insignificant.

EMBEDDED STRUCTURES ON SITES FOR NUCLEAR INSTALLATIONS

4.98. 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, as follows:

(a) The consequences of underground walls acting as retaining walls (see paras 4.10–4.13);

(b) The consequences for the building itself (see paras 4.99–4.102).

4.99. 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 criteria for the underground walls (particularly in relation to leaktightness) to be met under different loading cases. For this purpose, the possible cracking of concrete (limiting the stresses in reinforcement bars and concrete) should be taken into account in the design of the foundation and the construction joints of buildings. If the embedded structure is credited or considered as a containment structure, the recommendations in relation to containment considerations are provided in IAEA Safety Standards Series No. SSG-53, Design of the Reactor Containment and Associated Systems for Nuclear Power Plants [10].

4.100. 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 consideration. At coastal sites, the possible adverse effects of varying levels of groundwater salinity on the foundation material and isolation material should be considered.

4.101. 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 cases, 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 the effects of the interaction of soil with the underground walls.

4.102. For stability analysis of mechanically embedded foundations under seismic loads (see paras 4.55–4.66), the friction between soil and walls should be disregarded.

BURIED PIPES, CONDUITS AND TUNNELS ON SITES FOR NUCLEAR INSTALLATIONS

4.103. The layout of buried pipes or conduits should be considered in the geotechnical site investigation programme. Adequately spaced boreholes, drillings, soundings and/or test pits should be made along the pipe routes. Special consideration should be given to identifying areas of discontinuities or changes in the foundation material along the route of the piping. Areas that are susceptible to inundation by floods or tsunamis should be avoided for buried pipes or conduits. In areas that are susceptible to frost the effects of frost depth and frost heave should be considered in the design and analysis of buried pipes, and if necessary, frost protection measures should be implemented.

4.104. The investigation boreholes, drillings, soundings, or test pits should be to a depth that will depend on the stratigraphy of the foundation material below buried piping but at a minimum should extend to a competent soil layer below the foundation level.

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

4.106. Buried piping should be placed at a sufficient depth to prevent damage due to surface loading (e.g. traffic loads) or should be designed to resist the surface loads to which the pipes will potentially be exposed.

4.107. Piping should be placed on well compacted granular material over competent foundation material, so that no damage or distortion of the piping due to displacements (e.g. heaving, settlement, lateral spreading) or liquefaction of the foundation material can occur. Foundation improvement techniques may be used for weak subsurface conditions.

4.108. Safety related buried piping, conduit systems and tunnels should be designed to resist the effects of earthquakes.

4.109. 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 (i.e. an anchor point) and the ground surrounding the buried system. The following seismically induced loadings should be considered for long buried piping, conduits and tunnels:

(a) Strains induced by the passage of seismic waves;
(b) Differential displacements in zones of different materials;
(c) Additional loads due to seismic oscillations resulting in sloshing of internal liquids;
(d) Deformation and shaking of the ground or anchor points relative to the ground;
(e) Ground failures such as surface fault rupture, liquefaction, landslides, settlements and discontinuous displacements.

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

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

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

4.112. Unless it is otherwise justified, it may be assumed that sections of a long, linear buried pipe, which are 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. An estimate of axial strain 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.

4.113. 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 (e.g. building attachment points) 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.

4.114. Discontinuous displacements (both parallel and perpendicular to the length of the system), axial strains, and/or inclinations of the structure that could compromise the function of buried pipes, conduits or tunnels should be evaluated.

4.115. 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.

4.116. The consequences of the failure of safety related ducts and pipes and other underground features passing near or through other structures at the nuclear installation site should be given appropriate consideration. If hazardous effects are expected, appropriate measures should be taken to protect the installation; alternatively, the site layout should be reconsidered.

5. MONITORING OF GEOTECHNICAL PARAMETERS ON SITES FOR NUCLEAR INSTALLATIONS

PURPOSE OF MONITORING GEOTECHNICAL PARAMETERS ON SITES FOR NUCLEAR INSTALLATIONS

5.1. Field monitoring, in particular quantitative measurements of performance outputs, should be implemented to define and monitor the geotechnical parameters necessary for the safe design, construction and operation of the nuclear installation. Electrical devices have become the standard method of monitoring, and widely used in geo monitoring applications.

5.2. Subsurface investigations, 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 enables criteria for foundation design to be established, including for the performance of the foundation materials and structures under anticipated loadings. In order to verify the performance of the foundations and earth structures, their actual field behaviour should be monitored from the beginning of siting activities through construction to operation.

5.3. The monitoring of actual loads and deformations enables a field check to be made of the predicted behaviour of the foundations and buried structures. Since the construction stage is generally lengthy, 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 OF SITES FOR NUCLEAR INSTALLATIONS

5.4. The phases of construction of a nuclear installation usually consist of excavation, backfilling and building construction. The behaviour of the soil should be monitored during each of these phases. During the excavation and backfilling phase, deformation of subsurface material (e.g. heave and settlement, lateral displacements) should be monitored, and load evaluations should be made. Monitoring should be continued throughout the lifetime of the nuclear installation.

5.5. The groundwater regime under buildings and in adjoining areas at the site of a nuclear installation should be monitored to verify the conditions outlined in the design assumptions, especially if deep drainage systems or permanent dewatering systems are installed. Groundwater monitoring should be undertaken early in the geotechnical investigation to inform the hydrological and hydrogeological models.

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

5.7. The seismic behaviour of the nuclear installation 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.

5.8. If the site has the potential for slow bedrock movements, such potential relative movements between recognized bedrock blocks (e.g. on opposing sides of fracture zones) should be monitored.

5.9. 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 of monitoring other sites for nuclear installations. In deciding on the number of devices and manual measurement points, their expected failure rate should be evaluated, with special consideration of their need for replacement.

5.10. If a specific geotechnical monitoring device needs to be replaced, the replacement procedure should be documented in detail. The new device may represent an updated technology and direct equivalence in measurement capacity is not compulsory, provided that the minimum requirements for resolution, accuracy, data collection and environmental impact during installation are satisfied. Where possible, a final set of measurements should be taken from the device to be replaced, to be calibrated with respect to reference measurements from the new device.

5.11. The geotechnical monitoring programme should be documented, clearly indicating the procedures for data collection, standardized storage of data, data management and visualization. The programme should include the necessary qualifications of technical personnel, as well as the specification and qualification of hardware and software systems that collect and report data, along with protocols for data dissemination. The monitoring programme and monitoring records should include the entire monitoring history beginning from site selection, through to construction, commissioning, operation and decommissioning of the nuclear installation.

5.12. A periodic review of the monitoring programme should be performed. The review period should be dependent on the rate of technological advances in the field, geotechnical and/or structural requirements during the lifetime of the installation, and any other conditions that would necessitate an updated monitoring programme.

MONITORING DEVICES FOR SITES FOR NUCLEAR INSTALLATIONS

5.13. Specifications for the selection of geotechnical monitoring devices — including preferences in terms of sensors, data acquisition systems and related components and accessories — should be defined based on an assessment of long term exposure to environmental conditions, including atmospheric conditions, temperature, hydrogeological conditions, hydrochemical conditions, electromagnetic interference and sources of background noise. For seismic monitoring devices see paras 3.54-3.59 of SSG-9 Rev. 1 [2].

5.14. All operational geotechnical monitoring devices should be regularly maintained. Procedures should be defined, and documented with respect to the management system, for maintaining commissioned monitoring devices, including, where applicable, protocols for harmonizing data obtained from failed devices with reference readings of the newly installed equivalents. Additionally, data harmonization and calibration among all operational devices of different type, technology or method of measurement (e.g. digital, digital output with manual data collection procedure, fully digitized and automated systems, fully manual and analogue systems) should be assured.

5.15. Monitoring devices should be used to observe the behaviour of the foundation and related materials. Table 4 contains a list of available devices that can be used for monitoring soil and buildings (e.g. extensometers, load and pressure cells), depending on the site, the monitoring requirements and the type of nuclear installation.

5.16. Monitoring of safety related structures should include total and differential settlements, lateral displacements and deformations, earth and pore pressures, and inclinations along sloping ground surfaces. Monitoring of the performance of other structures with a potential impact on safety related structures, systems and components should also be considered.

TABLE 4. GEOTECHNICAL MONITORING DEVICES FOR SITES FOR NUCLEAR INSTALLATIONS

Type of Device	Principle	Location	Parameter Measured	Purpose
Piezometers, water level meters	Hydraulic pressure	Boreholes, reservoirs, weirs	Pore pressure, water table	Monitoring of water table, positive and negative pore pressure monitoring, hydrogeological characterization, monitoring of water level in reservoirs, drainage channels and weirs
Hydraulic devices	Hydraulic, U-tube, hydraulic load cells	On basement and beneath, on isolated foundations of operating machinery	Deformations and stresses of the basemat, loads on soil nails, rock bolts and prestressed ground tendons	Behaviour of the soil–structure system, high-sensitivity settlement monitoring of foundation systems
In situ settlement plates	Topography	Ground surface	Displacements, settlements	Settlement of structures
Settlement monuments	Topography	Ground surface, fill layer base or along intermediate layering within fills	Displacements, settlements	Settlement of structures and fills
Rod extensometers	Mechanical, electromechanical	Boreholes, excavation support structures	Settlement, heave, lateral deformations, stability of jointed rock masses	Deformation of structures, stability of natural soil and rock slopes
Magnetic extensometers, induction current type extensometers	Electromagnetism	Boreholes	Settlement, heave	Deformation of fills and human-made slopes
Gammagraphy, photogrammetry	Superposition of picture	Ground surface	Deformation of topography	Deformation of structures
Global positioning system	Aiming by satellite	Ground surface, site	Topography of the site. XYZ-coordinates (particularly Z)	Site evaluation. Relative movements between bedrock and blocks
Interferometric synthetic aperture radar (InSAR)	Synthetic aperture radar	Remote sensing of ground surface	Deformation	Settlement of structures, ground subsidence

Type of Device	Principle	Location	Parameter Measured	Purpose
Georadar	Radar based proximity measurement	Ground surface	Distance	Deformation of structures, monitoring performance of slopes
Lasermeter	Laser light source	Ground surface, underground openings, interior spaces in industrial facilities	Distance	Behaviour of structural systems, convergence of underground openings
Inclinometers, tiltmeters, pendulum systems	Electromechanical, electrolytic, microelectromechanical systems, optical, laser	Borehole, isolated locations on structural members, embankments, fills, route structures, tall structures	Tilt, absolute inclination, deformation profile derived from tilt measurements along predefined axes, three dimensional deformation profile using three dimensional measurement of inclination along an array	Stability of slopes, embankment loading related deformations, retaining structures, walls, determination of fill settlement profile, performance of machine foundations
Crackmeter, jointmeter, tape extensometer	Electromechanical, mechanical	Surface of structural members, foundation members, retaining structures, surface of rock masses along discontinuities	Displacement measurement in one dimensional to three dimensional.	Performance of structural and architectural joints, construction joints, performance of retaining structures, slope stability monitoring, prefailure identification of unstable rock masses (e.g. rock fall hazards, toppling, planar and wedge type failures)
Soil extensometer	Electromechanical	Soil mass (embankments), superstructures	Lateral deformations under tensile stresses	Crack under tensile stresses, lateral movements in embankments and fills
Strain gauges	Electromechanical, fibreoptic	Deep foundation elements, deep excavation elements, basemats, tunnel and gallery linings, embedded within soil for the case of distributed fibreoptic strain sensing	Strain (uniaxial, biaxial, triaxial)	Behaviour of soil–structure system (e.g. deep foundations, deep excavations), foundation stress distribution, deformation monitoring of rock slopes

Type of Device	Principle	Location	Parameter Measured	Purpose
Earth pressure cells, stress cells	Electromechanical	Embankments, retaining structures, tunnel and gallery linings	Total earth pressure, stresses within concrete members	Monitoring of vertical and lateral earth pressures, measurement of lateral earth pressure coefficient, monitoring of behaviour of underground openings
Load cells	Electromechanical	Soil nails, rock bolts, prestressed ground anchors	Loads on soil nails, rock bolts and prestressed ground anchors, piles	Behaviour of the soil–structure interaction system, performance verification of piles
Seismometers	Accelerometers, triggers	Free field, buildings	Acceleration time histories	Operability of nuclear installations; seismic behaviour of structures; floor response spectra, early warning triggers due to natural hazards.
Acoustic emission	Acoustic signal emission	Ground surface, underground openings, pipeline systems	Acoustic waveform, time and frequency domain waveform analysis	Detecting leaks in buried piping, early detection of unstable rock masses in slopes and underground openings.
Temperature sensing	Thermistors, resistive temperature detectors, thermocouple action, contact based, distributed fibreoptics	Mass concrete (embedded), concrete, steel (surface), soil mass, embankment, drainage features, boreholes	Temperature, spatial and temporal variation of temperature	Seepage detection, temperature induced strains and stresses, structural integrity (piles), performance of steel structural systems, performance of energy piles, buried pipelines
Tachymeter	Laser	Ground surface	XYZ-coordinates (particularly Z)	Relative movements between bedrock and blocks

6. APPLYING A GRADED APPROACH TO GEOTECHNICAL ASPECTS IN SITING AND DESIGN OF NUCLEAR INSTALLATIONS OTHER THAN NUCLEAR POWER PLANTS

6.1. For nuclear installations other than nuclear power plants, a graded approach is required to be applied (see Requirement 3 of SSR-1 [1]). Paragraph 4.3 of IAEA SSR-1 [1] states that “The level of detail in the evaluation of a site for a nuclear installation shall be commensurate with the risk associated with the nuclear installation and the site and will differ depending on the type of nuclear installation.”

6.2. The application of a graded approach to the geotechnical site investigation and characterization (see Requirement 22 of SSR-1 [1]) might increase the uncertainty in the geotechnical parameters used as input for the design bases. This larger uncertainty might result in a reduction of the reliability of the design. It should be ensured that any reduction of reliability is considered acceptable with respect to the overall safety objectives.

6.3. The risk associated with a nuclear installation depends on the potential failures of the installation and on the consequences of such failures (also see Section 9 of SSG-67 [3]). The overall safety objective in site evaluation, as established by Requirement 1 of SSR-1 [1], is the same for all nuclear installations. However, for a particular nuclear installation, the radiological consequences of failures might be so small that reliability levels lower than those in nuclear power plants could be accepted without compromising the safety objective.

RADIOLOGICAL HAZARD CATEGORIZATION OF SITES FOR NUCLEAR INSTALLATIONS

6.4. The application of a graded approach to the geotechnical site investigation should be based on a site specific consequence analysis (simplified, as appropriate) that categorizes the installation in terms of the radiological hazard. Four radiological hazard categories are defined in Table 5, from ‘high’, which corresponds to large nuclear power plants, to ‘conventional’, which corresponds to conventional industrial facilities, with a negligible or no radiological hazard.

TABLE 5. RADIOLOGICAL HAZARD CATEGORIES BASED ON THE CONSEQUENCES OF FAILURES IN A NUCLEAR INSTALLATION

Hazard category	Consequences on the site	Consequences off the site	Remarks
High	Radiological or other exposures that might cause loss of life of workers in the facility.	Potential for significant off-site radiological consequences.	Hazard category of nuclear power plants. Graded approach is not applicable.
Medium	Potential for significant on-site consequences. Unmitigated radiological release would necessitate on-site evacuation.	Small potential for off-site radiological consequences.	See para. 6.10
Low	Potential for only localized radiological consequences (within 30–100 m of the point of release).	No off-site radiological consequences.	See para. 6.10
Conventional	No radiological consequences.	No radiological consequences.	Geotechnical investigation with the same scope as for conventional industrial facilities.

6.5. The radiological consequences of potential failures depend on the nature of the nuclear installation and the characteristics of the site. The following factors should be considered (see also para. 9.5 of SSG-9 (Rev. 1) [2]):

(a) The radioactive inventory at the site, including the distribution of radioactive sources in the installation;
(b) The hazard associated with physical and chemical processes at the installation;
(c) The thermal power of the nuclear installation, if applicable;
(d) The configuration or operating status of the installation for different kinds of activity;
(e) The distribution of radioactive sources in the installation
(f) The design of safety systems for the prevention of accidents and for mitigation of their consequences;
(g) The characteristics of structures, and the means of confinement of radioactive material;
(h) The characteristics of processes or of engineered features that might show a cliff edge effect in the event of an accident;
(i) Characteristics of the site that are relevant to the dispersion of radioactive material (e.g. topography, dominant winds, water masses, demography of the region);
(j) The potential for on-site and off-site contamination.

6.6. The simplest consequence analysis that should be performed corresponds to an unmitigated release of the full radioactive inventory present in the nuclear installation. This is a conservative bounding analysis, which provides a first approximation of the hazard category of the nuclear installation. If the result of such a radioactive release is negligible radiological consequences (i.e. for workers, the public and the environment), then the installation should be classified at the lowest radiological hazard category and the geotechnical design basis should be established in the same way as for a conventional industrial facility.

6.7. Consequence analyses for radiological hazard categorization of a nuclear installation (see para. 4.3 of SSR-1 [1]), in which a design-dependent set of source terms is used and credit is taken for some engineered mitigating features, should be considered acceptable, provided the source terms reasonably envelop all potential accident scenarios, and the robustness of the mitigating features for design basis events can be clearly demonstrated[12].

THE APPLICATION OF A GRADED APPROACH TO GEOTECHNICAL SITE INVESTIGATION AND CHARACTERIZATION BASED ON RADIOLOGICAL HAZARD CATEGORIZATION

6.8. For nuclear installations in the 'high' hazard category (see Table 5), the scope of the geotechnical site investigation and characterization should be the same as for large nuclear power plants.

6.9. For nuclear installations in the 'conventional' hazard category (see Table 5), the scope of the geotechnical site investigation and characterization should be the same as for non-nuclear industrial facilities.

6.10. For nuclear installations categorized in the 'medium' or 'low' hazard categories (see Table 5), the application of a graded approach to the geotechnical site investigation and characterization should be considered. Typically, for a 'medium' hazard category installation, a narrower scope compared to that used for a 'high' hazard category installation should be considered; for a 'low' hazard category installation, an increased scope compared to that used for a 'conventional' hazard category installation should be considered.

6.11. The amount to which a graded approach is applied to the geotechnical site investigation and characterization depends on the foundation requirements for the nuclear installation and on the complexity of the subsurface conditions. The appropriate approach should be determined based on the judgment of qualified geologists and geotechnical engineers. At a minimum, a graded geotechnical site investigation and characterization should address the following items:

(a) The geological structure of subsurface materials, with a description of the stratigraphic sequence of soil or rock strata, and the nature and dimensions in plan and depth of the different formations;
(b) The static and dynamic geotechnical properties of subsurface materials, as necessary to assess the stability and bearing capacity, evaluate seismic and other hazards, and to define design basis parameters;

[12] Robustness of these features can be 'clearly demonstrated', for instance, by showing a design margin up to several times the design basis event.

(c) The potential presence of complex subsurface conditions, such as underground cavities or expansive soils or rocks;

(d) Hydrogeological conditions at the site, including the presence and thickness of aquifers, the groundwater regime, groundwater levels, the amplitude of fluctuations, as well as the chemical composition of groundwater and the potential effects on the materials of underground structures.

The application of a graded approach may include the level of detail (e.g. number and layout of boreholes, types and number of laboratory and field tests) used in the investigation of these items, but the scope of the geotechnical site investigation should always include these items[13]. Variability and uncertainty in subsurface materials should always be addressed.

GEOTECHNICAL EVALUATION OF SITES FOR NUCLEAR INSTALLATIONS

6.12. Geotechnical characterization is required to provide sufficient information to perform a reliable and defendable site evaluation with respect to geotechnical hazards, including slope instability (see paras 5.27 and 5.28 of SSR-1 [1]), soil liquefaction (see paras 5.30 and 5.31 of SSR-1 [1]), and collapse, subsidence or uplift of the site surface (see para. 5.29 of SSR-1 [1]). A graded approach is required to be applied (see paras 4.4 and 4.5 of SSR-1 [1]), depending on site conditions, and this may mean that simplified bounding analyses or expert judgement could be acceptable to screen out these hazards.

6.13. If, as a result of the site evaluation (see Requirement 4 of SSR-1 [1]), one geotechnical hazard cannot be screened out, then a more detailed investigation and characterization should be conducted, in order to refine the evaluation. As a result of this refinement and further evaluation, the site may be considered suitable on the basis of specific established suitability criteria, and corresponding specific design bases should be established to ensure the safety of the nuclear installation through design, construction and operation measures.

DESIGN BASIS OF NUCLEAR INSTALLATIONS DERIVED FROM GEOTECHNICAL SITE CHARACTERIZATION

6.14. The application of a graded approach to the geotechnical site characterization might result in an increased level of uncertainty in the geotechnical parameters used as input for the design basis. This larger uncertainty should be taken into account when defining the design basis.

6.15. The application of a graded approach to site characterization might also result in less detailed knowledge of the structure of the subsurface materials (e.g. variability of soil profiles within the site) or

[13] Defining an appropriate geotechnical site investigation programme for a nuclear installation is very site-specific and it is common that the programme is developed in several phases, in which the level of detail is progressively increased, based on the outcome of the previous phase. The application of a graded approach may be achieved by eliminating or reducing the effort in the final phases.

of other characteristics (e.g. physical or geochemical properties of the soil). The design basis should account for such uncertainties by defining reasonable ranges of variation to be considered in the design or by selecting the most unfavourable conditions.

7. APPLICATION OF THE MANAGEMENT SYSTEM TO THE GEOTECHNICAL ASPECTS OF SITES FOR NUCLEAR INSTALLATIONS

7.1. A management system applicable to all organizations involved in the site geotechnical investigation, characterization and evaluation is required to be established before the start of the programme (see Requirement 2 of SSR-1 [1]). Requirements for such a management system are established in IAEA Safety Standards Series No. GSR Part 2, Leadership and Management for Safety [11], and supporting recommendations are provided in IAEA Safety Standards Series No. GS-G-3.1, Application of the Management System for Facilities and Activities [12].

7.2. Organizations in the supply chain[14] are required to either have their own arrangements for managing safety (see Requirement 11 of GSR Part 2 [11]). They may have their own management system approved by the main contractor, or else adhere to the management system of the main contractor.

SCOPE OF THE MANAGEMENT SYSTEM IN RELATION TO THE GEOTECHNICAL EVALUATION AND MONITORING OF SITES FOR NUCLEAR INSTALLATIONS

7.3. The management system should cover all the processes and activities described in this Safety Guide, as applicable to each site. This includes the following:

(a) Compilation of data from relevant literature or previous investigations;

(b) Field investigation campaigns, including sampling, logging and storage of samples;

(c) Field testing, measurement or monitoring;

(d) Laboratory testing;

(e) Data processing and reduction of test data;

(f) Calculations;

(g) Verification and validation of computer software;

(i) Documentation control and archiving.

DOCUMENTATION OF THE MANAGEMENT SYSTEM IN RELATION TO THE GEOTECHNICAL EVALUATION OF SITES FOR NUCLEAR INSTALLATIONS

7.4. Documentation describing the management system should be organized into different tiers. In the first tier, there should be a management system manual including the following information:

(a) General statement of policies and objectives of the manual;

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[14] In the context of this Safety Guide, this includes site evaluation services, such as area topographic surveying, drilling and sampling, surface geophysics, borehole geophysics, laboratory testing, field testing and field monitoring.

(b) Definition of processes and activities within the scope of the management system;

(c) Organizational structure for all processes within the scope of the management system, including the responsibility and authority of organizations and personnel involved in their development;

(d) Definition of how the performance of all processes will be supervised, reviewed or verified;

(e) Description of planning and performance of audits and reviews;

(f) Management of documents, samples and records;

(e) Provisions for the training of personnel, including the review and verification of training activities;

(f) List of technical and administrative procedures to be applied, including references to procedures in the second tier of management system documentation.

7.5. Documentation in the second tier should normally be grouped into a manual of management and administrative procedures, and a manual of technical procedures.

7.6. Owing to the potential large variety of investigations and analyses to be performed in site geotechnical siting activities, technical procedures and instructions should be developed to facilitate the execution and verification of these activities. These procedures and instructions should normally refer to existing codes and standards, especially for field testing and laboratory testing[15].

7.7. Each procedure and instruction in the second tier of the management system documentation should include:

(a) Purpose and scope of the procedure.

(b) Definitions of terms with an uncommon or specific meaning.

(c) References.

(d) Responsibilities, in which the primary responsibility for successful outcome should be identified. The primary responsibility may be different from responsibilities for specific activities.

(e) Qualification and training requirements for personnel.

(f) Actions, or step by step instructions to be performed to achieve the purpose of the procedure.

(g) Documentation and reports to be produced.

(h) Necessary quality management records, and their classification in accordance with the management system manual.

7.8. Procedures should be prepared and reviewed by personnel with sufficient experience in the subject area.

7.9. To ensure document control, each document should be assigned a unique identification number.

[15] Many geotechnical correlations or methods use the results of standardized tests: departing from the standardized tests would invalidate these correlations.

Procedures should define how documents are numbered, and how obsolete documents are marked to prevent further use.

IMPLEMENTATION OF THE MANAGEMENT SYSTEM IN RELATION TO THE GEOTECHNICAL EVALUATION OF SITES FOR NUCLEAR INSTALLATIONS

Control of studies, evaluations and analyses

7.10. Studies, evaluations and analyses should be peer-reviewed by qualified individuals who have not participated in their specification or in their development, with the purpose of ensuring that the intended scope has been met, the technical approach and method of analysis are valid, and the results are correct. Evidence of the review work should be produced and kept as a quality management record in the project archives. The qualifications of the reviewers should be such that they could have competently performed the study, evaluation or analyses that they are reviewing.

Control of field activities

7.11. Field activities should be supervised to ensure that they are performed by qualified personnel, in accordance with established procedures and using specified equipment. Evidence of this supervision should be produced and kept as a quality management record in the project archives.

Control of samples

7.12. Procedures for identification and control of samples during handling, storage and shipping should address cleaning, packing, preservation and identification, in order to prevent the deterioration and loss of samples. The identification of samples of limited lifetime should include the date of acquisition and expected life.

7.13. The preservation of cores from subsurface characterization boreholes may be necessary for a long period of time, since they may need to be available for inspections by the regulatory body. The period of time during which the cores need to be preserved should be agreed in advance with the regulatory body and specified in the procedures.

Control of laboratory testing

7.14. Specified testing should be performed by registered laboratories that have been assessed as competent by the relevant national authorities. Appropriate current certificates of the laboratories should be kept as quality management records in the project archives.

Control of software

7.15. Commercial software for data acquisition, data processing, evaluations or analyses, used under a licence agreement with the developer, should be installed in accordance with the procedure provided by the developer and checked accordingly. Evidence of this check should be produced and kept as a quality management record in the project archives.

7.16. Commercial software developers should be considered as part of the supply chain to the site geotechnical investigation, characterization and evaluation, and the requirements provided in para. 7.2 should be met. Appropriate certificates of the software developers should be kept as quality management records in the project archives.

7.17. For non-commercial software and software developed internally, a verification programme should be devised and performed by qualified personnel before it can be used in the site geotechnical investigation and evaluation. Evidence of the verification work should be produced and kept as a quality management record in the project archives.

7.18. The verification of commercial and non-commercial software does not imply that the mathematical formulation implemented within the software is adequate to represent a particular configuration. The suitability of a piece of software should be assessed based on the available validation information.

Measuring instruments

7.19. The accuracy of measuring equipment should be maintained within prescribed design limits, to ensure the necessary reproducibility and traceability of results. Instrument calibration records should be kept as quality management records in the project archives.

7.20. Data processing software used in association with the recording/measuring instruments should be verified, as described in paras. 7.16 and 7.17.

Audits, non-conformances and corrective actions

7.21. Periodic audits by a team that is independent from the development team should be performed, to verify compliance with the procedures for geotechnical siting activities and to check the effectiveness of the management system.

7.22. The results of audits should be recorded, including details of non-conformances and the corrective actions derived from them. Reports from audits should be kept as quality management records in the project archives. The implementation of corrective actions should be kept under review, and the closure of non-conformances should be kept as quality management records in the project archives.

7.23. The frequency of audits will vary. However, at least one audit should be performed at the project mid-term, to ensure that conditions that might adversely affect quality are identified and corrected in time.

APPLYING A GRADED APPROACH TO THE MANAGEMENT SYSTEM IN RELATION TO THE GEOTECHNICAL EVALUATION OF SITES FOR NUCLEAR INSTALLATIONS

7.24. The application of a graded approach (see Section 6 of this Safety Guide) to the management system is also required (see Requirement 7 of GSR part 2 [11]). As described in para. 6.10, the application of a graded approach should be considered for nuclear installations in the ‘medium’ or ‘low’ hazard categories.

7.25. The application of a graded approach to geotechnical site evaluation should involve ensuring that the documentation and administrative effort is commensurate with the radiological hazard of the installation, while still observing the main safety objectives, as described in para. 6.3. Further recommendations are provided in paras. 2.41 and 2.44 of GS-G-3.1 [12]. For example, the application of a graded approach may result in the following:

(a) Supplier qualification documentation is accepted without further audits or third party certification;
(b) Review and evaluation are performed on a sample basis;
(c) The levels of approval of the documentation are reduced;
(d) Distribution lists are reduced or eliminated;
(e) Quality records to be generated and retained are reduced.

7.26. In whatever way a graded approach is applied, at a minimum, the management system should retain the following aspects:

(a) Definition of activities to be performed, with their input, output and main guidelines;
(b) The qualification and training requirements for personnel;
(c) The processes for review and evaluation of results;
(d) Document control.

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