APR 20 1988

Dr. P. Giuliani
Scientific Secretary
International Atomic Energy Agency
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Austria

Dear Dr. Giuliani

Thank you for sending me the draft document on "Probabilistic Safety Assessment for Seismic Events." Dr. N. Chokshi of Nuclear Regulatory Research and I performed a "quick" review of the subject draft report. Our comments on the overall report are attached.

If you have any questions or need any further clarification, please let me know. Also, I am attaching my views on the state-of-the-art of the use of Probabilistic Seismic Safety Assessment (PSSA) in the USA.

I hope you find these comments of some help and look forward to receiving the final report.

With best regards.

Sincerely Yours,

/S/

Abou-Bakr Ibrahim
Technical Review Branch
Division of High-Level Waste Management
Office of Nuclear Material Safety
and Safeguards

Attachments:
As stated

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NHPA

Comments

1. The document did not include a table of contents, this has made it hard for the reviewer to follow.
2. Additional materials are added to the document as Appendixes without marking which chapter they belong to?
3. Some of the chapter titles which we agreed to at the meeting in November 1987 to be changed has not been changed.
4. The document includes a lot of materials but these materials are not coherently put together.
5. Some of the materials in the document are copied from the attached Appendix e.g., section 2.5.2 on p.11 is a copy of p.17 of chapter 4 titled "Seismic Probability Safety Assessment Procedure." This section 2.5.2 should be eliminated if we are going to attach the appendix.
6. In chapter 3 "Seismic Hazard Analysis", the discussion on uncertainty of the seismic hazard and the sensitivity to the model parameters should be expanded.
7. The report is almost totally silent on one of the most important ingredient of a seismic PRA, the plant walk-down. The plant walk-down has been found to be very important for the identification of critical failure modes and components. Walk-downs also uncover potential interactions between seismic induced fires and floods. A number of plants have made modifications based on the plant walk-downs.
8. Similarly, spatial interaction are given very cursory treatment in this document. It may be instructive to include a chapter in this report which summarize lessons learned from the past seismic PRA. Importance of walk-downs and interaction will be clearly evident from such a summary.
9. A concept of 'event flow chart' is presented in this report. This concept begins with a gross failure of structure (e.g., loss of bearing pressure capacity of a foundation) and tracer through effects of this failure on various components (e.g., loss of content or slight leakage of a tank in the given structure). This is a very interesting and powerful concept and may be useful; however, this concept does not address the question. "In how many ways can I loose functions of a component important from the risk perspectives? In similar vein, the report in general, has failed to bring out the importance of interactions between fragility analyst and system analysts.
10. The level of details in various sections of the report is uneven. I think two much detail is included in chapter 5 and chapter 6 regarding structural mechanics and is repetitions, however, actual fragility methodologies are not clearly specified. Chapters 5 and 6 have lost something in translation.

Some additional comments are indicated on the attached pages.

In many cases these PSA's have been limited to internally initiated failures and important external initiators such as earthquakes have not been included in the analysis.

fire and [illegible]

It was anticipated that the design procedures adopted against seismic and other external events were such as to provide the plants with a degree of protection larger than that for the accidents of internal origin. Furthermore, the analysis was deemed beyond practical feasibility because of the large uncertainties associated with the various steps and of the lack of adequate methodologies for dealing with the quantitative treatment of the risk.

Both arguments have lost most of their strength today. Actually, a number of seismic PSA's conducted for existing plants have shown that earthquake-induced accident sequences may have frequencies of occurrence of the same order of magnitude of those caused by internal events. Also, in the last decade probabilistic methods of analysis have developed to a point where seismic PSA's of NPP's can be performed with acceptable effort and adequate degree of confidence.

1.2 Purpose of the document

The purpose of this document is to provide information and some measure of guidance to those who are considering starting a seismic PSA. It tries to give an overall picture of the seismic PSA and attempts to bridge the gap existing between an internal event PSA and a seismic PSA.

Aside from giving an order of magnitude of the frequency of core damage associated with earthquakes, which might contribute not negligibly to the total frequency of core damage, a seismic PSA involves a number of beneficial side effects. Basically, these benefits are all related to the abandoning of the deterministic, largely conventional and conservative logic, a present characteristic of ordinary design which has to go through a licensing process.

By considering, for example, different possible spectral shapes, and by determining their separate effects on structures, systems and components, and by finally weighing the latter with their expected frequencies of occurrence, one has more rational bases either for checking the adequacy of existing items or for the optimal design of new ones.

The PSA makes use of alternative hypotheses regarding design basis earthquake parameters, soil properties, structural models, material characteristics, etc. In short, this sort of weighted sensitivity analysis, is an instrument for detecting the weaker links in the chains leading to undesired events.

Finally, one should add that resorting to a PSA is an appropriate means for checking the adequacy of an old plant in case the progress made in the knowledge of the seismic environment involves significant modifications with respect to the design basis earthquake originally specified for the plant.

?

1.3 Scope of the Document

There is a variety of measures of risk which may be desired from a seismic PSA. The particular measures chosen depend upon the types of facilities being studied, the resources (both financial and computational) available to the analysts, and the intended use of the PSA results. For example, in seismic PSA for nuclear power plants, core damage frequency, radioactive release frequency, and total public exposure to radiation are three distinct and useful steps in the measuring of the risk.

Using the results of the accident sequence analysis to arrive at the final measure of risk is, generally, no different for external initiators PSA than it is in internal initiator PSA. However, there are some important considerations which one should be aware of, particularly if adapting the results of an internal initiator PSA. An internal initiator PSA may utilize certain assumed factors in translating facility damage into other estimates of risk. These factors may not be suitable for use in a seismic PSA of the same facility since they include assumptions which may not apply during or immediately following a seismic event.

In this document, a level 1 PRA will be considered. Therefore potential releases and offsite consequences are not included and are considered to be outside the scope of this document. The scope of this document includes mainly the frequency of occurrence of ground motion, the seismic accident sequence initiators, the fragility analysis of safety related items, the capability of systems to mitigate accidents from seismic events and the integration of these aspects which might lead to a core damage. The reason for this simplification is to limit this subject to the essential aspects of the smooth integration of seismic related initiating events into the systematic framework of a PSA.

This does not imply, however, that effects of earthquakes after the event of a core damage may be neglected. Some of the aspects which may have significant contribution to overall risk, but have not been considered in this document, are

- increased probability of human error subsequent to the occurrence of a destructive earthquake,

- increased probability of leakage from the containment structure,

- significant probability of damage to lifelines and other infrastructures which may have been planned for use in the context of emergency planning and evacuation.

- increased probability of delayed response to the nuclear accident (by authorities and the public) due to the interference of another catastrophic event.

This document will concentrate on the PSA studies for NPP's. Other facilities such as research reactors, fuel cycle facilities, gamma irradiation facilities, fuel storage facilities are simpler in design and operation compared to NPP. It will be possible to extrapolate the methodologies used for NPP's to these other facilities.

– uncertainty due to design/construction error ?

Though this step may be straightforward, it is by no means inconsequential. Some seismic PSAs have found that the dominant contributors to degraded core accident probabilities involve random failures of safety equipment to operate on demand. The failure is still considered to be seismic because the accident is initiated by an earthquake even though the subsequent equipment failures may be dominated by random failure modes.

2.3 Component Failure Quantification

Given the data described in Section 2.2, it is possible to calculate the probability of seismic failure of the various structures and equipment in a nuclear power plant. The most fundamental quantity which can be calculated is the marginal failure probability of an individual component. This probability is conditional on the occurrence of the earthquake level, and the marginal failure probability could be transformed to an unconditional probability by folding in the occurrence probability of the earthquake. A marginal probability can be helpful in isolating the contribution made by a component to the safety of the whole plant. This contribution may arise from either the relative intrinsic weakness of the component or from its location within the plant.

2.4 Accident Sequence and System Failure Quantification

The method for obtaining safety system and accident sequence cut set probabilities from the logic models is discussed in Section 7.2. These cut sets must be quantified to obtain the probabilities of safety system failures or accident sequence occurrences. These quantities are useful in showing the relative contribution of systems and sequences to core damage. The calculation of cut set probabilities presents one of the most fundamental differences between seismic and internal initiator PSA. In internal initiator PSA, component failures are usually treated as independent and random events. Consequently, the probability of a cut set involving independent random failures would be evaluated simply from combinations of the random failure probabilities of each of the elements of that cut set. In a seismic PSA, the component failures represented in a cut set may be correlated through their

External

respective responses and fragilities. This correlation is in fact one of the reasons why seismic initiators are of particular concern in nuclear power plants; earthquakes can cause the simultaneous failure of several redundant safety equipment items, and the correlation is a measure of the potential for this simultaneous failure. Calculation of the probabilities of cut sets containing correlated events involves multivariate integration of the joint probability density function of the cut set elements [NUREG/CR-3428, chap 6]. The process is considerably more complicated and costly than simple multiplication but is necessary to account for the increased cut set probabilities which result from correlated failures.

2.5 Uncertainty Analysis

2.5.1 Sources of Uncertainty

The estimate of the frequency of core damage produced from a seismic PSA has considerable uncertainty associated with it and, without a measure of the uncertainty, the point estimate itself is almost meaningless. Two distinct sources of uncertainty are recognized as making separate contributions to the overall uncertainty. These two sources are generally termed random and modelling.

2.5.1.1 Random Uncertainty

There is, of course, variation in every physical measurement and therefore even the empirical data used in a seismic PSA involve uncertainty. This variation may arise in part from the stochastic nature of underlying physical processes and in part from the inability to measure precisely the parameters which characterise those processes.

2.5.1.2 Modelling Uncertainty

The uncertainty which enters into the estimates of the frequency of core damage from a seismic PSA as the result of the availability of a number of methods for modelling each step of the overall procedure is referred to as modelling uncertainty. It is distinguished from random uncertainty because it originates with the methods used to model the seismic hazard and the plant response rather than as the result of inherent variability in the physical processes being modelled.

2.5.2 Distinguishing Between Sources of Uncertainty

Figure 2.3 provides an example of the distinction between random and modelling uncertainty. This figure shows 14 seismic hazard estimates which were developed by various seismicity experts for the same nuclear power plant site. The two types of uncertainty, random and modelling, can be clearly distinguished in this figure. The individual seismic hazard estimates represent the inherent variability in earthquake size and frequency at the site. If there were no inherent variability in earthquakes at the site, then it would be possible to state just how many earthquakes of a given intensity would occur in any one year. Since earthquakes cannot be predicted with anywhere near this level of certainty, their future occurrence can only be assessed with probabilistic statements exemplified by the individual seismic hazard functions shown in the figure.

The modelling uncertainty introduced as part of the process of characterizing the seismic hazard at the site is seen in the fact that there are 14 separate seismic hazard functions. Each of these functions represents a different opinion about the seismic hazard. Since each of the experts had access to the same body of physical data for the site and surrounding region, the variability between the various hazards is due more to differences among the experts' methods for interpreting the physical data than to variability in the physical data.

these paragraphs are copies of page 17 of the attached in the document

The distinction between random and modelling uncertainty is, at its root, an artificial one. Ultimately, all uncertainty stems from our lack of knowledge about physical processes. For example, if we had perfect knowledge about all of the factors affecting the seismicity of a site, we would be able to predict just exactly when and what type and size of earthquake would occur at that site. Unfortunately, we have a very imperfect understanding of the factors involved. We can, however, distinguish to some degree the uncertainty which is practically irreducible from that which is not. The reducible uncertainty is that caused by our imperfect interpretation of the limited data that we have. The 14 seismic hazard curves shown in Figure 2.3 constitute an expression of that imperfection.

2.5.3 Presentation of Results

It is convenient to treat the two recognized sources of uncertainty separately in the analysis of seismic risk. The contribution from randomness leads to a single point estimate of risk while that arising from modelling uncertainty introduces a dispersion around this point estimate.

To exemplify this, Figure 2.4 shows a distribution of risk estimates obtained from a seismic PSA with a full uncertainty analysis. Also indicated is the point estimate from that PSA. Clearly, the results from the uncertainty analysis paint a different picture of risk at the facility from that of the point estimate. The median of the distribution is seen to be an order of magnitude greater than the point estimate; while the 90th percentile estimate is 200 times greater than the point estimate.

We refer the reader to other sources for a full discussion of the

EPRI

merits of and methods for conducting an uncertainty analysis (reference 2.1). The point we wish to stress here is that seismic PSA, as with internal event PSA, is fraught with uncertainties, both in the data and in the methodology used to analyze those data. Consequently, an uncertainty analysis is essential to the meaningful interpretation of a seismic PSA.

3. SEISMIC HAZARD ANALYSIS

3.1 General

The seismic input to be utilised within the scope of the PSA is derived from probabilistic considerations which comprise stochastic and deterministic forecasts.

The aim is to construct a seismic hazard curve for the primary effects (e.g. vibratory ground motion for the freefield or bedrock) for the site. The hazard curve which characterises the seismic exposure of the site is derived from a seismotectonic model, frequency-magnitude relationships and attenuation relationships. The process is shown schematically in Fig. 3.1. and Appendix A and gives examples of seismic hazard analyses carried out in Japan and the USA respectively. Bernreuter et al. (1987)

The resulting hazard curve is used in assigning probabilities to the initiators of the various seismic accident sequences as described in Chapter 4.

In general three effects have to be considered - regional and local seismotectonics and dynamical properties of the travel path. These three together determine the occurrence of earthquakes and the transmission of ground motion to the site.

The considerable uncertainties associated with the aforementioned factors are quantified by a probabilistic seismic hazard analysis. The uncertainties of the seismic hazard curve itself will be described in Section 3.3.

3.2 Probabilistic Assessment of Vibratory Ground Motion

3.2.1 Seismotectonic Modelling

Seismotectonic modelling of the region which may generate earthquakes affecting the nuclear facility comprises a methodology which has evolved over the past few decades and which takes into account the increasing amount of seismological data and advances in understanding of tectonics.

Bernreuter, D.L., J.B. Savy and R.W. Mensing, Seismic Hazard Characterization of the Eastern United States: Comparative Evaluation of NRC and EPRI Studies, NUREG/CR-4885, 1987.

anisotropic features special to the site. They can be incorporated in the seismic hazard curve by adjusting the regional attenuation relationships. This is also the case for regional effects as the energy contribution of different types of seismic waves (P, S, Rayleigh, Love, etc) depends largely on the general travel path characteristics. This might be especially important in considering linear or underground civil structures.

3.2.5 Attenuation Model and Seismic Hazard Curve

An indispensable step in deriving the hazard curve is the model giving the vibratory ground motion parameter (see 3.2.3) as a function of the focal event. Part of this, is the phenomenon of attenuation.

Figure... shows schematically how the mean curve is obtained from several single event attenuation curves as taken from isoseismal maps. In the deterministic approach one uses the mean curve, perhaps with an added safety factor. No systematic treatment of the single event variability is attempted.

In the probabilistic approach this important variability can be accounted for in a number of ways. The following expression illustrates in principle how the seismic hazard curve $\lambda(>I)$ is determined.

$$\lambda(>I) = \sum \sum \lambda(\Delta V,\Delta M).P(M,R,I)$$

Expand

where

$\lambda(\Delta V,\Delta M)$ is the frequency of a focal event of magnitude M in the crustal volume element ΔV at hypocentral distance R and

$P(M,R,I)$ is the probability that the event produces a site intensity $> I$.

The probability $P(M,R,I)$ is determined from the single event curves (see fig....) and their probabilities. In practice the integrations may be performed using Monte Carlo or other suitable techniques (see e.g. Appendix A).

For example, if a seismic PSA is being performed for a nuclear facility which is in the flood plain of a dam, then it is almost certainly the case that a flood analysis for the facility is being performed as well. This flood analysis will consider non-seismically induced modes of flooding arising from such phenomena as random failure of the dam, dam failure due to severe weather, improper operation of the dam, etc. It would be entirely possible to increase the frequency of the gross failure of the dam by the amount due to seismic events and, in this way, account for the risk to the nuclear facility from seismically induced dam failure. In fact, this seems to be a much more desirable approach since the analysis of plant damage will be similar for seismically induced failure of the dam as for non-seismically induced modes of failure.

The word similar is used because there is one important distinction between seismically induced flooding at a nuclear facility site and other flood events at the site, and that is the fact that the facility will also very likely be subjected to some seismic loading as well as flood damage, depending on the epicentral location of the earthquake, the proximity of the dam to the facility, the surrounding geology, etc. This problem of having to consider two simultaneous or closely sequenced external events impacting a facility is a very difficult one. It has not been extensively addressed and may, in fact, defy adequate treatment. One may wish, instead, to calculate the probability of sustaining two severe external events simultaneously and assume some maximum damage state to result. The frequency of such damage state arising from this simultaneous occurrence would then be simply equal to the expected frequency of simultaneous occurrence of the two external events. This frequency may be well within whatever is deemded to be an acceptable level of risk for the facility. If so, then no further analysis would be necessary. If not, then it might be necessary to reevaluate the situation to reduce conservatism and, possibly, resort to a specialized analysis which deals with the effects of two simultaneous external events. ?

5. RESPONSE

I thought that this title is going to be changed

5.1 Introduction

Two approaches may be considered in a seismic PSA model. Response is a good basis for this discussion. When an event occurs, having a value r_e, it is denoted as a random variable R or a sample value r. The real value of an event may be very large in some cases, and we should expect failure on the considered structure. During the design procedure, we usually assume a value for the design r_d, and the real value r_e of all events is expected to be lower than this design value. However, the relation is probabalistic, and r_e may in some cases exceed r_d.

One method, which may be used, is to assume the distribution and other stochastic parameters of R, for example, response factor f_r and its standard deviation β_r

Another method is to consider some real events deterministically and to examine the relative margin to the design value. The real values is, then, determined using this method. Several alternative ways may be used for this: one is the value estimated from actual records on structural behaviour in the site and its surrounding area. The second one is the value obtained from the design value after removing extra margins and finding the upper bound condition. The third one is a value which would be assumed as characteristic for a certain large region having the size of the east-coast of North America for example. The first approach is frequently used for the existing damage reports. This approach and the second approach may be useful to reexamine aseismic design of existing plants or to review the practice of design. They are also beneficial to evaluate the individual design of a new plant, and to modify the detailing of design. The third one is useful in examining the total safety margin of a plant which is going to be built in an area where we have no precise information. It is also used in decreasing the probability of its core damage by efficient improvement in the design.

this causes the fluctuation of response, but the lack of our knowledge on its path and the detailed structure of deep strata contributes to its uncertainty. We can estimate the randomness of the response, and add an adequate margin to its design basis response spectra in general.

5.2.2 Soil

Soil characteristics are studied prior to the seismic design of nuclear power plants. These characteristics are described by the Young's Modulus, density, and Poisson's ratio or the shear wave velocity V_s and compressive wave velocity V_p. There are several ways to measure or estimate these values, however, there may be significant differences on the result obtained depending on the method employed.

for what parameter?

In some cases, the difference may reach the order of 10^1 or 10^2. This is caused by unknown effects of soil composition or grain size distribution and also by testing methodology and environmental factors. It is difficult to decide on the values for these parameters and the degree of their scattering in deep strata without any samplling.

5.2.3 Building

The uncertainty of response of a reactor building is mainly governed by the Soil Structure Interaction (SSI). If the equivalent Young's Modulus* of a building is nearly equal to that of supporting soil, its response characteristic is very sensitive to the value of soil parameters.

frequency of building and soil column

Other uncertainties related to the buildings themselves are usually not so large, if their construction is adequately controlled, and if non-linear characteristics subjected to load exceeding their elastic limit are carefully examined by various levels of testing for both types of concrete structure R.C. and S.C. But there is still some uncertainty on their buckling behaviour, especially of pre-stressed concrete structures.

for concrete structure?

"Up-lifting" of a structure is a non-linear behavior, and it contains some uncertainty contributed by soil characteristics and boundary conditions.

The difference of magnitudes of each earthquake contributes to the flucturation in terms of the duration and the lower frequency limit of spectrum. In general, the larger magnitude earthquakes contain more low frequency components. The affect on the duration is important for structural failure of plants, but it does not influence the spectrum so explicitly.

The design basis spectra usually takes such points into consideration, and they cover such stochastic randomness, by incorporating one to three times the standard deviation σ. Therefore, the margin which covers such fluctuations in the design basis spectrum should be considered in view of their stochastic nature.

5.3.3 Parameters

Many parameters used throughout the process of response analysis. Degrees of fluctuations of these parameters depend on their stochastic characteristics as well as their uncertainties.

The details of the effects on each parameter will be discussed in the Section 5.4.

5.3.4 Aging Effects on Parameters

Aging effect has a significant contribution to response. Moreover, recently it was considered to use the plants far longer than the life planned in the beginning. Some items, especially, the strength of reinforced concrete and the total characteristics of steel components used for pressure boundary may reduce their margin after some years' use. Their structural characteristics vary from one year to another. However, research on this subject has not been thoroughly developed.

5.3.5 Man-induced Fluctuation

This is one of the more difficult subjects compared to others. The effect of "human factor" has not been discussed much on PSA studies in the

6. FRAGILITY

I thought the title is going to be changed

6.1 Introduction

The derivation of appropriate fragility curves for buildings, and equipment and piping systems requires substantial effort. In this chapter, simplified as well as certain sophisticated methods will be discussed that are used in the derivation of fragility curves.

This statement will not be supported by any experience.

Various methods may be proposed for the derivation of the fragility curve, however, at this moment, it difficult to say that the practice is well established. In the "PRA Procedures Guide", these methods are described, and Section 11.2 Seismic Risk Analysis of this Guide is attached as Appendix D. There are certain indications related to the behaviour of a structure which exceeds its design level under earthquake loading. The most important building is the reactor building, and the number of buildings which require fragility analysis is limited, therefore sophisticated methods are applicable for these. But the number of safety related equipment and piping is enormous, and it is not so simple to apply such methods. In this chapter, detailed techniques will not be described, but their main features will be presented.

The detailed relation of the failure of equipment and piping systems to the degree of failure of the supporting structure has not been discussed in recent years. This relation will be discussed in the next section and in Appendices A and B. This can bring some understanding for the failure modes of equipment and piping systems.

For the event flow chart, possible sequences initiated by the failure of a building and/or equipment and piping systems will be obtained based on certain scenarios. For this practice the fragility curves of the supporting building, supporting devices, equipment and piping systems are required. There are various ways to estimate the fragility curve; in any case, it is important to eliminate safety margins which exist in the design process. In the previous chapter, this problem was already discussed.

Selection of β_R and β_U as well as the median F are key points of this method. Information on randomness may be obtained through data analysis. It is possibile to obtain new and adequate local information for a particular site related to randomness. But it is difficult to obtain new information on uncertainty, and the existing values are generally used. It should be pointed out however, that values related to uncertainty are more significant than those for randomness for the evaluation procedure.

6.3.2.4 H.C.L.P.F. Method

H.C.L.P.F method will not give you core damage frequency!

High confidence, low probability of failure concept involves obtaining a value from the fragility curve having a high confidence. If this value can be calculated for each item, then the value for the total system can be obtained. In this procedure, it is not necessary to consider the stochastic parameters of each item. This may bring a new non-stochastic evaluation technique. By using this method, we can eliminate the fragility curve itself from further use. This value may be obtained by testing or other methods.

6.3.2.5 Fracture Mechanics Approach

In this approach, first we assume the existence of a certain number of undetected cracks on the wall of a piping or a vessel, and evaluate its growth by using the theory of fracture mechanics. This method is not practical for the evaluation of the total system. It is applicable to a degraded system, especially to combine with other load effects.

6.3.2.6 Total Energy Absorption Method

This is also a kind of simulation, however, it is applicable to all the items. Based on the critical energy to failure, which is obtained by testing individual items, we estimate the degree of failure using the energy absorbed by the structure. Methods previously discussed, especially the load factor

6.5.7 Storage and Thin-walled Vessels

RWST and refueling very important tanks

These storage tanks related to the safety of NPP are rather few except for the waste management system. The failure mechanism has been studied for oil storage tanks, but the exact mechanism of failure including so-called "Elephant Foot Buckling" or "Bulging" is not clear. For smaller size (maybe less than 5 meters in diameter) tank, it is rather simple, because the consideration on EFB is not necessary, but the ordinary buckling criteria should be applied, if its proportion requires it.

6.5.8 Control Rod and Control Rod Drive Mechanism

CRD Mechanism both for BWR and PWR are easily subjected to the effect of relative deformation between R/V and the supporting structure. It may be caused by the skirt deformation in the case of BWR, and by a failure of R/C as wall or a slip of PCL (primary coolant loop) with R/V in the case of PWR. In a case of PTR, it depends on their detailed design. For CANDU reactors this may occur by relative deformation of a main building floor or partial or complete failure of anchorage of the reactor vessel.

As already described in the previous section, the significant delay of insertion of control rod is a key mode of failure. There are several modes of failure, like loss of driving function of CRD system, buckling of control rod itself (PWR), jamming of control rod cover plate (BWR), and so on, but these modes are only related to their operability, as long as the significant delay does not occur.

6.5.9 Active Components

The definition of a safety-related active component is as follows: an item which is required to satisfy its function during and after the event without any additional condition compared to its normal state. The items described in the following several sections are active components in this sense.

8. INTERPRETATION OF THE RESULTS

Within the scope of the present document, the results of a seismic PSA consists of the distribution function of the annual frequency of core damage [Fig. 8.1]. It is customary to synthesize this information into the median or mean value of the frequency and two fractile values (lower and upper) defining a range of frequencies within the 90% (or any other desired fraction) of the frequency probability is contained.

in log normal distribution
mean reflect uncertainty

The central value (median or mean) can be roughly thought to reflect the contribution to the risk due to instrinsic randomness while the "confidence interval" gives the measure of uncertainty with which the core damage frequency is obtained.

An assessment of the adequacy of the design is made and identification of the major items which may contribute to the core damage is listed.

For example, the frequency of core damage resulting from seismic event is compared to that of other external events such as flood, wind and fire. The frequency of core damage resulting from each of these events is ~~then~~ identified, Design modifications can then be made to reduce the core damaged frequency.

and the percentage of contribution from each of these ~~external~~ events is listed.

Also, the core damage frequency resulting from seismic events or all external events should be compared to the core damage frequency resulting from the ~~the combination of~~ internal ~~and external~~ events, taking into consideration the uncertainty associated with each of the events.

Improvement in the fragility values of the systems and components may contribute in lowering the core damage frequency resulting from a seismic event.

Probabilistic Seismic Safety Assessment
Review of the USA Approach and Interests

A.K. Ibrahim
NRC

1. INTRODUCTION

In the United States, Probabilistic Seismic Safety Assessment (PSSA) has been used by the NRC staff for the last decade. Within that decade, a lot of developments and improvements in the methodology have come about. Generally the staff has used the results from PSSA in comparative manner in order to gain some insight on improving structures, systems, and components important to safety.

This summary provides my views on the current status of PSSA in the USA. It describes an outline of the past and current NRC application of PSSA to the resolution of seismic safety issues.

2. DESIGN

Within the last two decades, increasing attention has been devoted to the seismic design of nuclear power plants. Two of the main concerns which are frequently raised are:

1. How adequate is the seismic design as compared to current safety requirements?

2. What is the margin of safety in the seismic design?

In response to the first concern, the Systematic Evaluation Program (SEP) was undertaken. This program reviewed old plants in the eastern and central USA making use of probabilistic seismic hazard estimates. These estimates were used in a relative manner and no emphases were placed on the absolute numbers. Another example is the use of PSSA in resolving the Charleston Earthquake issue. In this issue, probabilistic seismic hazard estimates are being made for nuclear power plants located in the eastern US. The main purpose of this study is to examine the likelihood of exceeding the SSE of these nuclear power plants at different levels of ground motion.

The second concern dealing with the margin of safety lead to the formulation of several Unresolved Safety (USI) and Generic Issues. These programs make use of insights gained from probabilistic analysis and its consequences at selected nuclear power plants, and recommend the revision to the SRP to reflect the state-of-the-art.

3. LICENSING

In the rules and regulations of NRC there is no statement requiring an applicant to perform a PSSA as one of the requirements for granting a license for NPP. The NRC staff uses the results of PSSA mainly to gain insight and suggest improvement in some situations when needed based on the PSSA. There is a great awareness in the USA that performing a PSSA will enhance the safety of the plant and in some instances may result in a favorable cost/benifit outcome. Also it can be said that performing a PSSA may provide a warm feeling about the safety factors which may be existing in the seismic design.

Recently, the NRC Commission has mandated that in order to obtain final design approval for future standard plants, a PSSA should be performed to show that the design is acceptable.

4. RESEARCH

From the experiences gained so far from performing several PSSA, new avenues of research have been opened up such as the seismic design margin program, elastic-plastic seismic analysis methods, nonlinear structural dynamic analysis procedures, and the soil-structure interaction. All these programs will provide insight and will lead to improvement of requirements to build safer nuclear power plants.

this paper may provide you with additional insight of the use of PRA.

8th Symposium on Earthquake Engineering, Roorkee
December 29-31, 1986, Vol I, PP 609-617.

OVERVIEW OF SEISMIC DESIGN CRITERIA FOR NUCLEAR POWER PLANT STRUCTURES*

N.C. Chokshi (I)
G. Bagchi (II)

SUMMARY

A number of significant changes from the current licensing acceptance criteria are being considered in the area of the seismic analysis/design of the U.S. nuclear power plant structures. These proposed changes are discussed along with the current requirements.

I. INTRODUCTION

In the United States, for the licensing of nuclear power plants, it is required, in part, that (Ref. 1) "Structures, systems, and components important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, ...without loss of capability to perform their safety functions..." In order to assure compliance with this requirement, the U.S. Nuclear Regulatory Commission (NRC) staff issued several Regulatory Guides (e.g. Ref. 2) and a Standard Review Plan (SRP) outlining the NRC staff's requirements in each step of the seismic analysis/design of a nuclear power plant structure. Broadly, these steps include: (1) seismic design ground motion; (2) soil-structure interaction analysis; (3) structural dynamic analysis; and (4) development of input motion for the design of components and equipment, within a structure. The seismic design requirements for equipment are discussed in a companion paper in this conference (Ref. 3).

The seismic design of nuclear power plants have received increasing attention since the late sixties and early seventies. The NRC staff issued its first SRP in 1975 (Ref. 4). Subsequently, a revision to this SRP was issued in 1981 (Ref. 5). In the mid-seventies, as the total seismic design process evolved, two questions faced those concerned with the seismic safety of nuclear reactor facilities:

a. How adequate are the plants in earlier generations with respect to current safety requirements?

b. What is the margin of safety in the overall seismic design process?

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In order to address the first question, a Systematic Evaluation Program was undertaken which examined a number of older plants in a comprehensive manner. The second question lead to a formulation of a so-called Unresolved Safety Issue (USI) titled Seismic Design Criteria. The stated objectives of this USI were "...to investigate selected areas of the seismic design sequence to determine their conservatism for all types of sites, to investigate alternate approaches to parts of the design sequence, to quantify the overall conservatism of the design sequence, and to modify the NRC criteria in the Standard Review Plan if changes are found to be justified." However, it was decided later to restrict the scope of this USI to recommend the revision to the SRP to reflect the state-of-the-art.

Several research studies under this USI included the following: (1) elasto-plastic seismic analysis methods; (2) site-specific response spectra; (3) nonlinear structural dynamic analysis procedures; and (4) soil-structure interaction. The final task in the USI was to review the results of other studies and recommend changes in the Standard Review Plan and Regulatory Guides. Thus, NUREG/CR-1161, "Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria," (Ref. 6) is considered to present technical findings of the USI. In addition to the research related to this USI, the NRC staff also sponsored a workshop in June 1986 (Ref. 7) to discuss licensing criteria in the soil-specific interaction analysis.

The objective of this paper is to describe the current seismic requirements described in 1981 SRP (Ref. 5) and then discuss the proposed changes to these requirements based on the findings of the above-discussed USI and workshop.

II. CURRENT SEISMIC DESIGN REQUIREMENTS

For the purposes of discussion, the current seismic design requirements for structures are divided into the following major four areas: (1) design ground motion; (2) soil-structure interaction analysis; (3) modeling and seismic analysis methods for structures; and (4) development of the within-structure (floor) response spectra for the analysis of components and equipment. An extensive discussion of the current total seismic design requirements of nuclear power plants can be found in Ref. 8.

Design Ground Motion: Nuclear plants in the U.S. are currently designed for two levels of earthquake. The "safe shutdown earthquake" or "SSE" is that earthquake which is based upon an evaluation of the maximum earthquake potential considering the regional and local geology and seismology and specific characteristics of local subsurface material. It is that earthquake which produces the maximum vibratory ground motion for which certain structures, systems and components are designed to remain functional. The "operating basis earthquake" or "OBE" is that earthquake which, considering the regional and local geology and seismology and specific characteristics of local subsurface materials, could reasonably be expected to affect the plant site during the operating life of the plant; it is that earthquake which produces the vibratory ground motion for which those features of the nuclear power plant, necessary for continued operation without undue risk to the health and safety of the public, are designed to remain functional.

Based on geological and seismological investigations, the SSE and OBE levels are generally defined in terms of the peak ground acceleration level and associated design response spectra. For most U.S. plants, OBE is, generally, at least one-half of SSE. The design response spectra can be either site-specific spectra or a broad band design spectra defined in NRC Regulatory Guide 1.60 (Ref. 2). The design response spectra in Regulatory Guide 1.60 are derived by analyzing, evaluating and statistically combining a number of individual response spectra derived from the records of significant past earthquakes. It is important to recognize that Regulatory Guide 1.60 spectrum does not represent a single seismic event and, therefore, is not necessarily consistent with the local site characteristics.

In order to perform structural dynamic analysis, particularly to generate floor response spectra, in the past it has been necessary that a time history be available. Use of a synthetic time history whose response spectra generally envelope the design response spectra for all damping values is permitted. In particular, the acceptance criteria for the use of a synthetic time history states that "When spectral values are calculated from the design time history the frequency intervals are to be small enough such that any reduction in these intervals does not result in more than 10% change in the computed spectra." An acceptable set of frequencies at which the response spectra may be calculated is also included. Another acceptable method is to choose a set of frequencies such that each frequency is within 10% of the previous one.

The acceptance criterion for meeting the spectra-enveloping requirement is that no more than five points of the spectra obtained from the time history should fall below, and no more than 10% below, the design response spectra.

Soil-Structure Interaction (SSI) Analysis: Once the design ground motion (response spectra and time-history) is determined for soil founded structures the next step is to perform the soil-structure interaction analysis. The current requirements for the SSI analysis specify that the design ground motion discussed earlier is to be specified at the foundation level of the structures in the "free-field." In addition, requirements state that:

"At present, most commonly used methods are the half-space and the finite boundaries modeling methods and there is no indication as to which one is more reliable, especially when too many assumptions are involved. Therefore, modeling methods for implementing the soil-structure interaction analysis should include both the half-space and finite boundaries approaches. Category I structures, systems and components should be designed to accommodate responses obtained by one of the following:

a. Envelope of results of the two methods,

b. Results of one method with conservative design consideration of effects from use of the other method,

c. Combination of a. and b. with provision of adequate conservatism in design."

Modeling and Seismic Analysis Methods for Structures: For modeling of the structure, the following guidance and requirements are spelled out:

a. In order to minimize the complexity of modeling, criteria for components within the structure which are not required to be explicitly included (except for their masses) in modeling are defined in terms of the mass ratio, R_m, and the frequency ratio, R_f.

$$R_m = \frac{\text{Total mass of the supported subsystem}}{\text{Total mass of the supporting system}}$$

$$R_f = \frac{\text{Fundamental frequency of the supported subsystem}}{\text{Dominant frequency of the support motion}}$$

The following criteria are acceptable:

(1) If $R_m < 0.01$, decoupling can be done for any R_f.

(2) If $0.01 \leq R_m \leq 0.1$, decoupling can be done if $0.8 \geq R_f \geq 1.25$.

(3) If $R_m > 0.1$, an approximate model of the subsystem should be included in the primary system model.

b. The number of masses is considered adequate when additional degrees of freedom do not result in more than a 10% increase in responses. Alternately, the number of degrees of freedom may be taken equal to twice the number of modes with frequencies less than 33 cps.

c. The specific percentage of critical damping values used in the analyses of structures are considered to be acceptable if they are in accordance with Regulatory Guide 1.61, "Damping Values for Seismic Design of Nuclear Power Plants." The damping value for soil must be based upon actual measured values or other pertinent laboratory data considering variation in soil properties and strains within the soil.

For conducting dynamic analysis, the requirements are as follows:

a. Use of either the time history method or the response spectrum method.

b. Consideration of the torsional, rocking, and translational responses of the structures and their foundations.

c. Investigation of a sufficient number of modes to assure participation of all significant modes. The criterion for sufficiency is that the inclusion of additional modes does not result in more than a 10% increase in responses.

d. Consideration of maximum relative displacements among supports of Category I structures, systems, and components.

e. Inclusion of significant effects such as piping interactions, externally applied structural restraints, hydrodynamic (both mass and stiffness effects) loads, and nonlinear responses.

f. When the response spectrum method of analysis is used to determine the dynamic response of damped linear systems, the most probable response is obtained as the square root of the sum of squares (SRSS) of the responses from individual modes. For closely spaced modes (two modes having frequencies with 10% of each other) special consideration is identified in Regulatory Guide 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis."

g. When the analysis of a structure is carried out separately for each of the three components of earthquake motion, the resulting responses are combined using the SRSS rule. When three components are applied simultaneously to a structure (as in a time-history analysis), the responses can be combined algebraically provided the components are statistically independent.

h. Either the composite modal damping approach or the modal synthesis technique can be used to account for element associated damping.

Development of With-in Structure Response Spectra:

For the design of components within structure, which are not integral to the structural model discussed above, there is a need to develop with-in structure or floor response spectra. These spectras are used as input for components design. Currently, to be acceptable, the floor response spectra should be developed taking into consideration the three components of the earthquake motion. The individual floor response spectral values for each frequency are obtained for one vertical and two mutually perpendicular horizontal earthquake motions and are combined according to the "square root of the sum of the squares" method to predict the total floor response spectrum for that particular frequency, Regulatory Guide 1.122.

In general, development of the floor response spectra is acceptable if a time history approach is used. If a modal response spectra method of analysis is used to develop the floor response spectra, the justification for its conservatism and equivalency to that of a time history method must be demonstrated by representative examples.

Consideration should be given in the analysis to the effects on floor response spectra (e.g., peak width and period coordinates) of expected variations of structural properties, dampings, soil properties, and soil-structure interactions. Any reasonable method for determining the amount of peak widening associated with the structural frequency can be used, but in no case should the amount of peak widening be less than ±10%. If no special study is performed for this purpose, the peak width should be increased by a minimum of ±15% to be acceptable.

III. PROPOSED CHANGES TO SEISMIC DESIGN REQUIREMENTS

The discussion in this section is again divided into four areas as in the previous section. Additional requirements for special structures are also briefly discussed.

Design Ground Motion: Recent studies (Ref. 9) indicate that numerically generated artificial ground acceleration histories produce power spectral density (PSD) functions having a quite different appearance from one individual function to another, even when all these time histories are generated so as to closely envelop the design response spectra. For example, the use of the available techniques of generating acceleration histories that satisfy enveloping Regulatory Guide 1.60 (Ref. 2) spectra usually results in PSD functions which fluctuate significantly and randomly as a function of frequency. It is also recognized that the more closely one tries to envelop the specified response spectra, the more significantly and randomly do the spectral density functions tend to fluctuate and these fluctuations may lead to an unconservative estimate of response of some structures, systems, and components. Therefore, when a single artificial time history is used in the design of seismic Category I structures, systems, and components, it is proposed that this time history should satisfy requirements for both response spectra enveloping and matching a PSD function smoothly distributed over the frequency range of significance.

For Regulatory Guide 1.60 spectra, a target spectral density (Kanai-Tajimi form), is specified. An artificial time history, having satisfied both the response spectrum and power spectral requirements, may be used as a representative seismic input for design purposes after being properly scaled.

In order to overcome the deficiency resulting from the use of single time-history, an option to use multiple real or artificial time histories is also being proposed. The use of multiple time histories is attractive from many points of view (Ref. 10). When multiple analyses are performed in a systematic format, one can explicitly account for the recognized variability in definition of the seismic input and in the system characteristics (properties of the soil, structures, piping systems, equipment, etc.). The degree of conservatism due to the response calculational process is, hence, quantified. This leads to a more balanced design, in particular, for subsystems whose input environment is defined by in-structure response spectra; their values being smoother and broader than spectra obtained from conventional single time history analysis. Currently, a minimum of five time histories are proposed for such multiple time histories analyses.

Soil-Structure Interaction Analysis: The NRC sponsored a workshop on Soil-Structure Interaction (SSI) in June 1986 to discuss licensing issues. The proposed changes in this area are based on recommendations resulting from this workshop (Ref. 7). It was noted at the workshop that in order to employ a suitable state-of-the-art approach to perform the SSI analysis without resorting to the current enveloping requirements (see Section II), one must use site-specific ground motion. It is also being considered that this ground motion should be defined to be at a free ground surface (rather than at the foundation levels as in the current requirements).

Two cases are identified depending on the soil characteristics at the site. For relatively uniform sites of soil or rock with smooth variation of properties with depth, the control point should be specified on the soil free surface at the top of the grade. The free-field ground motion or control motion should be consistent with the properties of the soil profile. For sites composed of one or more thin soil layers overlying a competent material, the control point is specified on an outcrop or a hypothetical outcrop at a location on the top of the competent material at the free surface. The control motion specified should be consistent with the properties of the competent material.

It is noted that there is enough confidence in current methods used to perform the SSI analysis to capture the basic phenomenon and provide adequate design information; however, the confidence in the ability to implement these methodologies is uncertain. Therefore, in order to assure proper implementation, the following general guidelines should be observed in performing SSI analysis.

a. Perform sensitivity studies for important parameters to assist in judging the adequacy of the final results;

b. Through the use of some appropriate benchmark problems, the user should demonstrate its capability to properly implement any SSI methodologies; and

c. Perform enough parametric studies with the proper variation of parameters to address the primary uncertainties (as applicable to the given site).

Modeling and Seismic Analysis Methods for Structures: The proposed requirements in this area essentially remain unchanged except for consideration of high frequency modes (i.e. modes in excess of about 33 Hz) in the dynamic analysis.

The SRSS combination of modal responses is based on the premise that peak modal responses are randomly time phased. However, at frequencies approximately equal to the frequency at which the spectral acceleration, S_a, returns to the peak zero period acceleration, ZPA, or greater, this is not a valid premise. At these high frequencies, the seismic input motion does not contain significant energy content and the structure simply responds to the inertial forces from the peak ZPA in a pseudo-static fashion. The phasing of the maximum response from modes at these high frequencies (roughly 33 Hz and greater for the Regulatory Guide 1.60 response spectra) will be essentially deterministic and in accordance with this pseudo-static response to the peak ZPA.

There are several solutions to the problem of how to combine responses associated with high frequency modes when the lower frequency modes do not adequately define the mass content of the structure. The procedure, suggested in Ref. 6, appears to be the simplest and accurate for incorporating responses associated with high frequency modes (beyond about 33 Hz), and is the procedure incorporated in the proposed revision.

In addition, it is also recognized that the dynamic analysis can be performed by any suitable method such as response spectrum analysis, frequency domain analysis, time history analysis, etc.

Development of With-in Structure Response Spectra: When a single artificial time history is used for the design, the provisions discussed earlier are still applicable. However, now two other options are also proposed to develop the floor spectra: (1) to develop floor spectra through the use of multiple time histories; and (2) to develop floor spectra through direct generation.

The overall benefits of the use of multiple time histories include smoother, less sharply peaked floor spectra without additional conservatism introduced by peak broadening, spectra easier to replicate in tests, and easier to recognize and directly include uncertainty. Of course, the use of multiple time histories will require more calculations.

The proposed revision also includes the use of direct generation methods which allow the generation of in-structure response spectra directly from the ground response spectra without time history analysis provided these methods are shown to produce results comparable to the results obtained by a time history analysis. The direct generation methods are economical for parametric studies and would reduce the uncertainties associated with in-structure spectra generated from synthetic time histories.

Special Structures: Requirements are being proposed for special structures such as above ground vertical tanks and underground buried piping structures. In the past, the tanks have been designed on the assumption that the tank walls are rigid (Ref. 6). However, typical large metal tanks in the nuclear power plants have fairly flexible walls and it is now recognized that one can underestimate the tank forces by a factor of 2 to 2.5 if the tank wall flexibility is ignored. The proposed requirements detail the tank analysis approach which accounts for the tank wall flexibility.

IV. CONCLUSION

A very broad overview of the current seismic design requirements and changes under consideration has been presented. Much more detailed information on the proposed changes and other changes are discussed in Refs. 6 and 10. Some other related changes, such as elimination of two levels of earthquake (OBE and SSE) for design purposes and piping analysis requirements, are discussed in Ref. 8.

In conclusion, it is noted that the proposed changes in the requirements for the seismic analysis of the nuclear power plant structures will lead to: (1) upgrading of requirements to reflect the current state-of-the-art; (2) elimination of potential sources of nonconservatism, such as the consideration of the wall flexibility in the tank design and consideration of high frequency modes; and (3) removal of unquantifiable excessive conservatism by permitting the option of the multiple time-histories and direct generation of the floor spectra.

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