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GOVERNMENT OF INDIA

AERB SAFETY MANUAL

FOR
CIVIL ENGINEERING & BUILDING WORKS
OF
NUCLEAR POWER PLANTS

ATOMIC ENERGY REGULATORY BOARD

AERB/SM-S-1

SAFETY MANUAL

FOR

CIVIL ENGINEERING AND BUILDING WORKS

OF NUCLEAR POWER PLANTS

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FOREWORD

The Atomic Energy Regulatory Board (AERB) has been given the responsibility to develop the Safety Codes, Guides, Standards and Manuals for Nuclear Power Plants (NPPs) in India. AERB has already taken up the development of safety codes in the areas of Quality Assurance, Design and Operation, which are expected to be issued shortly. The codes would establish the objectives and minimum requirements to be met to provide adequate assurance of safety during operation of Nuclear Power Plants in India. Safety Guides describing the acceptable methodology of implementation of the Codes would follow in due course.

The assurance of safety in Nuclear Power Plants is not limited to just meeting the basic safety objectives and requirements. These have to be backed up with systematic approach as well as good and safe practices evolved with long working experience in the field. Keeping this in view AERB envisaged the development of Safety Manuals in areas relevant to safety based on current Indian and international practices. The Manuals would provide quantitative values where possible and detail the good practices evolved by experience which may or may not have been specified in the Codes, Guides and Standards. The Manuals as such would act as a consolidated document to provide detailed safety information for guidance of the persons actively engaged in the area.

One of the areas in which AERB felt that abundant experience and expertise are available in our country and where relevant information exists but needs consolidation in a single document was the field of Civil Engineering and building works for Nuclear Power Plants. Shri V. Ramachandran, Chief Engineer (Civil) of Nuclear Power Corporation was entrusted by AERB to take up the preparation of this Manual to cover all Civil Engineering aspects of NPP relating to siting, design, construction, quality assurance and acceptance, maintenance and decommissioning. The preparation of this document was taken up in August 1986 and involved stupendous efforts by Shri Ramachandran and his colleagues in consulting national standards, consolidating experience in the DAE and information both published and unpublished currently available on the various relevant topics.

The "Safety Manual for Civil Engineering and Building Works for Nuclear Power Plants" was reviewed by expert members of the Working Group, who were drawn from various organisations in the country. Before finalising, the document was circulated to various Heads of Groups of DAE units and to outside experts. Their comments have been collated and appropriately incorporated.

The Manual specifies quantitative values for design wherever possible. In such areas where this was not feasible recommendations on design philosophy and objective have been made. Areas where further development work is needed have been identified.

AERB would like to thank all the members of the Panel who have put in their considerable efforts and also those who have so generously offered their comments and suggestions. AERB would like to specially express its gratitude to Shri V. Ramachandran for readily accepting the task and finalising the Manual in its present form.

This manual is issued by AERB as first version to solicit views of all concerned and its revised version would call for a wider review.

It is hoped that the Civil Engineering designers, quality assurance staff, contractors of NPP as well as the regulatory body personnel will find the Manual very useful and handy.

(A.K.De)

Chairman, AERB

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SAFETY MANUAL FOR
CIVIL ENGINEERING AND BUILDING WORKS
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Chapter 1 - SITING

1000 General

Evaluation of site related factors must ensure that the plant site combination does not constitute an unacceptable risk. Apart from the normal considerations that enter into the selection of sites for setting up thermal power stations, the requirements of site characteristics for Nuclear safety under normal and accident conditions is an additional factor in siting Nuclear Power Plants (NPP). The main aspects which have to be considered for assessing the suitability of the proposed sites for Nuclear Power Plants are as under:

1. Proposed sites shall be examined with respect to the frequency and the severity of man-induced events and natural phenomena that could affect the safety of the plant. Design Basis external man induced events relevant to the site shall be identified. For an external event (or the combination of events) the choice of values of the parameters upon which the plant design is based should be such as to ensure that structures, systems and components important to

safety in relation to that event (or the combination of events) will maintain their integrity and will not suffer loss of function during or after the design basis event.

2. The foreseeable development of the region having a bearing on safety should be evaluated for a period of 30 years to cover the life of the Nuclear Power Plant.

3. If, after evaluation, it is shown that the measures proposed cannot be regarded as providing adequate protection against design basis external events, the site shall be deemed unsuitable for the location of a Nuclear Power Plant.

1001 Earthquake and associated topics:
The seismology and geology of the region of the proposed site shall be evaluated.

In the various stages of the site evaluation attention should be given to:

1) Features that can have direct influence on the acceptability of the site.

2) Features that can substantially influence the severity of the design basis earthquake.

Details regarding following aspects should be collected.

i) Capable faults

ii) Slope instability

iii) Liquefaction potential

iv) Subsidence

v) Karstic phenomena

A preliminary estimate of the seismic activity of the site area should be obtained from the seismic zoning maps and from the maps and catalogues of past earthquakes. A major part of the information for determining the design basis earthquakes is a complete set of historical earthquake data extending as far back in time as possible. Most of these historical records will naturally be of a descriptive nature, including such information as the number of houses damaged or destroyed, the behaviour of population etc. which form a measure of intensity scale value of each earthquake to be evaluated. The intensity scale most commonly used is the Modified Mercalli Scale.

Since the intensity scale is damage related, allowance shall be made for changes in the type of construction from time to time in the evaluation of the intensities of the historical earthquakes. Allowance should also be made for a subjectiveness in the information obtained from old chronicles and individuals who may have experienced the event with due consideration to a tendency for exaggeration while reporting on the damage and other phenomena associated with them.

To the extent possible, data should be collected for all historical earthquakes within a region that includes the seismotectonic province of the site. This usually requires consideration of an area the radius of which depends upon the characteristics of the region. This radius is usually taken as 150 to 300 kilometres. The data to be obtained are:

Historic data -

(1) Intensity scale value at the epicentre or maximum intensity scale value.
(2) Intensity at the site
(3) Isoseismal maps
(4) Magnitude of earthquake
(5) Location of the epicentre, and, if known, the hypocentre.

Intensity scale values, building damage, and ground effects data, in conjunction with a knowledge of local faults, should be used to the extent possible to determine the epicentre and magnitude of each historic , non-instrumented earthquake.

Instrumental and reported data :

To the extent that it is available, the following information should be collected:

(1) Locations of epicentre and hypocentre
(2) Time of origin
(3) Magnitude of earthquake
(4) Aftershock zone
(5) Maximum reported intensity scale value
(6) Isoseismal map
(7) Ground motion intensity at the site
(8) Other available information that may be helpful in evaluating seismotectonics.

1002 Geological information:

Regional geological information provide the knowledge of the general geological setting and tectonic frame work of the region needed for interpreting the earthquake data and for defining the seismotectonic provinces.

The following regional scale information should be obtained:

1) Characteristics of the ground:

Where the geological maps are available special attention should be given to identifying lithologic units - e.g. crystalline, volcanic, sedimentary, alluvial etc.

2) Stratigraphy.

Superposition and age of strata, their lateral extent, possibly their depth, thickness and relationship to one another.

3) Regional tectonics.

Special attention should be given to faults. Topography and geomorphology may be useful for showing possible recent ground displacements. Consideration should also be given to the tectonic setup of the region e.g. horizontal continuity of strata, folding and faulting. The tectonic history should also be considered, particularly, the age of faulting and recent activity if any.

4) Characteristics of tectonic features.

Style and type of faulting in the region and large faults associated with seismotectonic provinces should be described. The extent and activity of the faults should be studied for information on the possible presence of seismically active or capable faults. Particular attention should be given to the evaluation of quaternary deposits and detailed neotectonic studies.

5) Subsurface characteristics.

Where there is no surface manifestation of baserock, and where appropriate data exist, a structural map of the baserock surface (hypogeological map) may be prepared. Information available from regional geophysical investigations, should be used to obtain the necessary subsurface details. This map may permit a determination of possible relationships between historic earthquake activity and deep tectonic structures, which may lack a direct expression at the surface.

1003 Site vicinity investigations.

The general geological and geomorphological characteristics of 500sq.km area should be examined. Geomorphological features, such as the nature of undulations,the inclinations of the ground surface, the conditions of streams,the patterns of erosion, the kinds of vegetation, and the conditions of drainage, or exposed rocks, may provide initial indications of the geological structure

1) The following subsurface investigations are required to be conducted in the site vicinity.

The possibilities of surface faulting, slope instabililty, surface collapse, subsidence or uplift etc. should also be assessed. The safe bearing capacity and settlement characteristics of the strata at the foundation levels should be assessed to decide on the foundations. Adverse foundation conditions add to the cost of foundations and extend time required for the construction.

2) Hydro-geological conditions.

In order to make an assessment of the site on geo-hydrological considerations information on the type of top soil, the depth of water table and its variation are required. Following aspects should also be considered.

i) Location of the nearest ground water source.

ii) Principal ground water discharge points, to surface water bodies.

iii) Depth of regional and local water tables.

iv) Ground water flow directions and gradients to determine pathways and travel times to accesible environment.

v) Proximity of site to principal regional aquifers and their recharge areas.

vi) Ground water related human activities that could affect site.

3) The ground water levels and their seasonal fluctuations are of importance in determining the leaktightness in the design of structures and foundations. Quality of the ground water also has some significance since presence of certain salts such as sulphates, chlorides etc. would require suitable precautions to be taken in the foundations to avoid

adverse chemical action. Hence, a study on water table variation for a period of at least one year is necessary. In the absence of any other data, available data from existing tube-wells or open wells could be used for preliminary investigations. The bores drilled at the site for geotechnical investigation should be observed to obtain information on water table level.

4) Access to site

The access road has to be designed to cater for the maximum loads in terms of weights and dimensions that have to be transported only by road for short distances from the rail head/port to site. The roads for these short distance should be able to carry higher axle loads than normally permissible on National or State highways. The weights and dimensions of the consignments for 235 MWe and 500 MWe single unit NPPs indicate that the maximum weight that can be considered for transportation by road is approx. 70Te and approx. 265Te by rail. The maximum width of the consignment for road movement is about 7.0 M. For transportation by rail, the size and shape of the consignment should fall within the prescribed profile for maximum moving dimensions permissible on the concerned sections of the railways.

For road movements on the National and State highways, the load carrying capacities of Cross Drainage(CD) works and the geometrics enroute shall be assessed. Where deficiencies are noted, improvements should be carried out. These roads can also serve as communication lines during plant emergency requiring evacuation.

The plant roads have to carry the heavy equipment during erection. Heavy cranes (650Te lifting capacity) are moved on these roads. Special care has to be taken during design of these roads, with particular reference to the crust thickness and road widths for such movements.

5) Flooding of sites:

Inland sites:

At inland sites, flooding could be caused by heavy precipitation or by the release of large quantity of water from upstream dams. For sites situated on the bank of a river course with no upstream water retaining structures, the site elevation shall be adequately above the maximum historic flood level. The actual site elevation of all safety related structures shall be fixed at the design stage above the Design Basis Flood (DBF) level at the site resulting either from Probable Maximum Precipitation (PMP) with a mean recurrence interval of 1000 years or otherwise as applicable.

The estimate of the water levels arising out of a Design Basis Flood discharge (1000 years mean recurrence interval -MRI) shall be arrived at using standard flood routing methods.

The level of seismicity considered at a location for the design of dams is generally different and lower than the criteria adopted for NPPs.

The designs of most major dams are approved by the Central Water Commission(CWC), Government of India. If the upstream dam is safe, when checked as per the current I.S. codes and the dam is in a sound condition as assessed by the Dam Safety Organisations then it can be considered safe, during the preliminary site evaluation. In all other cases, however, dam failure may have to be postulated and the corresponding flood water level at the site evaluated.The finished grade elevation at the site shall be above this level by an adequate margin.

Downstream dam failure:

Where the NPP is located on the bank of a reservoir, then the safety of the dam is of concern for meeting the necessity of an ultimate heat sink, if required.

Coastal sites:

The flooding of sites could be due to the following:

1. Wind waves
2. Tropical cyclones

The wind wave effects are significant in high tidal range regions such as Saurashtra and Kutch coasts, Gulf of Cambay, Northern Andhra coasts etc.

Coastal flooding is primarily due to tropical cyclones. On the East Coast of India, the frequency of occurrence of cyclones and their intensity is greater than on the West

Coast. On an average, 50 percent of these cyclones cross theEast Coast during October and November and 30 percent in the months of April to June. Much of the northern Bay of Bengal is shallow, providing a favourable condition for surge amplification. Studies on the evaluation of storm surges for the East Coast have already been carried out by the India Meteorological Department (IMD) which can form the basis of preliminary predictions of peak still water levels including the maximum astronomical tide, at a place, for the design basis cyclones.

Wave hindcasting techniques have to be adopted for arriving at the Design **Basis** water level due to the Design Basis Cyclone transposed to produce the maximum effect at the site.

From a survey of the data for flooding of coastal sites, a minimum elevation of 4.0 m above Astronomical Tide Level (ATL) on the East Coast and an elevation of 3.0 m above ATL for West Coast are considered acceptable for the preliminary evaluation of the site. However, detailed evaluation has to be carried out to arrive at the still water level at design stage.

6) External man-induced events:

The activities close to NPP site should be closely examined to ensure the safe operation of the plant. The man-induced events which have a bearing on the power plant operation are:

1. Aircraft impact

2. Storage and manufacture of inflammable corrosive, explosive and toxic materials.

3. Mining and blasting activities.

In order to limit the probability of an aircraft impact on the NPP to less than 10^{-6} event per year it is necessary to locate the site away from airports, landing and take off zones, and air corridors by a design screening distance value(SDV). A distance of 10 Km from major airports is considered reasonable looking to the current air traffic density and the projected growth in air traffic.

With regard to places where toxic and inflammable materials are manufactured or stored, the SDV is taken as 5 kms. For mining and blasting operations the SDV is taken as 5 kms. In coastal areas, the movement of oil tankers causing oil slicks should also be considered.

Chapter 2

DESIGN CRITERIA

2100 Subsurface Investigation

2101 General

Information on sub-strata is required to arrive at the bearing capacity and settlement characteristics of foundations due to static loads, earthquake response (amplitudes of vibration and overall stability), as well as to take into account uplift due to water table, stability of excavations, and interaction between neighbouring foundations, in foundation design and to decide on methods such as consolidation grouting to improve the founding media. Detailed field geotechnical investigation and laboratory studies are necessary to evaluate the performance of the subsurface material under the design loading conditions.

The subsurface investigation shall include the following ad seriatim:

i) Studies relating to the general and engineering, geology of the region and the site.

ii) Geophysical studies

iii) Bores, trial-pits, shafts, insitu tests, laboratory tests on soil and rock samples.

2102 Spacing and depth of boreholes:

The subsoil investigations enable to draw the soil profile indicating the characteristic soil properties at different depths.

Depending on the complexity of the natural deposits, in the direct method of exploration, bore holes should be such as to reveal any major changes in thickness, depth or properties of strata over the base area of the structure and its near vicinity. The spacing should be determined depending on the ground conditions and the importance of the structure.

These investigations are to be carried out to a depth beyond which the properties are no longer influenced by the superimposed loads so as to affect the safety of the plant. Such exploration should be taken

(a) below all deposits which may be unsuitable for foundation purposes, including cases where weak strata are overlain by a layer of strata of higher bearing capacity

(b) to a depth where the increase in stress is negligible in relation to the strength of the strata. In any case, the depth of the borehole should not be lesser than 1 1/2 times the width of the foundation.

A few boreholes shall be taken to a depth greater than the rest to gain a full appreciation of the soil profile and overall ground conditions. These holes should be completed first, so that subsequent exploration may be amended in the light of the information gained. The depth chosen should also take into account the need to determine geological conditions such as dipping strata, lenses, faults, buried channels, or extent of made up ground. It may also be necessary to prove the rock level, identify the extent of highly weathered rock, or investigate the possibility of voids or previous mine workings through these boreholes.

Where a considerable thickness of competent rock is expected at a site, a minimum of 6 m beneath the foundations should be proved by core drilling taking continuous cores.

2103 Methods Related to Ground Conditions

Depending on the types of ground conditions likely to be encountered, the method of exploration should be suitably selected. A general guideline is given below:

1) Normally consolidated or lightly overconsolidated clays:-

These clays are often too soft and unsuitable for foundations. However, they are to be carefully investigated as they influence the behaviour of structures/foundations. The methods of investigation to be used are undisturbed sampling, insitu vane shear test, static cone and Standard Penetration Test (SPT).

2) Overconsolidated Clays

These clays are generally of firm or stiff to very stiff consistency.

Large proportions of granular material will significantly increase the disturbance of the samples, particularly if the clays are at moisture contents below the plastic limit (PL).

These clays are very often fissured and therefore testing of undisturbed samples in the laboratory is not possible as they break up on extrusion from the sampling tube. In the absence of undisturbed samples, the shear strength may be broadly estimated on the basis of SPT N value. In-situ determination of strength and elastic modulus could be carried out using pressuremeter.

3) Fine-grained granular deposits

Deposits of silts and sands are generally investigated by means of in-situ tests within boreholes or by sounding methods from the surface (such as the static cone penetrometer), because these materials are difficult to sample and test.

Control of groundwater often plays an important part in construction works in fine grained granular materials, and observation wells should be installed, depending on whether the soil is under simple hydrostatic pressure or a more complex groundwater regime. In situ permeability tests may also be carried out.

Cyclic shear tests to assess the liquefaction potential are to be carried out.

4) Medium and coarse-grained granular deposits :

Trial pits are often extremely useful in this type of material, if above groundwater level, to enable the ground conditions to be examined closely. Determination of material properties is usually restricted to assessment of

SPT results, using the 60 deg.cone, and the inspection of retrieved disturbed small and larger bulk samples.

In clean river gravels, SPT results are generally reliable if precautions are taken to carry out tests in undisturbed ground.

Assessment of groundwater conditions is very important.The investigation should include an assessment of particle size distribution of the soils with permeability tests carried out in addition to normal groundwater observations, so that a dewatering scheme could be designed. Pumping tests are necessary where excavations below water table are involved.

5) Fill (Made Ground):

Filled up ground can contain disturbed natural material and various industrial or commercial waste products. The uniformity depends on the type of fill, degree of mixing, and type of supervision during filling operations. Most filled ground is extremely variable, and a great deal of exploration and testing is normally required to determine representative results.

Trial pits and trenches are often the most suitable methods for exploration of filled up ground, providing maximum opportunity for examination of the various materials.

Fills are generally unsuitable for supporting foundations unless special ground improvement techniques are adopted.

6) Weak Rocks

Drilling in weak rock requires a high drilling expertise and specially designed core barrels for full core recovery. Monitoring of the drilling rate, colour and quantity of flushing media returns, core quality using rock quality designation (RQD) and fracture index (FI) provide useful information. High core recovery is a major objective, because, it is the weakest and therefore the most important material which tends to be lost. Good core recovery is often dependent on the expertise of the driller and the condition of the equipment.
Tests carried out within boreholes include permeability tests by water injection methods (Packer tests) and pressuremeter tests in small diameter rock sockets. The permeability tests can be used to assess the fissuring and fracturing of rock.

Structural discontinuities, infilling, grain size, texture, and cementing agents are particularly relevant to the behaviour of rock.

SPT tests with steel cone will give additional information on the in situ characteristics of weak rocks.

Where doubt exists about the in-situ rock condition, particularly with regard to fissuring and fracturing, closed circuit television cameras can be lowered into the hole for inspection of the walls provided that the casing is not required at the depth to be investigated. Cameras that can operate at depths in excess of 30m in water-filled drillholes are available.

Accurate assessment of the engineering properties of weak rock is sometimes difficult using borehole techniques alone. In situ tests like block shear test, footing load test, block vibration test, etc. should be carried out.

7) Hard Rock

The drilling and investigations in hard rock are required to be carried out to prove the competency of the rock material. If the competent rock is encountered at lesser depths than those earlier anticipated, boring should penetrate at the greatest depth where discontinuities, zones of weakness or alteration can affect foundation and should penetrate at least 6 metres into sound rock.

Rock cores should be intact and should have high percentage of core recovery and good Rock Quality Designation(RQD) value. For RQD determination core should be at least 50mm in dia. (NX Size) and the recovery with double tube swivel type barrels is to be preferred.

2104 Report:

On completion of all the field and laboratory tests, a detailed report shall be prepared which shall include:

i) Plan to a scale of 1:1000 showing the position of all the bore-holes and various field tests.

ii) Results of Laboratory and field tests in the standard proforma (on soils, rocks and ground water).

iii) Details of the mode of investigation works carried out.

iv) Description of the various laboratory tests carried out.

v) Analysis of the test data and information of the properties of soils and rock as applicable.

vi) Recommendations regarding the allowable bearing capacity of soil and rock, anticipated settlement of foundations and type of foundations to be adopted for various structures based on the loading intensity, functional and safety requirements.

vii) Recommendations regarding excavation slopes, methods of excavation including blasting and dewatering.

viii) Any other information of special significance which is likely to have a bearing on the design and construction of the foundation.

2105 Geophysical and allied methods

For indirect estimation of in situ rock properties such as both shear and compressive wave velocities, depth to bed rock, ground water potential, shallow cavities and geological features like fault, shear zone, dykes, buried channel etc. geophysical methods like seismic, electrical, magnetic, gravity, cross-hole method and borehole logging can be most effectively used. These methods when used judiciously can give detailed sub-surface information within a very short span of time and are thus most effective in comparative studies of alternative locations at reconnaissance stage of exploration. The use of these methods in specific geological situation should be left to the specialists who may employ one or more of these methods to obtain maximum sub-surface information within the shortest possible time. It is important and necessary to corroborate geophysical results from a few bore holes in the area and any discrepancies could be remedied through modifications of interpretation techniques. When large areas are to be investigated for suitable location of critical facilities, geophysical methods could be used. These methods could thus be used for both reconnaissance and intensive exploration purposes with speed and economy. The methods generally employed are briefly described as below:

1. Seismic refraction method:

In cases of underlying shallow high velocity layer, the method gives depths to bed rock and seismic wave velocity from records of explosion or shock waves in a row of geophones arranged in a straight line. The various details such as length of seismic profile, position of explosion etc. depend on depth to bed rock and other geological factors.

2. Seismic reflection methods:

Normally, reflection method could not be usefully employed for shallow exploration, but presently suitable systems are available and the same can be used for exploration purposes. Reflection method gives depth to different layers.

3. Electrical resistivity method:

This method can be effective for exploration of shallow geological strata having resistivity contrast. Water bearing strata of lower resistivity can be identified easily. Similarly, thick overburden, crushed rock, shallow cavities etc. are other features where the method can be usefully employed.

4. Magnetic method:

This method is normally used for exploration of magnetic rocks, dykes etc.

5. Borehole logging method:

Various physical properties of rock in borehole such as acoustic velocity, electrical resistivity etc. can be directly measured for use in sub-surface exploration. Nuclear logging is also used.

6 Cross-hole method:

Direct seismic velocities can be measured by cross hole technique which is more straight forward.

7 Gravity method:

Gravity method is rarely used for shallow exploration but, for exploration of faults, shear zone and other features of larger density contrast, this method may be used in special cases.

8 Microtremor method:

A very high gain system can be used to record 'microtremor' or very low ambient noise. Variation of predominant frequency in 'microtremor' is associated with foundation condition. Thus a rapid microtremor survey could serve as an efficient reconnaissance tool along with geophysical survey.

2200 Design Basis Floods.

2210 Inland sites

2211 General

The potential for flooding is one of the site characteristics which shall be evaluated.

The most suitable protection against flooding is the construction of the plant at such level that it will not be affected by floods. Therefore, the preliminary evaluation of the flood level is extremely important.

For each site all potential sources of floods shall be considered. For the discussion of methodology , floods are divided into three kinds: viz. floods due to precipitation, floods due to release from natural or artifical storages and floods due to other causes..

Apart from methodology the most important parameter to be derived is the water level at the plant site. The Design Basis Flood(DBF) is always chosen to have a very low probability of exceedence per annum. Normally the DBF is not less than any recorded or historical flood occurrence.

ii) Hydraulic routing methods which additionally take dynamic effects into account.

These methods yield values of the flow. To convert these values into water levels, the model to be used can either be based on steady or unsteady flow as follows. i.e.For floods with relatively small rate of change of flow or stage, steady flow routing will approximate channel flow rates with sufficient accuracy. Unsteady-flow routing is applied when flow variation is very significant; it computes simultaneously the time sequence of both flow and water surface elevation over the full length of the water course.

Routing a flood through a reservoir is a special case of channel routing, and is usually approximated by steady-flow methods. Where it is necessry to divide a watershed into sub-areas, runoff models are connected and combined with stream course models.

2214 Floods from sudden release of natural/artificial storage:

Natural or artificial storage of large volumes of water may exist upstream of a site (e.g. water may be impounded by a man-made structure such as a dam, for power generation, irrigation or other purposes). The failure of such water-retaining structures, due to hydrological, seismic or other causes such as landslides into a reservoir, or dam deterioration with time, may cause floods in the site area.

General considerations:

The mode and degree of failure shall be postulated using conservative judgement on stability analysis. Postulation of the failure mode should take into account the type of dam construction and the topography of the river channel immediately downstream of the dam.

It is recognised that floods originating from dam failures could be increased by flood waves due to landslides into rivers and reservoirs, which could result from severe precipitation. Floods caused by dam failures must generally be combined with an appropriate flood from other causes to obtain the controlling flood. The appropriate coincident wind-wave activity (wave setup and wave runup) shall be superimposed on the flood still-water level that has been determined.

Seismic dam failures:

Flooding can result from dam failures caused by seismic events or from the consequences of seismic events such as landslides into a reservoir. Failure may occur of both manmade dams and naturally created water-impounding structures.

2215 Combination of events.

The design basis flood may result not from the occurrence of one severe event, but from the simultaneous occurrence of more than one less severe event.

The following combinations can be considered:

(a) PMF due to precipitation

(b) Dam failure caused by a Safe shutdown earthquake(S2) equivalent earthquake, coincident with peak of 25-year flood

(c) Dam failure caused by an Operating basis earthquake(S1) equivalent earthquake, coincident with peak of flood caused by one-half PMP

(d) Inadvertent opening of all gates on an upstream dam

(e) Inadvertent opening of all low-level outlets in an upstream dam coincident with peak of flood caused by one-half PMP.

2216 Evaluation of the highest water level:

After evaluation of the magnitude of the combined flood, the highest flood water level may be estimated, giving due consideration to the afflux effect.

2220 Coastal sites

2221 General

The Design Basis Flood (DBF) for siting nuclear power plants on coastal sites which a Nuclear Power Plant is designed to withstand the flood due to the following.

(1) The flood resulting from the probable maximum storm surge (PMSS).

(2) The flood resulting from a reasonable combination of severe events considering the wave effects.

2222 Methodology:

Two basic methodologies are available for determining the probable maximum flood-causing event (PMSS, together with wind-waves): One methodology makes use of the knowledge of the physical model of the phenomena (deterministic methods) and the other is based on the analysis of the historical data set, of actual measured water levels in the region (stochastic methods). The choice of which methodology to use depends on the availability of a large, complete and reliable set of historical data appropriate to the method and on being able to model adequately the relevant event. If sufficient information exists to permit the use of both methods, the results should be cross-checked.

2223 Site vicinity information:

Shoreline stability

A preliminary investigation shall be undertaken to determine whether a potential for shoreline instability exists. Shoreline erosion could affect items important to safety.Erosion and tidal current maps, aerial photographs and satellite imagery are useful in studying regional coastal erosion.

Storm Surges:

The potential for storm surges at a site should be assessed on the basis of meteorological and hydrological information; if a potential is found to exist, an evaluation of the storm surges at the site has to be made. Case studies of severe past storms in the region may be used to identify the following characteristics of the critical design storm that would produce surges at the site with a sufficiently low probability of being exceeded:

1) Minimum central pressure and associated peripheral pressure.
2) Maximum sustained wind speed.
3) Wind fetch.
4) Duration of storm and associated winds
5) Direction and speed of movement of the storm
6) The storm track and particularly the point at which the storm track is closest to or crosses the coast.

The height of PMSS can then be made using the values of these parameters as an input to an empirical relationship. These results should be compared with historical records of storm surges to check the suitability of the method used.

Flooding by Storm Surges:

If it is established that there is a potential for flooding due to a storm surge resulting from a tropical cyclone, extra-tropical storm or moving squall line a PMSS shall be evaluated. Where a combination of flood-causing phenomena includes a surge, that surge which is expected not to be exceeded in a given mean return period(25 to 100 years) may be used in the evaluation.

In open areas, the water-level rise due to a surge is usually represented by a single-peak surge generated by a wind storm.

PMSSs are generally evaluated by using the time-dependent mathematical equations of mass and momentum conservation for flow of an incompressible fluid.

Probable maximum storms:

The storm generating the PMSS, can depending on the site location and characteristics of the region, be the probable maximum tropical cyclone (PMTC). For each site the generating storm for the PMSS should be selected on the basis of analysis of the information. For details on the assessment computing the PMSS, a reference water level such as high tide or high lake level, of sufficiently low probability of being exceeded, should be assumed to occur coincidentally with the storm surge.

The analysis consists in selecting those appropriate storms and other relevant parameters (e.g. maximum wind velocity, atmospheric pressure differential, bottom friction and wind stress coefficients), to be used as inputs to a one-or-two dimensional storm surge model which maximises the flooding potential. All parameters are conservatively evaluated.

2224 Surge derivation for Open Coastal Regions

An appropriate validated model for calculating the PMSS should be selected. Experience has shown that a one-dimensional model is appropriate for most open coastal sites.However,if the configuration of the coast or the structure of the wind field is very irregular, a one-dimensional model may be inadequate; then a two-dimensional model is used that has already been accepted for this purpose or has been demonstrated to be conservative. The most severe combination of the meteorological parameters and the critical path of the cyclone and its rate of movement are determined by preliminary calculations. The reduction of cyclone wind speed when a storm moves over land is an important phenomenon to be taken into account.

It is possible that the cyclone generating the PMSS, still-water level may not represent the critical conditions for design. Other cyclones may generate lower peak surges but cause longer-duration high water levels, or may produce higher wind speeds and waves. The wave activity associated with these cyclones could conceivably produce higher design water levels. Also, for sites located within a bay, cyclones that would generate lower peak surges on an open coast but of longer duration could generate higher peak surges and more severe wave conditions within the bay, resulting in higher design water levels. Hence, cyclones other than those generating the peak open-coast surge but which could produce such effects as those just described should be considered.

2225 Wave Effects

Wind-generated water-waves (surface gravity waves) shall be taken into consideration in the flood analysis of coastal sites.

Near Shore Wave Effect:

The near shore waves critical for the design of the plant are identified by comparing the histories of various heights of incident, deep water transition and shallow water waves and limiting breaking waves taking into account the storm surge still water hydrograph. An appropriate range of still water levels shall be considered when selecting design wave conditions.

For each structure important to safety which is potentially exposed to coastal water action, the design wave characteristics shall be evaluated from the selected nearshore waves, taking into account the propagation of these waves to the base of the structure.

Available historical data on extreme waves (observed/ hindcasted and/or measured) for the region should be reviewed to verify the results of the analysis of off shore waves and near shore waves.

2226 Reference water level:

Reference water level shall be established for each flooding event or each combination of flooding events. Evaluation of the astronomical high tide (10% exceedence) should also be done.

2227 Combined flooding event and the Design Basis Grade Level:

The design water level due to the flooding events shall be evaluated by combining the extreme events such as surge, wind waves and reference still water level. The location of all safety related areas of the plant and structures should be above the Design Basis Grade Level.

2300 Seismic Aspects

2301 Information and investigation on earthquake:

Historical data

A major part of the information for determining the design basis earthquakes is a complete set of historical earthquake data. Therefore,it is necessary that the available historical records be collected, extending as far back in time as possible. Most of these historical records will naturally be of a descriptive nature, including such information such as the number of houses damaged or destroyed, the behaviour of population etc. But from such information a measure of the intensity scale value of each earthquake in modern macroseismic intensity scale values may be determined.

To the extent possible, data should be collected for all historical earthquakes within a region that includes the seismotectonic provinces of the site. The seismic zoning map of India, subdivides the country into five zones, with maximum seismicity associated with Zone -V. The method and procedures outlined here would apply to major portions of India which are well within Zone-IV. Sites in Zone-V and contiguous to the boundaries of Zone-IV and Zone-V will need special considerations in view of the increased seismicity and tectonic movements associated with them. These are outside the scope of this manual.

The current USNRC stipulations require consideration of an area of 200 miles(320 Kms.) radius of the site. The radius defining the area of interest depends upon factors such as earthquake potential, attenuation characteristics of the ground and the minimum level of peak horizontal ground acceleration set for the seismic design (0.1g) for the S2 level earthquake and could be in the range of 150 to 300 Kms. from the site. Due consideration should also be given to the S1 level earthquake in determining the region of interest in the semismotectonic evaluation.

The data to be obtained are:

(1) Intensity scale value at the epicentre or maximum intensity scale value, as appropriate

(2) Intensity at the site area

(3) Isoseismal maps

(4) Magnitude

(5) Location of the epicentre.

Intensity scale values, building damages, and ground effects data in conjunction with a knowledge of local faults should be used to the extent possible to determine the magnitude of each historic, non-instrumented earthquake.

Instrumentation and reported data:

To the extent available, the following information should be collected.

(1) Location of epicentre and hypocentre

(2) Origin time and duration.

(3) Magnitude

(4) Aftershock zone

(5) Maximum reported intensity scale value

(6) Isoseismal map

(7) Ground motion intensity at the site area

(8) Other available information that may be helpful in evaluating seismotectonics.

2302 Methods for deriving design basis ground motions:

Design basis ground motions shall be evaluated for each site. Two levels of severity are usually specified, S1 and S2.

The S1 is considered to be the maximum ground motion which reasonably can be expected to be experienced at the site area once during the operating life of the nuclear power plant with an estimated return period of 100 years or so.

The S2 level is the level of ground motion that has a very low probability of being exceeded (Estimated Return Period of 10,000 years) and represents the maximum level of ground motion to be used for design purposes of seismic category I safety related structures. The evaluation of S2 should be based on the seismotectonic approach and on the history of earthquakes in the region.

2303 The Seismotectonic approach:

Seismotectonic techniques consist of:

(a) Identifying the region, the seismotectonic provinces the seismically active structures and their maximum earthquake potential. The standard methods of investigation viz. geological historical and seismological normally specified for identification of active structures can be used.

(b) Evaluating the design basis ground motions produced at the site due to the postulation of the occurrence of the maximum earthquake at the point on the seismically active structure or at the borders of the seismotectonic provinces nearest to the site. If the seismically active structure is close to the site the physical dimension of the source may, if possible be taken into account.

The S2 should be defined by appropriate response spectra and time histories of motion. The motion should be defined for free field conditions at the surface of the ground.

Identification of seismotectonic provinces:

Usually, the purpose of seismotectonic studies is to define geographical regions within each of which similar earthquake potential exists.

Those seismotectonic provinces should be determined for the region that can have a bearing on the definitions of S2 ground motion for the site. The seismotectonic provinces will be the areas identified by similarity of geological structures and of the characteristics of seismicity.

The seismic and geological data should be developed into a coherent well-documented description of the regional tectonic characteristics. Those characteristics of tectonic structure, tectonic history and present-day earthquake activity which distinguish various seismotectonic provinces should be listed.

A number of precautions should be observed in defining the boundaries of seismotectonic provinces. All the structures in a contiguous area having the same seismotectonic or geological style should be included in the same province. Each tectonic structure relevant to the seismicity should in its entirety lie within the same seismotectonic province. When there is doubt that one structure is a continuation of another, then both should be considered as one structure and consequently the province should be considered as extending to contain both.

2304 Association of earthquakes with seismically active structures and seismotectonic provinces:

The fundamental data needed for associating earthquakes with tectonic structure and seismotectonic provinces must be collected and properly prepared.

1) Association of earthquakes with seismically active structures .

Whenever an earthquake epicentre or a group of earthquake epicentres can reasonably be associated with a tectonic structure, the rationale for the association should be given together with consideration of the characteristics of the structure, its geographical extent and its structural relationship to the regional tectonic framework. This assessment should include consideration of methods used to determine the earthquake epicentre and an estimate of the errors in their locations. A detailed comparison of these tectonic structures with others in the same seismotectonic province with regard to factors such as age of origin, sense of movement, and history of movement should be made. Other available seismological information such as source mechanisms, stress environments and aftershock distributions should also be evaluated. Tectonic structures with which significant seismicity is correlated should be considered seismically active.

For seismically active structures, which are pertinent to determining the earthquake potential of the site, the maximum potential that can reasonably be expected in association with these structures should be determined.

2) Association of earthquake with seismotectonic province:

The maximum earthquake potential, not associated with tectonic structures, that can reasonably be expected with a very low probability in the tectonic province should be evaluated on the basis of historical data and of the seismotectonic characteristics of the region. Comparison with similar regions where very extensive historical data exist may be useful. The current state of the art should look into geology and lithology of the seismotectonic provinces to establish similarity. However, there may be other factors to be considered which are not fully appreciated at the present time. Hence, considerable judgement is needed in such evaluation.

2305 Evaluation of ground motion at the site:

For the evaluation of the ground motion at the site two methods are available; one is based on intensity value and the other a more recent one on magnitude value. Intensity of the earthquake is a subjective local parameter since it is damage based. Hence, where possible, the results obtained by this method should be corroborated by the more quantitative approach using the magnitude method.

Intensity method:

The intensity method is adopted when the earthquakes are defined in terms of epicentral intensity. In this case, attenuation relationships in terms of intensity are derived from the isoseismal maps of past earthquakes in the region. It is very important to have a representative sample of such maps for earthquakes of the region to evaluate the dispersion of the data. Comparison of the curve derived from the region with existing curves evaluated in a similar geological region should be made. After using this method to obtain the intensity at the site, the maximum ground acceleration or velocity corresponding to the design basis earthquake should be obtained with proper acceleration intensity curves.

Magnitude method:

The magnitude method is adopted when the earthquakes are defined in terms of magnitude. In this case relationships exist which give the ground motion parameters (peak ground acceleration or velocity usually) as a function of magnitude and distance from the site to the seismic source or to the fault.

Application of methods:

In applying either of these methods, it should be noted that the ground motion observed at a site is not completely specified by a single parameter. It may be dependent on intensity, magnitude, source mechanism, distance from the source, site and transmission path characteristics, and duration. In selecting response spectra for use in design of Nuclear Power Plants, all these factors should be considered along with the fact that different earthquakes may produce the largest response in different frequency ranges.

2306 Induced seismicity

Special attention should be given to potential for induced seismicity particularly resulting from large dams or reservoirs, from extensive fluid injection into or extraction from the ground and from other mining operations.

2307 Derivation of the S2

The S2 shall be derived on the basis of maximum earthquake potential associated with tectonic structures and maximum earthquake potential associated with seismotectonic provinces in the region.

The S2 shall be determined taking into account:

1) The maximum earthquake potential inside the seismotectonic province of the site associated with specific tectonic structures.

2) The maximum earthquake potential inside the seismotectonic province of the site not associated with specific tectonic structures

3) The maximum earthquake potential for the adjoining seismotectonic provinces associated with specific tectonic structures

4) The maximum earthquake potential for the adjoining seismotectonic provinces not associated with specific tectonic structures.

The evaluation may be performed as follows:

1) For each seismically active structure the maximum earthquake potential should be considered to be moved to the appropriate location on the structure closest to the site area. For earthquakes close to the site the physical dimension of the source may be taken into account.

2) The maximum earthquake potential in the seismotectonic province of the site that cannot be associated with seismically active structures should be assumed to occur at a certain distance from the site. In certain countries this distance may be accepted on the basis of studies and investigations which ensure that within this distance there are no seismically active structures and therefore, that the related probability of earthquakes occurring therein is very low. This distance may be in the range of a few tens of kilometres and depends on the focal depth of the earthquakes in the province. In evaluating it, the physical dimension of the source will also be considered.

3) Maximum earthquake potential in seismotectonic provinces adjacent to the province of the site should be assumed to occur at the locations on the province boundaries nearest to the site.

4) An attenuation function should be used to determine the ground motion at site resulting from these earthquakes.

Various workers have proposed attenuation relationships for evaluation of peak ground motion (Milne and Devanport 1969, Schnabel and Seed 1973, Esteva and Villaverde 1973, Nuttli 1974, Trifunac 1979, Magure 1977, 1978, Joyner and Boore 1981 and other workers). Bolt and Abrahamson(1982) indicated that the peak ground acceleration near to the source (distance less than 10 Km) for earthquake in the magnitude range 5.0 to less than 7.0 do not show any significant variation and that peak acceleration at the epicentre reach an upper threshold in relation to earthquake size. A least square regression for data in magnitude range $6.0 \leq M \leq 7.7$ was found by Bolt and Abrahamson as below:

$$a = 1.6 \; [(X + 8.5)^{2} + 1]^{-0.19} \exp [-0.026 \, (X + 8.5)]$$

Where `a' is peak horizontal acceleration in g and x(Km) is closest distance to the surface projection of the rupture(focus). For x=40 Km, peak horizontal acceleration evaluated from this relation is found to be 0.1 g.

2308 Design basis ground motion characteristics:

The design basis for S2 should form the basis for developing the design basis.

1) Response spectra for various damping coefficients

2) Time history or histories.

1) Shape of the response spectra:

Response spectra should be developed from strong motion time histories recorded at site. However, in the majority of the cases, records are not available at the site and cannot be obtained in a reasonable time. Therefore response spectra developed from strong motion data obtained from sites having similar seismic, geological and subsurface characteristics are used. In this case, it is important to define the site conditions by those parameters which are most related to site response.

An alternative method uses a standard response spectrum with a relatively smooth shape which is generalised for application and which has been obtained from many response spectra derived from records of past earthquakes. Examples of such spectra shape are shown in Appendix A. However, in certain parts of the world higher values (in the high frequency range for rock sites and in the low frequency range for soil sites) have been observed so that some modifications of this spectrum may be required.

In both methods, the design response spectrum is obtained from the response spectrum shape after fixing the zero period ground acceleration. Such design response spectrum should be frequency rich and ensure conservation in the frequency band of interest to the design.

Time History:

Till such time a matching time history for 5% critical damping of the standard response spectrum is specified, the spectra compatible accelerogram shall be established for each site to match the design response spectra for 5% critical damping. The matching characteristics of such time histories viz. duration, rate of zero crossing, rise time etc. shall be decided after consulting expert opinion.

Derviation of S1 (OBE):

The need for considering a level of ground motion at the site defined by S1 exists for the design of safety related structures. For normal structures designed according to the current IS i.e. IS 1893 - 1984, the levels of seismic design indicated are adequate. The generation of floor response spectra required for seismic qualification of components and systems may require the establishing S1 level earthquake for which the following approach may be adopted.

The S1 is derived on the basis of historical earthquakes that have affected the site area.

The S1 level may be fixed on the basis of a probabilistic approach. The approach may also be combined with seismotectonic considerations to take into account the uncertainties of the data set.

The S1 level event has a return period of over a few hundred years.

Ratio of motion in vertical and horizontal directions:

It is recommended that the design response ground spectra and design time histories for vertical motion be evaluated using the same procedure as for the horizontal direction. If no specific information is available on the peak acceleration of vertical ground motion at site, it may be taken as 2/3rd of the peak horizontal ground motion.

An alternative method establishes standard vertical response spectra shape based on statisticalanalysis which is given in Appendix B.

2400 Meteorological aspects

2401 General

The atmosphere may be approximately characterized by meteorological variables such as wind speed, air temperature, precipitation and moisture content. Such values may be derived from routine meterological observations.

The procedure to be adopted for establishing design bases is as follows:

1) The meteorological phenomena and variables are described and classified according to their effect on safety.
2) Data sources are identified and data are collected.
3) Meteorological variables such as air temperature wind, velocity etc. are analysed to determine their design bases.
4) As appropriate, the design basis value for the variable, or the design basis for the phenomena, is defined.

2402 Extreme meteorological phenomena

Two types of data should be collected for extreme meteorological phenomena.

Data systematically assembled by specialised organisations in recent years will include more events for lower intensity and be more reliable than historical data.

Historical data, obtained from a thorough search of information sources such as newspaper and local records. From this type of data and by using a qualitative scaling system for each phenomenon, a set of events and their associated intensities may be collected for the region. These data are likely to be:

a) very scarce in the range of the low severity events;
b) relatively dependent on population density at the time;
c) subjectively classified at the time of their occurrence, thus making it difficult to assign the appropriate intensity level in each case.

On occasions, a comprehensive collection of data may have been made soon after the event had occurred. These data should include measured values of variables, eyewitness accounts, photographs, descriptions of damage and other qualitative information which was available shortly after the event. Such detailed studies of rare events help in constructing a model of the occurrence and in conjunction with a known climatology for a particular region, may contribute in determining the design basis event for that region.

Often the actual area affected by an extreme meteorological phenomenon is comparatively small, which makes accumulation of relevant and adequate data extremely difficult to achieve in practice.

2403 Evaluation of design basis

To evaluate the design basis for the extreme meteorological phenomena, two basic methodologies are available. One is based on the knowledge of the fundamental physical characteristics of the phenomena (the deterministic method) and the other is based on a statistical analysis of the historical data (the probabilistic method). The choice of which method to use depends on the degree of development of the physical model for the relevant phenomena and on the completeness of the historical data, both in quality and quantity. If sufficient information exists to enable use of both methods, it would be prudent to cross-check the results of one with the other.

1) Extreme winds

Wind speeds are generally measured and recorded routinely at meteorological stations that are manned at all times or that have devices which are continuously recording. The principal data concerning extreme winds are obtained from these stations. If possible, data for a maximum 3 sec.gust and 60 sec. sustained wind speed should be recorded.

Normalization of data to standard height and to standard duration:

Not all wind data are collected at the same height above the ground; the height may vary from station to station; even for one station, data may be collected at different heights during different periods. In these cases, the data should be normalized to a standard height (usually 10 m above ground level).

Design basis wind:

Based on the normalized data appropriate for the plant site the "expected extreme wind speed" associated with a return period of 100 years is determined.

If the design basis values are evaluated by statistical technique, due regard should be given to the physical limits of the variable that can be experienced in the area of interest.

2) Extreme Precipitation value:

For the evaluation of extrme precipitation, the variable to be considered is the maximum depth of the precipitation during the running 24 hours period.
Two distinct approaches are used to evaluate extreme precipitation. The first one evaluates by probabilistic methods the extreme precipitation for a particular site, and

is associated with a specific Return Period(RP). This method may be used to provide information on the precipitation at the Nuclear Power Plant site, as it affects particular aspects of the design, such as total depth of accumulation, site drainage, roof run-off, water loading, etc.

The second is a deterministic approach involving the modelling of an event for arriving at the probable maximum precipitation. This is the amount of precipitation that would result in an estimated depth of water, which for a given duration, drainage area and time of year, has virtually no risk of being exceeded.

Design basis precipitation :
For establishing design basis precipitation values in the design of the drainage system of the site and the roofs of the Nuclear Power Plant structures the following approach shall be adopted:

The "expected extreme precipitation" associated with an appropriate reference time interval is used in designing the roofs of structures. The "low probability extreme precipitation" is evaluated for design of other items such as drainage systems. The associated probability level and the reference time interval are selected on the basis of the relevance of the precipitation to safety.

3) Design basis temperature

Temperatures are recorded continously at some recording stations and at frequent intervals at some 24-hour manned stations. At other locations, at least daily maximum and minimum temperatures are recorded. Such routinely collected data provide long-term records for analysis to determine extreme temperatures.

Selection of data:

An on-site measurement programme shall be conducted for obtaining the site data to be compared with data from existing meteorological stations of the region. By means of such a comparison, it is possible to identify stations which have long term meteorological conditions similar to those of the site. An on-site programme may not be necessary if it can be demonstrated that there is a sufficiently dense network of meteorological stations measuring the temperature in the region,and that the site has no peculiar micrometeorological characteristics.

The daily maximum and minimum temperatures for an entire year represent the data sub-set from which the extreme annual values should be selected to characterize the annual maximum and minimum daily temperatures. These values form the data set which should be analysed for extreme design values.

As is done when analysing other meteorological phenomena, the beginning of the meteorological year should be selected so as not to coincide with a season during which the temperature attains the extreme value. This will avoid arbitrary assignment to different years of the data from a single such season.

2404 Statistical analysis

Extreme temperatures generally follow the Gumbel distribution. However, caution should be exercised in attempting to fit the extreme value distribution to a data set representing a season or less, since for periods of less than a year the hypothesis on which the extreme value distribution is based may not apply.

2500 Liquefaction of Soils

2501 General

One of the major causes of destruction during an earthquake is the failure of the ground. The ground may fail due to fissures, abnormal or unequal movements, or loss of shear strength. The loss of shear strength may take place in sandy soils due to an increase in pore pressure. This phenomenon, termed liquefaction, can occur in loose saturated sands. Soil that has lost shear strength behaves like a viscous fluid. Liquefaction often appears in the form of "sand fountains" during earthquakes. When soil fails in this manner, a structure resting on it simply sinks into it. Examples of damage or ground failure caused by liquefaction include:

1) Settlement and tilting of buildings
2) Floating of buried structures
3) Major landslides
4) Failure or significant lateral movement of waterfront-retaining structures, dams and embankments.

Therefore, potentially liquefiable soil layers at a Nuclear Power Plant site shall be identified, their characteristics evaluated, and the potential of their liquefaction during the postulated seismic event assessed. Appropriate field and laboratory investigations are required to identify and evaluate the static and dynamic characteristics of these soil layers; similar investigations should be made on fill material that may be used for the support or embedment of seismic class I structures and foundation(designed for SSE).

2502 Factors affecting Liquefaction Characteristics

The factors that affect liquefaction characteristics of sands are the following:

1. grain-size distribution
2. density of deposit (initial relative density)
3. vibration characteristics
4. location of drainage and dimensions of deposit
5. magnitude and nature of superimposed loads
6. method of soil formation (soil structure)
7. period under sustained load

8. previous strain history
9. entrapped air

The effect of each of the above factors is discussed below:

1) Grain-size distribution of sands:-

Fine and uniform sands are believed to be more prone to liquefaction than coarse sands. Since the permeability of coarse sand is greater than that of fine sand, the pore pressure developed during vibrations dissipates more easily. Also, uniformly graded sands are more susceptible to liquefaction than well-graded sands.

2) Initial relative density:

Initial relative density is one of the most important factors controlling liquefaction. Both settlement and pore pressures are considerably reduced during vibrations with the increase in initial relative density. Sands having smaller initial relative density will undergo larger strains and greater settlements than those having higher initial relative density. Chances of liquefaction are therefore reduced with increased relative density.

A stratum with relative density greater than 65% is considered safe against liquefaction.

3) Vibration characteristics:

Liquefaction and settlement depend on the nature, magnitude and type of dynamic loading. The whole stratum may be liquefied at the same time under shock loading, while liquefaction may start from the top and proceed downward under steady-state vibrations. Under steady-state vibrations, the maximum pore pressure develops only after a certain number of cycles have been imparted to the deposit. In general, it has also been found that horizontal vibrations in dry sand lead to larger settlements than vertical vibrations.

Multidirectional shaking as in an earthquake is more severe than unidirectional loading. Under multidirectional shaking or stress conditions, pore-water pressures build up faster than under unidirectional stress conditions.

4) Location of drainage and dimension of deposit:

Sands are more pervious than fine-grained soils. However, if a pervious deposit has large dimensions, the drainage path increases and, under quick loading during an earthquake, the deposit may behave as if it were undrained. Therefore the chances of liquefaction are increased in such deposit.

5) Magnitude and nature of superimposed loads:

An isotropic stress condition constitutes the initial effective stress on a sample. To transfer a large initial effective stress to the porewater, either the intensity of vibrations must be large or the number of particular stress cycles must be large. Hence, large initial effective stress reduces the possibility of liquefaction.

6) Method of soil formation: (Soil structure)

Sands are generally considered not to display a characteristic structure as fine grained soil. But recent investigations have demonstrated that liquefaction characteristics of saturated sands under cyclic loading are significantly influenced by the method of sample preparation which influences soil structure.

7) Period under sustained load:

The age of a sand deposit may influence its liquefaction characteristics. A study of the liquefaction of an undisturbed sand and its freshly prepared sample indicates that the liquefaction resistance of undisturbed sand may increase by 75%. This strength increase is due to some form of cementation which may occur at contact points between sand particles and is being associated with secondary compression of soil. This effect must be recognized as different from that due to orientation of soil particles in the soil fabric.

8) Previous strain history:

Sands may be subjected to some strains due to earthquakes. It was found that liquefaction characteristics were influenced favourably by the strain undergone previously.

9) Entrapped Air:

If air is trapped in water in which pore pressures develop, part of it is dissipated due to compression of air. Hence, trapped air helps to reduce the possibility of liquefaction.

2503 Methods of evaluation

The liquefaction potential at a site can be evaluated by using either an empirical approach or an analytical approach coupled with appropriate laboratory tests. Each approach requires appropriate field tests to be conducted.

A common empirical approach is based on the correlation of observations from past earthquakes for a wide range of conditions and the use of a simplified procedure for calculating the level of stresses induced by the postulated design earthquake. Other empirical approaches, also relying on observations from past earthquakes, are available but are valid for the more limited range of site conditions and seismic excitation upon which they are based.

Analytical methods for evaluation of liquefaction potential comprises the following steps:

1. After establishing soil conditions and the design earthquake, determine the time history of shear stresses induced by earthquake ground motions at different depths within the deposit.

2. By appropriate weighting of the stress levels involved in the various stress cycles throughout the earthquake, convert the stress history into an equivalent number of uniform stress cycles and plot the equivalent uniform stress level as a function of depth. By this means, the intensity

of ground shaking, the duration of shaking, and the variation of shear stress with depth within the deposit are taken into account.

3. By means of available field data or laboratory soil tests on representative samples, conducted under various confining pressures, determine the cyclic shear stresses that would have to be developed at various depths to cause liquefaction in the same number of stress cycles as that determined in step 2 above to be representative of the particular earthquake under consideration. Either cyclic-load-triaxial compression tests or cyclic-load-simple-shear tests may be used for this purpose.

4. By comparing the shear stresses induced by the earthquake with those required to cause liquefaction, determine whether any zone exists within the deposit where liquefaction can be expected to occur (induced stresses exceed those causing failure).

2504 It is recommended that for sites in high seismic regions, both the analytical and as a check, the empirical approach be used to the extent possible. For sites of relatively low seismicity, the empirical approach is generally sufficient. The use of detailed geological information and past earthquake history of the region can be extremely useful in the final evaluation. The final determination of liquefaction potential often requires considerable judgement based on synthesis of all information available.

2505 Factors of safety

A factor of safety against the occurrence of liquefaction is computed as the ratio of the applicable available cyclic strength (based on the stress required to cause a specified level of strain) to the induced stress. This ratio should be greater than unity in all critical soil layers and fill. The minimum acceptable value of this safety factor should be decided on a case-by-case basis. (Generally, a minimum safety factor of about 1.5 should be used).

2506 Remedial measures

If the analysis as discussed indicates the possibility of liquefaction, improvements to the strata to eliminate the possibility of liquefaction should carried out. The most effective method known is improving the density of the strata by compaction using measures such as vibroflotation, dynamic compaction, compaction piles etc.

If a pile foundation is adopted, driven piles should be preferred.

2600 Safe Blast Charges for Rock Excavation

2601 Blastings for excavation in hard rock generate ground vibrations of considerable intensity in the vicinity. These ground vibrations above the threshold limit may cause damage to structures. It is thus essential to design safe blast charges so that ground vibrations are kept below threshold limit to avoid damage to nearby structures.

There has thus been widespread interest for collection and assessment of safe ground particle velocity data so that the same may be used for design of safe blast charges per delay of detonation. From numerous observations world over, normally, ground particle velocity of 30 mm/sec has been found to be a threshold limit for ordinary structures. Any ground vibrations above this threshold limit has been found to be unsafe. For critical and important structures, this threshold limit has been rated between 5 to 10 mm/sec. In very special circumstances, threshold limit of ground particle velocity may be as low as 2 mm/sec. In case of 'green concrete' for structures under construction, this threshold limit may be 2 mm/sec for important structures.

Corresponding safe blast charges per delay of detonation may be obtained from the following equation:

$$v = K.Q^{m} R^{-n}$$

where v = ground particle velocity (mm/sec)

Q = blast charge in Kgs.

R = distance between structure and blasting area in metres

K, m, and n are constants depending on rock and other factors.

Where excavations near important structures are involved, it is necessary to assess the values of m, n and k through designed blast experiments in similar geology. Thus safe blast charges `(Q)' per delay of detonation can be computed from the above equation knowing the distance betwen blasting area and the structure. In the above scheme, `v' may be taken depending on importance of the structure as mentioned above. Where millisecond delay detonators are used, ground vibrations are reduced substantially. 'Presplitting' is another technique that has also been used for reduction of ground vibrations by creating a barrier to the transmission of the blast wave.

It is always preferred to complete excavation beforehand in hard rock within at least 100m of the proposed site of any important structure so that the structure may not be subjected to the risk of harmful vibrations above threshold limit, later.

APPENDIX-A

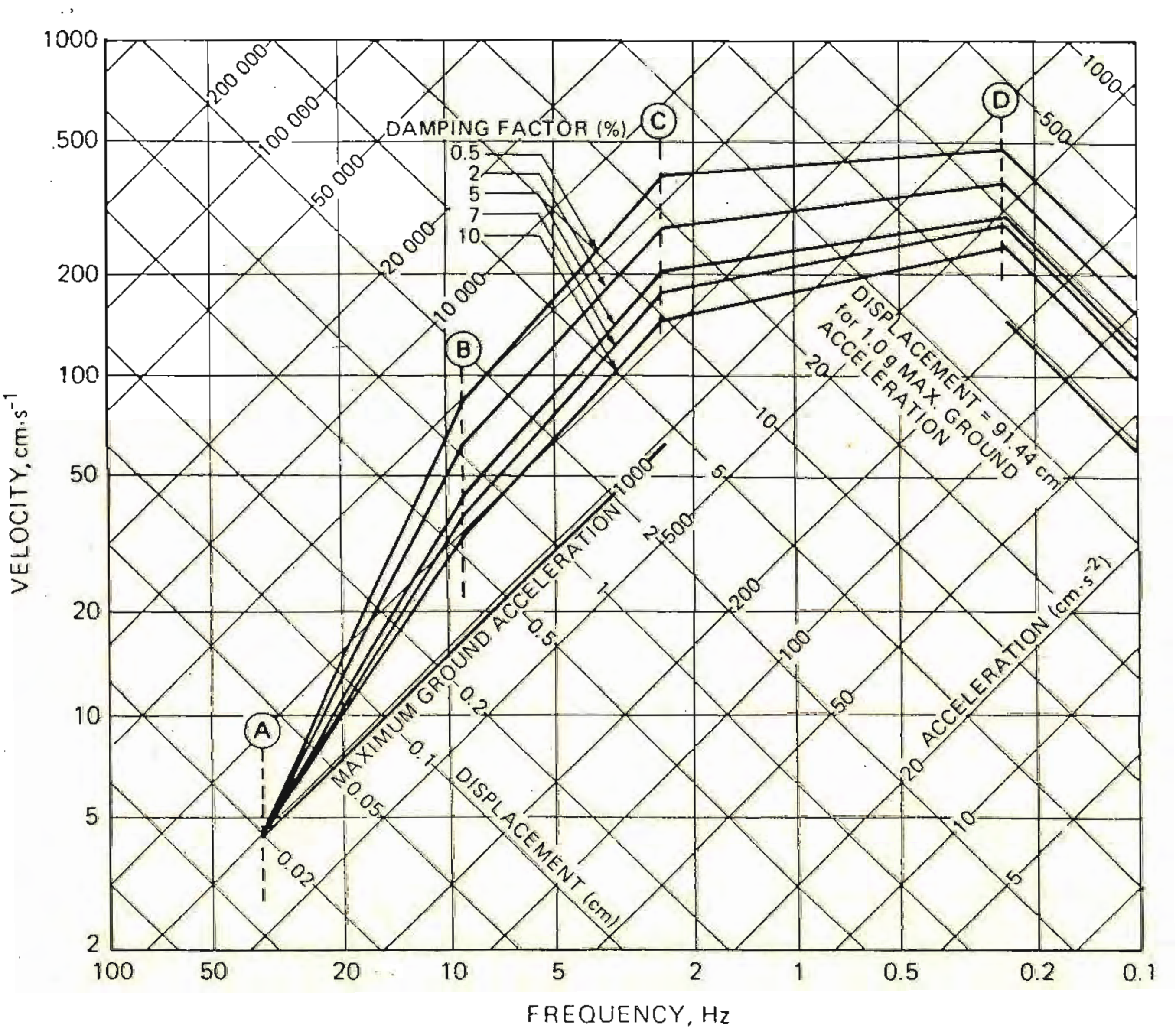

FIG.B1. Horizontal design response spectra. Letters A – D refer to the frequencies given in Table B1.

APPENDIX–A

STANDARD RESPONSE SPECTRA

TABLE B1. HORIZONTAL DESIGN RESPONSE SPECTRA (see Fig. B1)

Relative values of spectrum amplification factors for control points

Percent of critical damping	Amplification factors for control points			
	Acceleration			Displacement
	A (33 Hz)	B (9 Hz)	C (2.5 Hz)	D (0.25 Hz)
0.5	1.0	4.96	5.95	3.20
2.0	1.0	3.54	4.25	2.50
5.0	1.0	2.61	3.13	2.05
7.0	1.0	2.27	2.72	1.88
10.0	1.0	1.90	2.28	1.70

Note: **The maximum ground displacement is taken proportional to maximum ground acceleration based on 91.44 cm for ground acceleration of 1.0 gravity.**

APPENDIX–B

TABLE B2. VERTICAL DESIGN RESPONSE SPECTRA (see Fig. B2)

Relative values of spectrum amplification factors for control points

Percent of critical damping	Amplification factors for control points			
	Acceleration			Displacement
	A (33 Hz)	B (9 Hz)	C (3.5 Hz)	D (0.25 Hz)
0.5	1.0	4.96	5.67	2.13
2.0	1.0	3.54	4.05	1.67
5.0	1.0	2.61	2.98	1.37
7.0	1.0	2.27	2.59	1.25
10.0	1.0	1.90	2.17	1.13

Note: The Maximum ground displacement is taken proportional to maximum ground acceleration based on 91.44 cm for ground acceleration of 1.0 gravity. Acceleration amplification factors for the vertical design response spectra are equal to those for horizontal design response spectra at a given frequency, whereas displacement amplification factors are two-thirds of those for horizontal design response spectra.

APPENDIX-B

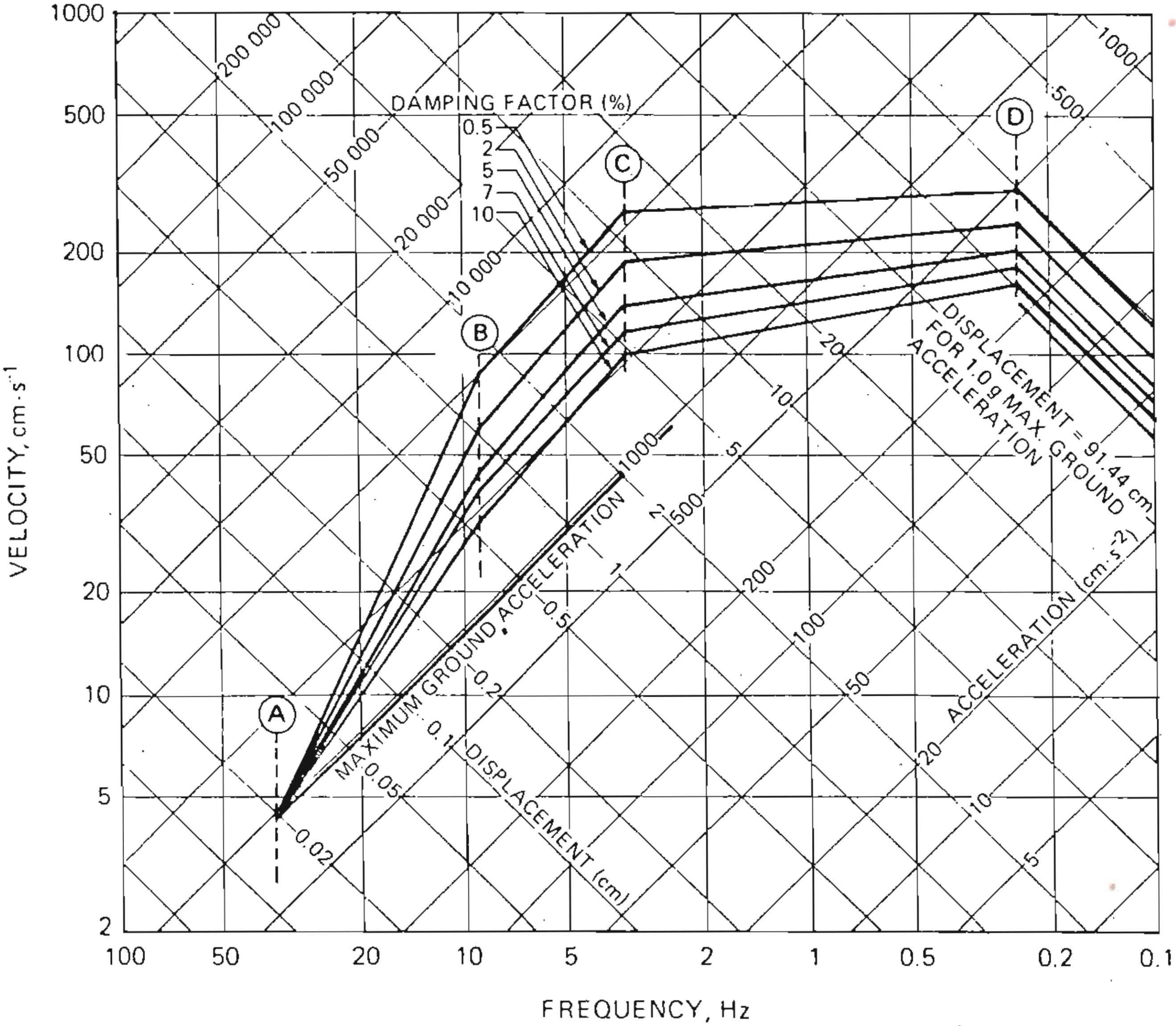

FIG.B2. Vertical design response spectra. Letters A – D refer to the frequencies given in Table B2.

Chapter 3

DESIGN APPROACH

3100 Design Aspects

3101 General

The design of Civil/Structural installations have significant influence in achieving the overall nuclear safety. Service requirement of the principal structures/buildings of a nuclear power plant are :

a) to provide sound operating conditions of plant/equipment/systems.

b) to protect them from hazards -internal and external

c) to contain the harmful effects of the normal operation and those during the failure of systems within acceptable limits.

3102 In their safety based design concept, possible events of failure are first postulated and the entire design is then carried out so that the structural system is adequate to prevent and/or mitigate the consequences of such failures.

For this purpose, the nuclear safety requirements are first identified and structures are classified based on the their role insatisfying these requirements. The postulated events which may result in accidents are then defined in respect of such structures. Radiological consequences of the accidents are then analysed to specify the design requirements for the buildings/structures.

3103 To achieve the goal, the design criteria shall be derived from the following Safety Requirements.

SR1 : Integrity and the leak tightness of the containment systems.

SR2 : Capability to shutdown (controlled) the reactor and maintain it in a safe condition.

SR3 : Capability to prevent or mitigate the consequences of the Design basis Fault(DBF) which could result in unacceptable exposure of radiation to the public.

3104 Layout:

The plant layout has to meet a variety of site conditions. The safety related buildings of a NPP comprise mainly of,

1. Reactor building
2. Reactor auxiliary building
3. Control building
4. Diesel Generator building
5. Spent Fuel Storage building
6. Service building, waste management area, ventilation stack and Heavy water upgrading area.

By proper placement of the safety related buildings in relation to the other plant buildings, they can improve the safety effects of man induced and other events.

3105 The design of the individual safety related buildings shall consider,

1. An internal arrangement of the systems to permit their categorisation in a single seismic class.

2. The ventilation zone for different areas shall have their ducting routed such that ~~ducts~~ carrying air from a higher radiation zone do not pass through a lower radiation zone.

3. Direct and easy escape routes with reliable lighting and other building services for the use of the plant personnel during emergencies.

4. Access control to ensure the effective control of personnel movement with a view to preventing the spread of activity within the plant and outside. For this purpose, adequate monitoring, wash and change facilities shall be provided with clear demarcation or barricades between the various radiation zones.

5. Personnel access to the reactor building through air locks equipped with sealed interlocked doors to ensure that separation of the containment air from the outside environment is achieved at all times.

6. Adequate fire protection through the use of fire resistant construction. Any new material of construction should be qualified for its fire resistance before its adoption on the construction. One of the effective means of protection to the reinforcing steel in reinforced concrete construction is through the provision of suitable concrete cover to the members as given in Annexure I.

7. All penetrations through the containment shall be engineered for high integrity to meet the leak tightness requirements of the containment.

8. Proper segregation of the plant areas to be achieved consistent with the plant safety requirements.

In the spent fuel storage and inspection bay structures the watertightness requirements of the concrete structure against the ingress of ground water shall be ensured through the adoption of the best methods of concrete construction followed by a rigrous qualifying watertightness test including the adoption of methods such as controlled concrete, waterproofing, injection grouting etc. besides the careful engineering of the construction joints.In spite of the metallic lining which ensures the leaktightness of the bays for exgress of water from the tank, a leak detection and control system should be incorporated where necessary.

The provisions of relevant IS codes shall be met in the design of safety class structures and where such codes do not cover a particular aspect, internationally accepted criteria should be followed. Design loads and material properties are the primary design parameters for the analysis and design work. The design values shall be determined to suit structural and safety functions.

3106 The general guidelines of the loadings which are to be considered are given below:

1)	Live load	L
2)	Dead load	D
3)	Equipment load	EL
4)	Construction load	CL
5)	Soil pressure	SP
6)	Pressure force	PF
7)	Effect of shrinkage and creep of concrete	LS
8)	Effect of heat of hydration	LH
9)	Environmental loading	
	Seismic	
	* SSE	ES
	* OBE	EO
	* Conventional (codal)	SC
	Wind	
	* Extreme wind	WE
	* Conventional wind	WC
	Flood	
	* Hydrostatic Load due to external flooding	F1
	* Hydrostatic Load	F2

due to internal flooding

Load due to solar radiation effect SR

10) Man induced impact loadings:

Missiles due to transport MT

* To land, water and air transport

Missiles due to external events other than transport due to wind or Turbine and other component disintegration ME

11) Loadings due to postulated initiating events

Loading due to internal missiles MI

Loading due to jet impingement and pipe whip and pressure transient MP

Pressure P

* Test pressure

* Operating pressure (internal)

Temperature T

* Test

* Operating

Hydrodynamic loading H

12) Effect of Prestressing:

Load combination based on initial prestress load shall consider losses due to:

* Friction and Wobble

* Elastic shortening of structures

* Jack and lock off losses

In the load combination of later life, further losses are considered which arise from:

* Concrete creep

* Concrete shrinkage

* Temperature effect
* Relaxation of steel.

3107 Load combinations:

The design shall cater to the most adverse credible combination of loads. Those loads that are common to any civil engineering building/structure shall be combined in the usual manner consistent with the IS Codes and the permissible increase in the allowable stresses as specified therein. Under earthquake conditions the allowable stresses shall be as given in Annexure III.

However, in the case of loads arising from extreme environmental and postulated initiating events, the probabilistic evaluation to assess the credibility of the particular combination shall be adopted. In such an assessment, the order of magnitude of the probability of a Loss of Coolant Accident (LOCA) shall be accepted as 10^{-3} event per year. The credibility test for the combination of loads shall be the limiting combined order of magnitude probability value of 10^{-6} event per year, where a single safety related unit is involved and the value of 10^{-7} event per year where the entire plant is affected involving more than one safety related unit.

3108 Design approach

The design of various safety related structures should be carried out after a detailed analysis of the structural systems has been completed for various loads and their combinations. The detailed analysis should comprise of static as well as dynamic analysis corresponding to static and dynamic loading conditions.

Static and Dynamic analysis:

The Static analysis for various static loads such as dead load, live load, equipment load, erection load etc. should be carried out considering the appropriate structural configuration. Suitable parameters representing material properties should be used considering various design loads and their combinations. The provisions in the Indian Standard should be followed. For the purpose of evaluation of responses due to dynamic loads is an approximation which has to be examined with reference to its influence on the results of the analysis and should be a design based decision.

Damping:

Damping is the most important constant used in response analysis of structures under dynamic loading. However it is the most difficult one to predict analytically and to this date damping levels have been determined from observations and experimental measurements. An equivalent viscous damping value is recommended, based on judgement, experimental work etc. The values used in the design are conservative.

Some basic forms of damping such as material damping aerodynamic damping and acoustic damping are reasonably well understood. However, assessment of structural damping is difficult may be due to the complex nature of energy dissipation. Damping depends on many paramters such as the structural system, mode of vibration, strain, normal force, velocity, material joint slippage etc.

Soil properties (i.e. stiffness and damping) are known to be strain dependent. Since the dynamic forces developed in a structural system, are determined to the stress level attained, the damping factors recommended for design have a bearing on the allowable stresses.

For analysing multidegree systems by modal methods, damping value is required in each mode. It is customary (and conservative) to assume the same value of damping factors in all higher modes as in the fundamental. This approach is recommended for use in the dynamic analysis of structues.

The recommended values for damping are given in Annexure II.

Components of Earthquake:

During earthquake, ground moves in all directions and so far it is measured by Strong Motion Accelerograph which records two horizontal (in mutually perpendicular directions) and one vertical component of earthquake. Torsional component of motion has been postulated recently by theoretical methods but has not been measured and is not considered in the current design practice.

As structures are essentially three dimensional systems they would be excited simultaneously by three components of ground motion. It is the practice so far to consider only two components (one horizontal component along with the vertical component) at a time. This may be acceptable for most of the cases for nuclear structures. However, a designer can opt for three dimensional analysis with simultaneous input of earthquake motion along three perpendicular directions.

Superposition of Modes:

In dynamic analysis, the system is assumed to behave linearly and the method of mode superposition could be used with advantage. The minimum number of modes to be considered is first three modes, though quite often a much larger number is decided by experience in the analysis of similar systems. As the contribution to total response generally decreases with increasing order of mode,the maximum number of modes to be considered may be decided on a case by case basis for civil engineering systems.

It is relatively simple to obtain maximum response in each mode for the various components of earthquake. Since the maximum values of response in each mode do not occur simultaneously, the total response is usually approximated either as (i) absolute sum (ABS) (ii) square root of the sum of squares of response (SRSS) in each individual mode (iii) a suitable weighted combination of ABS and SRSS response. However, given the time history of motion, it is always possible to obtain the time-wise response history and then the maximum values at various locations.

Several codes recommend SRSS criteria of combination of modes and as an alternative suggest percentage intensity when more than one component of ground motion is considered simultaneously. SRSS method may be suitable to compute stresses and deflections when structures are modelled as cantilever sticks. For other cases, SRSS can be applied for combination of each particular modal response like load, shear, moment etc. Further, SRSS criteria would fail if applied to combination of stresses in each mode as obtained, say, in a 2D or 3D model as squaring the individual stress components would indicate that at all locations it would be tensile or compressive depending on the notation adopted. Further,the deformed shape of a finite element from its original shape cannot be predicted by SRSS.

As a convenient design procedure, it is suggested that in all such cases, response be obtained at certain finite times from the beginning of earthquake. since the phase difference at those finite times can be preserved in all modes and for each componental direction. It is proposed that these finite times correspond to the maximum spectral response (both positive and negative maximum) in each individual mode of vibration for the various components. For example, if one considers four modes of vibration and two components of earthquake there will be sixteen finite times to consider.

The total response could then be assumed to be the maxima of the values obtained at these finite times.

FEM is the most common method of analysis being adopted for the structural response. If the objective is to evaluate the stresses and displacements in various structural members finite element model can be used. In case of computational limitations, analysis of substructures as decoupled subsystem can be adopted for getting the stress etc. for the local area and ultimately combining the models to develop overall structural model using proper boundary conditions.

In case of severe seismic environment, there is a possibility of partial lift off at the foundation soil interface using conventional method. This may call for a more rigorous analysis to account for the non-linear behaviour due to base uplift.

Displacement Consideration:

Shake space between adjacent structural parts or between adjacent buildings should be assessed and provided for.

Settlement Considerations:

Settlements of foundations arise due to both static and dynamic loads. The total settlement consists of (1) uniform settlement and (2) differential settlement.

Uniform settlements are normally not a cause for concern except where different foundations are interconnected.

Differential settlements resulting within a single foundation influence the design of the foundation and the acceptable total settlement is governed by the system housed in the structure as well as the structure itself.

Subsurface material parameters or properties are determined through extensive geotechnical investigation. Under the design loading conditions, based on these technical parameters, the anticipated settlement is computed. The actual settlement of the foundation structure is monitored during construction.

The differential settlement as applied to rigid raft type foundation is of significance in terms of the total tilting of the structure. The maximum allowable tilt varies between 1/500 to 1/1000.

With respect to subsurface materials (soil and rocks) the main task is to verify and guarantee their stability under static and dynamic loading conditions with adequate margins of safety. A unique value for the margin of safety cannot be defined, but has to be established on a case by case basis. This value depends on the soil conditions, the method of analysis and model assumptions.

The methodolgy being followed is as follows:

The stability is verified and evaluated using standard practice and state-of-the-art methods that require numerical models to represent the physical behaviour of the subsurface materials.

The following aspects are considered in the analysis of settlements:

Foundations of buildings, interconnections :
The design of the interconnection between adjacent buildings, and the machinery operability allowances require that preliminary conservative assessment of differential and total settlement be performed. This is done using classical methods and other design parameters.

The level and extent of instrumentation for settlement determinations and ground water observations are appropriately related to the soil characteristics and predicted behaviour.

The numerical models used to predict settlements are to be corrected by comparisions of the predictions with actual behaviour.

It is advisable to delay construction of connections between buildings or structures subjected to differential settlement until predicted settlement behaviour is verified by the instrument installations and measurements.

Assessment of Settlements:
Time-dependent settlements occurring during the operational life of the plant are of concern and their effects on the overall safety are to be considered.

Time-dependent settlements are computed by the classical theory of consolidation and other sophisticated non-linear analyses.

The parameters for material modelling viz. the coefficients of consolidation, initial modulus, tangent modulus, Poisson's ratio etc. which define the constitutive law, are collected and evaluated for the entire profile of interest, by conducting laboratory and in situ tests.

The state-of-the-art relating to the settlements under seismic loading conditions has not so far progressed far enough to allow their reliable prediction under dynamic conditions. This is mainly due to difficulties in modelling the stress-strain behaviour of the subsurface material with the available constitutive laws. Hence, considerable judgement is necessary in estimating settlements of structures due to seismic load.

Buried Structures:

Earthquake effect on long buried structures e.g. buried pipes, ducts etc. should be taken into account by considering deformations imposed by surrounding soil during the earthquake and differential displacement or loads at end connections to buildings or other structures.

Dynamic Earth pressure:

For underground structures , the dynamic earth pressure effect should also be considered. This can be taken as incremental pressure over and above the static lateral pressure.

Fluid Interaction:

Where necessary, the dynamic effect of fluid in terms of added mass and sloshing effects should be considered in the analysis.

3109 Retrofitting:

The safety evaluation of NPPs extend into the operational phase through the performance of inservice inspection and testing of the structures, systems and components. Leaktightness tests on the containment are performed periodically and the results reviewed to ensure their adequacy to meet the safety criteria set in the design. The As Low As Reasonably Achievable (ALARA) target of the safety criteria ensures the implementation of future developments on an existing NPP during its operational life. Being a relatively new field of high technology there is significant improvement taking place with time which are required to be reviewd for incorporation in the existing NPPs. The feed back of operating experience, the postulation of new accident scenarios, the refinement in the calculational methods for safety analyses and the improved understanding of material behaviour in a radiation environment have resulted in increased demands being made on existing designs. Retrofitting thus involves two steps. The assessment of the revised design loads and the examination of the adequacy of the existing facility to meet the situation without compromising safety. The adoption of probabilistic methods is a direct consequence of this need. Where the situation demands strengthening an existing structure, this has to be carried out to enable the NPP to operate at its rated capacity. And if such measures are not feasible, then the operating level is lowered by the Regulatory authorities. Thus the mere fact that a plant is of vintage design cannot dilute the safety concerns relating to it. For the implementation of structural changes in the existing structures, a thorough examination of the interaction of the strengthening measures on the loaded structure involving both primary and secondary stresses is necessary. From the point of view of strengthening steel structures where the steelwork is exposed, offer greater flexibility than concrete structures. Even in steel structures, local strengthening caters to increased flexural tensile stresses and increasing the capacity of connections. In cases where compressive stresses are involved, the strengthening is extremely difficult if not impossible. Also, as between the structure and the foundations, strengthening of the latter involves concerns and innovative techniques bordering on applied research. Thus retrofitting has been the result of a growing need being met by improvement in the methods of design and construction.

3200 Site Related Activities

3201 Monitoring of Geotechnical Parameters

The subsurface exploration, in situ testing and laboratory testing should provide parameters and site characteristics suitable for predicting the performance of foundation systems under various loading conditions. The use of these parameters allows foundation design criteria to be established for the performance of the foundation materials and structures under anticipated loading. The preferred method of verifying foundation performance is to monitor the actual field behaviour over the entire period starting from the beginning of siting activities during and after construction.

In order to monitor possible instability following deep excavation in rock, it is necessary to record micro acoustic emissions associated with incipient sliding at planes of weakness. These micro acoustic emissions are extremely localised as they are associated with very high frequency say 500 Hz or so. To make effective use of this technique, it is essential that planes of weakness in excavated rock portion may be identified, and micro acoustic pick ups may be installed near anticipated planes of weakness where movements are expected. Possible sliding might be predicted from these micro acoustic emissions if observed. Installation of micro acoustic pickup, their observation and interpretation should be handled by specialists so that effective use could be made of this technique. Depending on the geological situation, it may be necessary to make these observations for extended periods so as to ensure safety.

3202 Monitoring of Seismic activity.

Seismic instrumentation is provided for recording the response of certain safety related items of NPP to an earthquake and evaluating the results of post-earthquake inspection. Such instrumentation also provides valuable data on the behaviour of structures, systems and components of the nuclear power plant during earthquakes and enables the verification of the adequacy of the design. In regions of high seismicity, provision of strong motion sensors, enables the triggering of alarms during strong earthquakes for safely shutting down the reactor.

Instrumentation to be provided at site

1. At least one triaxial strong motion recorder installed to register the free field motion time history.

2. Triaxial strong motion recorders installed so as to record

   (a) the motion of the base mat of the reactor building,

   (b) the motion of one, other category I structure (Designed for SSE)

   (c) the motion of the base mat of other important structures of the same category.

The aim is to achieve an understanding of the response to an earthquake of all the different types of structures important to safety with different foundations.

3. Triaxial strong motion recorders installed on the most representative floors of some category I buildings to obtain the response of different types of buildings important to safety.

These instruments should be set to be triggered at levels of motion consistent with the seismicity of the area. Consideration should be given to having a common trigger system and a common time scanning device.

3203 Ground Water Monitoring

Monitoring Objectives

There are two main objectives of the monitoring programme : First, to determine whether there have been any changes in the hydrogeological characteristics in the vicinity of a plant that could affect the outcome of the site evaluation; and second, to detect released radioactivity. The need for monitoring and the nature and extent of such a programme, will depend on the hydrogeological conditions of the site and on the use of the water extracted from near the site. It is usually possible to integrate the measurements with those planned for other purposes and to use the same set of samples. In some cases it may be necessary to review and modify the monitoring programme in order to take into account any changes that have occurred in the hydrogeological system since the programme was first developed, or any additional information that has been obtained about either the hydrogeological system or water use.

Monitoring methods:

The basic method used in this programme is to monitor groundwater through boreholes and wells; it may, however, sometimes be possible to monitor the groundwater that reaches the surface through springs or natural depressions. If boreholes are drilled or wells constructed for the purpose of monitoring groundwater in the region, they should be designed to last for the lifetime of the plant.

Methods and frequencies of measurement

Groundwater	Methods	Frequency*
Water table level	Piezometric analysis	Monthly
Chemistry	Chemical analysis	Monthly
Direction of flow	Tracer tests/ calculations from the hydraulic gradient	One initial measurement, and then after any significant changes in water table levels
Velocity	Tracer tests/ calculations from the hydraulic gradient	One initial measurement, and then after any significant changes in water table levels.

* The sampling frequency will depend on site characteristics; for example, if short-term fluctuations are of large magnitude, more frequent measurements should be made. In some cases, continuous recording may be necessary.

3204 Meteorological Observation

A micrometeorological station is normally set up at site for collecting meteorological data for use in the environmental hazard evaluation. The observation and data collection on temperature, humidity and rainfall, dry and wet bulb temperatures, wind data etc are made and monitored. This facility can be utilised for the collection of meteorological data at the site.

3205 Mockups

For ensuring the proper construction of certain specific engineered details of Nuclear Power Plants, mockup trials are carried out of selected parts of structures, preferably to full scale models and the results analysed for adoption on the prototype.

3206 Site Survey Instruments

Survey instruments like theodolites, levels, optical plummets, distomats etc. of high precision should be used for layouts and checking the levels and alignments during construction. This would ensure the desired dimensional accuracies and would also avoid secondary stresses in the structural members due to unacceptable deviations from specified tolerances.

Annexure - I

FIRE PROTECTION REQUIREMENTS OF R.C.MEMBERS

Element	Parameter	Minimum Dimension (mm) for Resistance Period			
		1 hour	2 hour	3 hour	4 hour
Column	Side	200	300	400	450
	Cover	25	35	35	35
Load bearing wall	Thickness	125	150	200	250
	Cover	15	25	25	25
Slab	Thickness	100	125	150	175
	Cover	15	25	35	45
Beam	Width	125	200	250	275
	Cover	30	60	70	80

Note: Covers indicated relate to main reinforcment bars.

Annexure - II

DAMPING VALUES FOR SAFETY RELATED STRUCTURES

(Percent of Critical Damping)

Sl. No.	Structure of component	Operating Basis Earthquake	Safe Shut-down Earthquake
1.	Welded Steel Structures	2	4
2.	Rivetted/Bolted steel structures	4	7
3.	Prestressed concrete		
	(i) Containment	3	5
	(ii) Other Structures	5	7
4.	Reinforced Concrete		
	(i) Containment	4	7
	(ii) Other structures	7	10
5.	Analytical Model (complete structure) with soil interaction on firm soil		
	1. Soil in rocking	7	10
	2. Horizontal deformation	20	25
	3. Vertical deformation	30	35
6.	Analytical Model (complete structure) without soil interaction	5	7

Modal damping must be evaluated using soil-structure interaction model by wieghted energy technique.

LOAD COMBINATIONS, ALLOWABLE STRESSES LOAD FACTORS UNDER SEISMIC LOADING

Sl. No.	Structure	Loading Criteria	Allowable Stresses (Working stress design)					Remarks	Partial Safety Factor for Loads (Limit State Design)*						
					SSE (Class I)	OBE (Class II)	Codal (Class III)								
1.	Prestressed Concrete	D + 1/2L + To + E + F	Concrete :	Compression	1.5 fc	N.A	N.A	1. Structures to be designed		SSE		OBE			
	Primary Containments	D + 1/2L + Ta + Pa + E + F		Shear	1.5 fq			for uncracked section.							
			Direct Tension		2/3 f (cr)			F ≯ f x d	DL	1.0	1.0	1.0	1.0		
								cr	LL	1.0	1.0	1.3	1.0		
				Combined					To	1.0	-	1.0	-		
				Bending +				2. Stresses around openings should	E	1.0	1.0	1.5	1.25		
				Direct				not be increased beyond permis-	Ta	-	1.0	-	1.0		
				Tension	0.8 f (Cr)			sible normal design stress.	Pa	-	1.0	-	1.25		
				{avg. tension ≯	2/3 f(cr)}				F	1.0	1.0	1.0	1.0		
			Reinf. steel :	Tension	0.6 fy			3. Use min. 0.15% reinforcement for							
								crack control on each face and							
								in each direction							
			H.T. Steel	Tension	0.8 f(ult)										
					or										
					f(pr)										
					Whichever smaller										
2.	Reinforced concrete	D + 1/2L + T + E/W	Concrete :	Compression	1.5 fc	N.A	N.A.	1. Calculate reinforcement steel		SSE		OBE		WIND	
	Secondary containment	D + 1/2L + T + Pd + E/W		Shear	1.5 fq			on cracked basis using steel							
				Tension	2/3 f(cr)			stress 0.6 fy.	DL	1.0	1.0	1.0	1.0	1.0	1.0
									LL	1.0	1.0	1.3	1.0	1.3	1.0
				Bending +				2. Stresses around opening should	To	1.0	-	1.0	-	1.0	-
				Direct				not be increased beyond permi-	E/W	1.0	1.0	1.5	1.25	1.5	1.25
				Tension	0.8 f(cr)			ssible normal design stress.	Ta	-	1.0	-	1.0	-	1.0
				{avg. tension ≯	2/3 f(cr)}				Pd	-	1.0	-	1.25	-	1.25
			Reinf. steel :	Tension	0.6 fy										

Sl. No.	Structure	Loading Criteria	Allowable Stresses (Working stress design)		SSE (Class I)	OBE (Class II)	Codal (Class III)	Remarks	Partial Safety Factor for Loads (Limit State Design) *
3.	Prestressed concrete structures other than containments	D + 1/2L + To + E/W + F D + 1/2L + Ta + Pa + E/W + F	Concrete : Reinf. steel : H.T. Steel	Compression Shear Tension Combined bending + Direct Tension	1.5 fc 1.5 fq f(cr) f(cr) {avg. tension} 0.9 fy 0.8 f(ult) or f(pr)	1.33 fc 1.33 fq 2/3 f(cr) 0.8 f(cr) 2/3 (fcr)} 1.33 fs 0.7 f(ult) or 0.9 f(pr)	As provided in IS : 456 IS : 1343	1. The minimum prestressing force on the member $\ngtr f \times A_{cr}$	SSE / OBE / WIND DL 1.0 1.0 | 1.0 1.0 | 1.0 1.0 LL 1.0 1.0 | 1.3 1.0 | 1.3 1.0 To 1.0 - | 1.0 - | 1.0 - E/W 1.0 1.0 | 1.5 1.25 | 1.5 1.25 Ta - 1.0 | - 1.0 | - 1.0 Pa - 1.0 | - 1.25 | - 1.25 F 1.0 1.0 | 1.0 1.0 | 1.0 1.0
4.	Reinf. concrete structures other than containments and water retaining structures	D + 1/2L + To + E D + 1/2L + Ta + Pa + E	Concrete : Reinf. steel	Compression Shear Tension Combined bending + Direct Tension	1.5 fc 1.5 fq f(cr) f(cr) {avg. tension} 0.9 fy	1.33 fc 1.33 fq 2/3 f(cr) 0.8 f(cr) 2/3 f(cr)} 1.33 fs	As provided in IS : 456	Suitable details for ductility as per IS 4326	SSE / OBE / CODAL DL 1.5 1.5 1.2 1.2 | 1.5 1.5 1.2 1.2 As LL 0.0 0.0 1.2 1.2 | 0.0 0.0 1.2 1.2 Per To 1.5 - 1.2 - | 1.5 - 1.2 - IS: E 1.0 1.0 1.0 1.0 | 1.2 1.2 1.2 1.2 456 Ta - 1.0 - 1.0 | - 1.2 - 1.2 Pa - 1.0 - 1.0 | - 1.2 - 1.2
5.	Reinforced concrete structures - water retaining structures	D + 1/2L + To + E D + 1/2L + Ta + Pa + E	Concrete : Reinf. steel	All stresses 1.5 x allowable stresses (as per IS 456 & 3370 as applicable) All stresses 1.5 x all stresses as per IS 3370 (Part II)	Concrete : Reinf. steel	All stresses 1.33 x allowable stress (as per IS 456 & 3370 as applicable) All stresses 1.33 x all stresses as per IS 3370 (Part II)	As per IS 456 & IS 3370 (part II) as applicable		SSE OBE CODAL As in 4 above. Check for serviceability criteria for limiting crack width to 0.1 mm under normal condition and 0.2 mm with SSE/OBE

Sl. No.	Structure	Loading Criteria	Allowable Stresses (Working stress design)				Remarks	Partial Safety Factor for Loads (Limit State Design) *
				SSE (Class I)	OBE (Class II)	Codal (Class III)		
6	Structural steel	D + 1/2L + To + E D + 1/2L + E + Pa + Ta	Axial or bending tension	fy		AS provided in IS : 800	Suitable details for ductility to be adopted as per IS Handbook SP 16 (6) - 1972	Plastic design for steel structures not adopted.
			Bending Compression	fy	1.33 fn			
			Axial or bending Compression (determined by buckling)	1.5 fn				
			Shear & bearing	1.5 fn				
			Connections	1.5 fn	1.25 fn			

N.B The containments structure will be designed for following additional loads :

1) Effect of Hydrodynamic loads is due to steam condensation in suppression pool.
2) Hydrostatic forces both inside and outside containment.
3) Erection loads, and loads due to vibration etc. will be considered appropriately.

In case of limit State Design, design strength of steel and concrete shall be as per IS : 456 - 1978

Abbrevations Used

D - Dead load inclusive of appropriate equipment load & hydraulic loads
L - Live load
To - Operating Temperature
T. - Ambient temperature
Ta - Accident temperature
Pa - Peak accident pressure
Pd - Differential pressure between atmosphere and annular space
Eo - Operating basis Earthquake
Es - Safe shutdown eqrthquake
E - Earthquake load SSE, OBE or Codal as the case may be (as per seismic classification of structures)
F - Prestressing force
d - Section thickness
A - member cross sectional area
W - Design Wind

fc - permissible stress in compression for concrete
fq - permissible stress in shear for concrete
ft - permissible stress in Direct Tension for concrete
fs - permissible tensile stress in steel
fn - permissible normal stress
fy - Yield stress in steel
f (cr) - Modular of Rupture of concrete
f (pr) - 0.2% proof stress for H.T. steel
f (ult) - ultimate tensile stress for H.T. steel

Chapter-4

QUALITY ASSURANCE.

4001 General:

This chapter covers all aspects of construction to ensure strict compliance of technical specifications as regards materials and workmanship for the implementation of the design intent as reflected in the drawings and specifications. The quality control aspects on the work is the link between design and quality assurance. It deals with materials and workmanship. Those aspects regarding the acceptance criteria relating to area important to safety and such work in areas not covered by any codes, standards etc.are covered hereunder:

Quality assurance comprises all those planned and systematic actions necessary to provide adequate confidence that all the facility buildings are designed and constructed to specified quality.

4002 A quality assurance programme of any design activity ensures the adequacy of the design in all interface areas of the multidisciplinary activity. It determines the design interfaces between the constituent agencies. It also identifies specific responsibilities of each group in the organisation.

Established procedures to ensure that the design inputs or any subsequent changes,during construction are correctly identified, documented and approved, are necessary. These procedures ensure that the inputs are specified promptly and contain the level of detail necessary for the activity to be carried out correctly and on a consistant basis for making decisions. Periodic meetings held between all concerned agencies streamline the information on design inputs.

The design would be carried out as per the specified practice and as per the codes and specific guidelines identified in the design inputs.

Design basis reports including the General Arrangement(GA) drawings and loading diagrams are to be prepared and reviewed before their adoption in the design. The analysis has to be performed and documented to enable their proper review at any future date by an independent qualified agency, if necessary.

Drawing office manual/procedures must be followed in the preparation, revision and issue of drawings. Standard drafting practice should be employed. Preliminary drawings shall invariably be reviewed and commented by system designers regarding layouts, loadings and embedments and other related areas before finalisation of design.

Design changes and design concessions must be documented.Site construction activities shall be planned and documented. Level two and level three planning networks should be prepared, describing various activities involved and the period allocated to execute them in accordance with the drawings and specifications in a planned manner within the specified time period.

4003 The material procurement activity must also be coordinated to ensure that the construction material specified is available on time to meet the construction programme.

Construction materials arriving at the construction site should be inspected to ensure that there is no damage due to:

1) Improper handling
2) Tie-down failure
3) Transportation
4) Environmental factors
5) Fire
6) Any other causes.

4004 Storage shall be provided as specified to segregate and protect material, prior to their use. Procedures for methods and conditions of storage to prevent the lowering of quality, due to such effects as corrosion, contamination, deterioration and physical damage, must be prescribed.

Procedures to control storage areas must be established and implemented. These procedures must prescribe the following as appropriate:

1) Access to storage area
2) Cleanliness and housekeeping practices
3) Fire protection

4) Identification and marking
5) Protective cover and seals
6) Coatings and preservatives
7) Prevention of physical damage
8) Removal from storage
9) Environmental control (such as temperature and humidity)
10) Preventive maintenance.

4005 Authority and responsibility:

(a) The authority and responsibility of persons and organisations performing activities affecting quality must be clearly established. Persons and organisations performing quality assurance functions must have sufficient and well-defined responsibility, authority etc. to:

1) identify quality programmes;
2) initiate, recommend or provide action which results in solutions through designated channels;
3) verify implementation of solutions;
4) control further processing of the technical aspects after any design concession until proper disposition has been made.

b) The persons responsible for defining and measuring the overall effectiveness of the Quality Assurance Programme shall be designated, and need to be sufficiently independent from the pressures of physical progress and shall have direct access to reponsible management at a level where appropriate action can be enforced. He shall report regularly on the effectiveness of the programme. In general, assurance of quality requires management measures which provide that the individual or the group, assigned the responsibility of checking, auditing, or otherwise verifying that an activity has been correctly performed, is independent of the individual or group directly responsible for performing the specific activity.

c) The programme must provide for the regular review by management or organisations participating in the programme of the status and adequacy of that part of the Quality Assurance Programme for which they have designated responsibility.

The programme must be based on consideration of the technical and construction aspects. Checklists shall be prepared and the observation during inspection recorded thereon.

During the process of construction, if any special equipment is employed such as equipment used say for excavation, backfill operation, compaction, density control etc. for manufacturing and placement and vibration and curing of concrete,etc. or any special equipments for erection, checking of alignments etc. the same should be highlighted in the works specification.

Quality control methods for concrete construction and steel construction should be spelt out in the form of detailed instructions for carrying out:

i) Preconstruction verification

ii) In-process inspection and testing

iii) Post construction/erection inspection

4006 Scale model testing and mock-ups should be planned for difficult area works to identify the appropriate construction technique.

The planning and procurement of construction material management for initiating procurement action, verification of bids with respect to specification,inspection and acceptance of material including transporting it to project site may be documented in details.

A programme for inspection of activities affecting quality shall be established in writing and executed by or for the organisation performing the activity to verify conformity with the specifications. Such inspection shall be performed by individuals other than those who performed the activity being inspected. Such individuals shall not report directly to the immediate supervisors who are responsible for the activity being inspected.

4007 Inspection procedures:

All inspection shall be performed in accordance with procedures which incorporate the requirements and acceptance limits specified.

All records required for quality assurance shall be maintained.

(a) Records shall be suitably protected from loss, deterioration and damage.

(b) Records shall be retrievable. Records shall be indexed and cross-referenced where necessary.

4008 Concrete used in nuclear applications has to meet biological sheilding functions in addition to the structural requirement. The control of density of concrete used in the construction of shield concrete is very important from the safety angle. Moreover, engineering of joints and joint configurations to meet the radiation shielding requirements is also very important from safety considerations. The placement of concrete and the method of construction of concrete is also very important as the quality of concrete and workmanship should be of a high order for structures like reactor containment. Batching plants and other construction plant shall be used in the mixing, transportation, placement and vibration of structural and shield concretes. Cold joints should be minimized by the institution of proper concrete placing sequences as far as possible. Techniques like slipforming may also be adopted. Adequate inspection and control should be exercised during selection of materials for making concrete. Only Ordinary Portland Cement (OPC) conforming to the latest Indian Standards should be used on important structures of the plant buildings. Control should be exercised on the testing of aggregates and quality of water or ice used in the production of concrete. Concrete admixtures should be used only after checking their performance by prior laboratory and field trials on concrete and the designed concrete mixes should be standardised for adoption on construction. Proper curing and approved methods for repair of defective concrete should be adopted to ensure high quality of work.

The concrete pours should be well engineered and documented to form the `pour' records. Adequate number of samples should be taken and tested in order to ensure proper quality of concrete being placed in important structures like Reactor containment building, foundations and other safety related structures.

Chapter.5

ACCEPTANCE CRITERIA

5010 Acceptance of Containment Structure:

The acceptance of the containment structure is based on a proof test and an integrated leakage rate test.

1. The purpose of the proof test is to demonstrate the integrity of the containment structure at a uniform internal pressure of 115% of the design pressure with the deflections and strains in the structure exhibiting a linear elastic behaviour within their predicted values.

2. The integrated leakage rate test is to demonstrate the leaktightness integrity of the containment to meet the requirement of leak rate less than or equal to the specified leak rate at the design pressure. This test shall follow the proof test.

5011 Containment test requirements:

Primary Containment:

Proof test pressure: The proof test pressure for the primary containment shall be one hundred and fifteen percent of the greater of the following:

a) The design basis leak test pressure.

b) Calculated design pressure from postulated double ended rupture of the largest diameter piping in the secondary steam or feed water system or any other accident causing internal overpressure with insignificant or no radiological release.

Design basis leak test pressure:

Design basis leak test pressure, shall be equal to or greater than the calculated peak pressure in the containment following the postulated design basis accident for the containment.

Permissible leakage rate:

The permissible leak rate would be smaller of the following:

a) The leakage rate (typically 0.3% of free internal volume per hour) acceptable on the basis of radiological consequences following the postulated design basis accident.

b) Best achievable leakage rate with the state-of-the art technology at the time of design and/or construction of containment.

Secondary Containment:

Design basis leak test pressure:

i) The maximum calculated peak internal overpressure assuming no purge from secondary containment and inleakage from primary containment during the postulated design basis accident.

Permissible leakage rate : The leak rate would be the smaller of the following:

a) The leakage rate acceptable on the basis of radiological consequences following the postulated design basis accident in the primary containment.

b) Best achievable leakage rate with the state-of-the art technology at the time of design and/or construction of the containment.

ii) A vacuum of at least 12 mm of water column selected on the basis of clear indication of vacuum under various conditions and also the measurability.

Note: Proof testing of secondary containment is not considered on the following grounds:

a) The secondary containment is intended to be a leak interceptor.

b) Overpressurization following a steam line/feed water line break is relieved to the atmposphere and it does not involve release of radioactivity beyond allowable limits.

5012 Checks and observations during test.

The containment proof test instrumentation dummy runs should be over. The stability of the instruments should be confirmed from dummy runs. The drift should be calculated from the daily readings about the same time of the day/night under stable atmospheric conditions (i.e. same atmospheric temperature, humidity etc.).

The structure should be inspected visually and the cracks if any, noticed on the exposed concrete surface, deformation if any noticed in the structure and, other relevant data that may be of use in interpreting the behaviour of the containment recorded. The pattern of any cracks exceeding 0.25mm in width and 150 mm in length shall be mapped.

It should be ensured that predicted values of stress, strain and deflection are available at site.

The strain and deflection readings at any of the measurement location should not exceed the predicted values by more than 30% with global deformation staying within the predicted value.

The deformation recovery in prestressed concrete works, in areas prestressed in two directions should be better than 80% at the end of 24 hours and for RCC structure better than 70% in 24 hours. After the containment is fully depressurized, entry may be made into the containment to assess the damage, if any, to the paint on concrete walls and floors, construction joints etc.

5013 Acceptance of containment.

The containment structural integrity may be considered proven if no yielding in reinforcement develops as determined from deflection and strain readings.

No visible permanent damage to either the concrete surface or in the junctions with appurtenances are detected.

The deflection recovery at the points of maximum expected deflection withn 24 hours after complete depressurization is 80% or more in the prestressed concrete structures and 70% or more in the conventionally reinforced concrete structures, taking into account the inaccuracies of such measurements.

The measured maximum deflections at points of maximum predicted deflections do not exceed predicted values by more than 30%. Whereas during the test if the deflection exceeds the predicted value by the above limit, the containment behaviour may be considered within the limits if following the test, 24 hours after depressiruzation the deflection recovery in those areas is more than 90% in prestressed concrete structure and more than 80% in RCC structures.

5020 Acceptance Criteria for Spent Fuel Storage Pool.

For acceptance, the leaktightness test shall be carried out; after initial grouting work, raft water proofing, and water proofing treatment on the external surface of walls etc. but prior to metal lining on the inside surface.

Following procedure shall be followed.

Case-1: Where the ground water table is quite high.

The leakage test for ingress of ground water will be conducted first by allowing the water table to rise slowly to a stable level by stopping dewatering system. All the leakage spots and wet surfaces on the inner face of the wall and raft surfaces will be marked accurately. The water table outside the pool will be lowered by starting dewatering system and corrective treatment such as various types of injection grouting, surface coatings, repairs to the concrete etc. shall be carried out. After satisfactory completion of the corrective measures, the water level outside the pool will be allowed to rise again. If any wet surfaces persist the procedure will be repeated until no wet patches appear on the inside of the outer wall.

Case - 2: Where the ground water table is not substantially high above raft.

The test will be conducted by filling the storage pool with the water. The filling will be done in stages to identify properly the leakage spots and leakage paths if any. All the wet patches and leak spots on external surfaces will be demarcated properly. After emptying the pool corrective treatments such as various types of injection grouting, surface coating, repairs to concrete etc. shall be compeletd. The pool will be filled again and external surfaces will be observed. In case wet patches appear, the above procedure will be repeated till such time the wet patches do not appear.

After satisfactory repairs, the pool will be filled with water. A period of seven days will be allowed for absorption of water and saturation of the concrete. The exposed water surface of pool may be protected by covering it with polythene sheets or suitable thin oil film to minimise evaporation loses. After initial fill of 7 days, the water level shall be recorded every 24 hours for a further period of 7 days. Water loss will be to the extent of losses accountable due to evaporation only since the process of absorption by concrete would be completed by then. A drop in water level at the rate of 6mm or 12 mm for 24 hours may be allowed as evaporation losses for covered or open pool condition as the case may be.

After successful completion of testing for acceptance as detailed above, the pool shall be emptied and water proof treatment will be carried out on the exterior faces of walls and metal lining will be provided on the inside surface. Backfilling around the storge bay will also be allowed only after satisfactory testing of the pool.

D) The test aims at achieving zero leakage giving the due allowance for the evaporation and absorption losses as specified.

Chapter 6

DECOMMISSIONING

6000 General:

Taking an industrial plant out of operation and shutting it down permanently is called 'Decommissioning'. In the past, factories and conventional power plants were simply demolished or converted to other use when they became uneconomic or were worn out. Generally no special care was taken to prevent hazardous material being spread into the environment once the plant had been torn down or abandoned. In nuclear plants the potential risk lies in the radioactivity of the materials to be handled. Because the risks of radiation have been understood from the outset, a comprehensive system of rules have been developed for avoiding or minimising the risks.

Procedures for decommisssioning nuclear installations have been developed over the last 20 years with a knowledge of risk and problems involved.

At the end of the 50 years life, a nuclear plant contains material, structures and worn out equipment some of which are radioactive and some are not.

6001 The decommissioning will be carried out in three stages as follows:

finely divided concrete fragments is best accomplished using vacuum collection system. Another concrete surface removing method is flame scarifying.

Where relatively thick (75 - 150 mm) layers of concrete must be removed, controlled blasting may be employed. Special care must be taken to control dust and to prevent damage from flying fragments through the use of water mist sprays and blasting mats. Removal of large segments of concrete can also be accomplished using such methods as rock splitting, flame or thermic lance cutting, diamond sawing, coring and high pressure water jet cutting.

Soil Decontamination:

Decontamination of contaminated soils has usually been accomplished by removing the affected surface layers of soil and conditioning the removed material for disposal. However, for situations where large areas of soil surface are involved, the costs can become very large. In one instance, polyurethane foam was placed on the contaminated surface, allowed to set and then removed as a solid, carrying with it about 85% of the radioactive materials originally present. Later development work has applied wet screening, and attrition scrubbing at high and low pH values to the removal of plutonium and americium from contaminated soil. As much as 99.9% of the original radioactive material was removed via the wash solution but recycling of the soil through the process,four or five times,was required to achieve that degree of decontamination.

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DEFINITIONS

1. **Accident Conditions**

   Substantial deviations from Operational States which are expected to be infrequent, and which could lead to release of unacceptable quantities of radioactive materials if the relevant engineered safety features did not function as per design intent.

2. **Aggradation**

   A rise in the level of a river channel and flood plain. It has various dynamic causes.

3. **Annual Mean High Spring Tide**

   The average value of all spring high tides in a year.

4. **Baserock**

   A well consolidated geological formation which can be considered as homogeneous with respect to seismic wave transmission and response.

5. **Bathystrophic Storm Tide Theory**

   A theory dealing with slow-moving large-scale storm systems, in which the storm surge responds instantaneously to the onshore wind stresses.

6. **Beach Berm**

   A nearly horizontal part of a beach formed by the deposit or erosion of material by wave action.

7. **Decommissioning**

   The process by which a Nuclear Power Plant is finally taken out of Operation.

8. **Deep Water**

   Water of a depth greater then L/2, where L is the wavelength of the surface wave under consideration.

9. Degradation

A lowering of the level of a river channel and flood plain. It has various dynamic causes.

10. Design Basis for External Events

The parameter values associated with, and characterizing, an external event or combinations of external events selected for design of all or any part of the Nuclear Power Plant.

11. Design Basis External Man-Induced Events:

External man-induced events slected for driving design bases.

12. Design Basis Flood(DBF)

The flood selected for deriving a design basis for a Nuclear Power Plant.

13. Design Basis Natural Events

Natural events selected for deriving design bases.

14. Design Floor Response Spectrum

Response spectrum defined at a particular building elevation which is obtained by modifying one or more floor response spectra in order to consider the variability and uncertainty of input ground motion and of the characteristics of both building and foundation.

15. Design Floor Time Histories

Time histories of floor motion of structure under consideration derived from the design basis ground motion, including the variability and uncertainty in input ground motion and in building and foundation characteristics.

16. Deterministic Method

A method for which most of the parameters and their values are mathematically definable and may be explained by physical relationships.

17. Documentation

Recorded or pictorial information describing, defining, specifying, reporting or certifying activities requirements, procedures or results related to Quality Assurance.

18. Epicentre

The geographical point on the surface of earth vertically above the focus of the earthquake.

19. Examination

An element of Inspection consisting of investigation of materials, components, supplies or services to determine conformance with those specified requirements which can be determined by such investigation.

20. Fault

A fracture or fracture zone along which displacement of the two sides relative to one another has occurred parallel to the fracture.

21. Fetch

The extent of sea water over which the wind under consideration blows, measured along the direction of the wind.

22. Floor Response Spectrum

For a given ground motion, the floor response spectrum at a particular level of a structure is the response spectrum of the motion at that level.

23. Geometrics

Geometrics are the dimensions of those highway features which include alignment, grades, widths, sight distances, curves, slopes etc.

24. <u>Hydrologically Homogeneous Region</u>

A region for which a hydrological model can be used to transfer hydrological data using the same parameter or parameters systematically varied as a function of definable space-variable characteristics of the region.

25. <u>Hypecentre</u>

The location in space where the slip responsible for an earthquake occurs; the focus of an earthquake.

26. <u>Inspection</u>

Quality Control actions which by means of examination, observation or measurement determine the conformance of materials, parts, components, systems, structures, as well as processes and procedures, with predetermined quality requirements.

27. <u>Intensity of Earthquake</u>

The intensity of an earthquake at a place is a measure of the effects of the earthquake, and is indicated by number according to the Modified Mercalli Scale of Seismic Intensities.

28. <u>Isoseismal</u>

An imaginary line joining points of equal earthquake intensity.

29. <u>Item</u>

A structure, system, sub-system, piece of equipment or component taken individually or collectively according to the context.

Items Important to Safety

The items which comprise:

(1) those structures, systems, and components whose malfunction or failure could lead to undue radiation exposure of the Site Personnel or members of the public.

(2) those structures, systems and components which prevent Anticipated Operational Occurrences from leading to Accident Conditions;

(3) those features which are provided to mitigate the consequences of malfunction of failure of structures, systems or components.

30. Karstic Phenomena

Formation of sinks or caverns in soluble rocks by the action of water.

31. Lens

A geologic deposit that is thick in the middle and converges toward the edges, resembling a convex lens.

32. Liquefaction

Sudden loss, due to vibratory ground motion, of shear strength and rigidity of saturated, cohensionless soils.

33. Magnitude of Earthquake (Richter's Magnitude)

The magnitude of an earthquake is the logarithm to the base 10 of the maximum trace amplitude, expressed in microns, with which the standard short period torsion seismometer (with a period of 0.8 second, magnification 2800 and damping nearly critical) would register the earthquake at an epicentral distance of 100 Km. The magnitude M is thus a number which is a measure of energy released in an earthquake.

34. Meteorologically Homogeneous Region

A region for which a meteorological model can be used to transfer meteorological data using the same parameters systematically varied as a function of definable space-variable characteristics of the region.

35. Neotectonics

For seismic regions, the tectonics of the Quaternary era.

36. Nuclear Power Plant

A thermal neutron reactor or reactors together with all structures, systems and components necessary for Safety and for the production of power, i.e. heat or electricity.

37. One-Percent Wave Height

The average height of the upper one percent of wave heights in a wave record.

38. Operating Basis Earthquake (S1)

The Operating Basis Earthquake is that earthquake which produces the vibratory ground motion for which those Nuclear Power Plant structures, systems, and components necessary for power generation are designed to remain operable.

39. Postulated Initiating Events(PIE)

Events that lead to Anticipated Operational Occurrences and Accident Conditions, their credible causal failure effects and their credible combinations.

40. Probable Maximum Extra-Tropical Storm(PMETS)

The hypothetical extra-tropical storm (often termed a "depression" or "low pressure area" which is generated in mid or high latitudes above about 25 deg N or 25 deg S) having the most severe combination of meteorological storm parameters, from the point of view of flooding, that is considered reasonably possible in the region involved, and which approaches the point under study along the critical path and at a rate of movement which will result in the most adverse flooding.

41. Probable Maximum Flood(PMF)

The hypothetical flood(characterized by peak flow, volume, and hydrograph shape) that is considered to be the most severe reasonably possible, on the basis of probable maximum precipitation and comprehensive hydrometeorological application of other hydrological factors favourable for maximum flood runoff such as sequential storms and snowmelt.

42. Probable Maximum Precipitation(PMP)

The estimated depth of precipition for a given duration, drainage area, and time of year, of which there is virtually no risk of exceedance. The probable maximum precipitation for a given duration and drainage area approaches and approximates to that maximum which is thought to be physically possible within the limits of contemporary hydrometeorological knowledge and techniques.

43. Probable Maximum Storm Surge(PMMS)

The hypothetical storm surge generated by either the PMTC, the PMETS, or the probable maximum squall line.

44. <u>Probable Maximum Tropical Cyclone</u> (PMTC)

The hypothetical tropical cyclone characterized as a rapidly revolving storm having that combination of characteristics which will make it the most severe, from the point of view of flooding, that can reasonably occur in the region involved, and which approaches the point under study along the critical path and at a rate of movement that will result in the most adverse flooding.

45. <u>Quality Assurance</u>

Planned and systematic actions necessary to provide adequate confidence that an item or facility will perform satisfactorily in service.

46. <u>Quality Control</u>

Quality Assurance actions which provide a means to control and measure the characteristics of an item, process or facility in accordance with established requirements.

47. <u>Quaternary Deposits</u>

The geologic formations belonging to the second period of the Cenozoic geologic era, following the Tertiary, and including the last 2-3 million years.

48. <u>Records</u>

Documents which furnish Objective Evidence of the quality of items and of activities affecting quality.

49. <u>Reference Water Level</u>

A conservatively estimated reference water level, either high or low(for flooding or minimum water level evaluation respectively), including, as appropriate, components such as the tide, river flow and surface runoff but not including water level increases resulting from surges, seiches, tsunamis and wind-waves(for flood evaluation) or drawdown(for minimum water level evaluation).

50. <u>Region</u>

A geographical area, surrounding and including the Site, sufficiently large to contain all the features related to a phenomenon or to the effects of a particular event.

51. Relevant Bodies of Water

All streams, rivers, artificial or natural lakes, ravines, marshes, drainage systems and sewer systems that may produce or affect flooding on or adjacent to the Nuclear Power Plant. Bodies of water located outside the watershed in which the plant is located, but which may, by overflowing the watershed divide, produce or affect flooding of the plant, are also considered relevant bodies of water.

52. Reliability

The probability that a device, system or facility will perform its intended function satisfactorily for a specified time under stated operating conditions.

53. Runup

The rush of water up a beach or structure on the breaking of a wave. The height of runup is the vertical height above still-water level tnat the rush of water reaches.

54. Safe Shutdown Earthquake (S2)

The Safe Shutdown Earthquake is that earthquake which produces the vibratory ground motion for which Nuclear Power Plant structures, systems, and components important to safety are designed to remain functional.These structures, systems and components are those necessary to assure:

(1) The integrity of the reactor coolant pressure boundary

(2) The capability to shut down the reactor and maintain in a safe shutdown condition, or

(3) The capability to prevent or mitigate the consequences of accidents which could result in potential off site nuclear exposures comparable to the guideline exposure of 10 CFR Part 100 of the United States of America (USA) Atomic Energy Commission.

55. Safety

Protection of all persons from undue radiological hazard.

56. Safety Report

A document provided by the Applicant or Licensee to the Regulatory Body containing information concerning the Nuclear Power Plant, its design, accident analysis and provisions to minimize the risk to the public and to the Site Personnel.

57. Screening Distance Value(SDV)

The distance value used for preliminary screening purposes, beyond which the potential sources of a particular type of external man-induced events can be ignored.

58. Screening Probability Level(SPL)

The probability value of occurrence per annum of a particular type of Interacting Event below which such an event can be ignored for preliminary screening purposes.

59. Sea Level Anomaly

An anomalous deparature of the water surface elevation from the predicted astronomical tide.

60. Seismically Active Structure

A structure or a fault which exhibits seismicity at a level which indicates significant coherent activity on the structure or fault, regardless of whether or not geologically young movement on it can be demonstrated at the earth's surface.

61. Seismotectonic Province

A geographic area characterized by similarity of geological structure and earthquake characteristics.

62. Shallow Water

Water of a depth less than L/25, where L is the wavelength of the surface wave under consideration.

63. Significant Wave Height

The average height of the upper third of the wave heights in a wave record.

64. Site

The area containing the plant, defined by a boundary and under effective control of the plant management.

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