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SUBJECT: Forwards comments on draft NUREG/CR, "Review of Oconee-3 PRA: External events Core Damage Frequency," transmitted in 851118 ltr. PRA has not omitted important technical info. Data which could not be put into final document requested.

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March 3, 1986

Mr. Harold R. Denton, Director
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Attention: Mr. John F. Stolz, Project Director
PWR Project Directorate No. 6

Subject: Oconee Nuclear Station
Docket Nos. 50-269, -270, -287

Dear Sir:

By letter dated November 18, 1985, the NRC transmitted a draft NUREG/CR document entitled, "A Review of the Oconee-3 Probabilistic Risk Assessment: External Events, Core Damage Frequency". The NRC requested Duke to review and provide comments regarding the draft document. Attached, please find Duke's comments.

In general, throughout the document it is stated that sufficient information was not available for review. It should be noted that all information available for the original PRA was at the disposal of the review team and that Duke was more than willing to supply this information. In summary, Duke disagrees that the PRA itself has omitted important technical information.

Finally, Duke is making every effort to maintain the Oconee PRA and improve its quality. As such, Duke would like to obtain copies of the detailed models, computer runs, and other data which, for practical purpose, could not be incorporated in the final document.

Very truly yours,

H.B. Tucker /HBT

Hal B. Tucker

PFG:slb

Attachment

A001
1/1

cc: Ms. Helen Nicolaras
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

COMMENTS ON BROOKHAVEN NATIONAL LABORATORY'S REVIEW OF THE OCONEE PRA EXTERNAL EVENTS ANALYSES

1. In general, the reviewers state throughout the text that sufficient information was not available to BNL to adequately review portions of the external events analyses. It should be noted that all information available for the original PRA is at the disposal of the BNL review team, and that Duke Power Company is more than willing to supply this information for BNL to use. In many cases, the Duke engineers disagree that the PRA itself has omitted important technical information.

2. Pg 2-5, section 2.2.1 (last paragraph) - The Turbine Building drain is referred to as a $6ft^2$ hole. In reality, the hole is 6ft in diameter. If the incorrect value was used by BNL, it would have a major impact on calculated Turbine Building flood levels.

3. Pg 3-36, Section 3.3.3 - It is stated by BNL that the capacity factors were based on OBE (0.05g) rather than SSE (0.10g). Does this mean that in Table 3.5 (page 3-93) the median (g) column is based on OBE rather than SSE? If this is so, the table should be footnoted to reflect the OBE base.

4. Pg 3-38, Section 3.3.3 - Duke agrees with statements on this page concerning relay chatter. We suggest that items 11, 35, 36 and 37 in Table 3.5 all have capacity factors associated with relay chatter. A note should also be added to Table 3.5 stating that relay chatter has not been confirmed to cause unreliable operation.

5. Pg 4-4, Section 4.2.1 - The reviewers of the OPRA fire analysis have interpreted "remotely" in the PRA to mean "remotely from the control room." The word "locally" should be substituted, implying operation at the location of the same pumps. This meaning will significantly change the BNL requantification of the sequence of interest.

6. Pg 4-4, Section 4.1.2 - The reviewers claim that a fire could occur which damages the cables to the HPI pump suction valves but not power or control cables to the pumps. This fire allows the possibility that the pumps could start without the HPI suction valves open, thus damaging the pumps. Since the control cables for both the pump and suction valves are located in the same cable shaft area, a fire that would destroy the valve control cables but not the HPI pump control cables is extremely unlikely.

   Furthermore, there are fire detectors in the cable shaft that alarm on the fire panel in the control room. Thus, the operations staff would be aware of the fire in the cable shaft. They would also be aware, from the readings of either control room or SSF instrumentation, that the HPI system had not activated, and that it would have to be locally (i.e., from the HPI pump room area) activated. The procedure for plant shutdown following a fire in the auxiliary building (OP/O/A/1102/24) includes a step to manually open valve 3HP-24 to insure suction to the HPI pumps.

   For the above reasons, Duke Power believes that the core-melt scenario postulated by the reviewers is probabilistically insignificant.

7. Pg 4-8, Section 4.3.1 - It appears that the heading of the right column of the table on the bottom of the page has been mislabeled. It should read "Dam Failure Frequency."

8. Appendix E - Law Engineering Testing Company has provided Duke with extensive comments on BNL's review of the Jocassee Dam failure. Their review is included, in its entirety, as an attachment to these comments.

E.2(a)

Comment

Only potential circular failure surfaces were considered for both the dam and dikes. No wedge studies, which may be of particular interest for the dam, were included.

Response

Seed and Sultan (1967) studied stability analyses for sloping core embankment dams wherein the core material was assumed to be clay with no frictional component in its shear strength and sloping at a flat angle under the granular shell. They concluded that, for such sloping core dams, the slope failure would occur by a sliding block (wedge) mechanism involving sliding of the shell material along the interface of the flat-sloping clay core.

Jocassee Dam has a central core whose sides slope steeply and is not a gently sloping core embankment like the ones studied by Seed and Sultan (1967). The foundation is rock, and thus contains no layers of material weaker than the embankment materials. The critical circular failure surfaces determined in the analyses of the dam pass mainly though the rockfill and random rock shells of the structure, intersecting the core and filter zones only near the crest of the dam. Thus, the critical failure surfaces are primarily influenced by the rock fill and random rock materials, and the geometry of the cross section of the dam is such that sliding wedge mechanisms should not be more critical than the circular arcs used.

For the dikes, the homogeneous cross sections coupled with the strong foundation materials and thus the absence of weak layers along which failure surfaces might concentrate, make the consideration of wedge-type failure mechanisms unnecessary, in our opinion, to assess the probable behavior of the dikes during seismic loading.

Seed, H. B. and Sultan, H. A. 1967, "Stability Analyses for a Sloping Core Embankment," Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM-4

E.2(b)

Comment

In choosing potential failure circles for the dam, only those circles which break out at or near the level of the impounded water (and which would thus lead to overtopping of the dam) were considered. Other failure circles which may be associated with internal piping or liquefaction failure modes were not considered.

Response

See the response in Section D.3(g) concerning liquefaction.

Internal piping was considered at two stages of the analysis: 1. When setting the criteria for potential failure circles, those that produced either critical lowering of the crest or disruption of the filters sufficient to induce internal piping, were considered critical. The latter criterion actually dominated the choice of critical circles; 2. The critical displacement (D=36") in the definition of failure was judgmentally derived from considerations of induced rapid internal erosion.

E.2(c)

Comment

In the rigid body moment equilibrium analyses performed for each circle (for both the dam and dikes), seismic effects were included only in the driving moment computation, but not in the resisting moment computation. In addition, the impact of the vertical seismic component on the calculation was not mentioned.

Response

Except as noted below, the input motions were assumed to act in the upstream-downstream direction of the dam. This is a common assumption in earth dam design since it constitutes the most severe condition of shaking. Vertical motions do not induce as large shear stresses as horizontal motions and, therefore, do not affect the stability of earth dams to the same degree as horizontal motions. The method for calculating displacements is discussed in the response to Section E.2(f). For the calculation of permanent deformation, we rotated the horizontal component of motion to correspond with the direction of sliding (the direction producing maximum moment) and assumed this to be the only active component of motion. According to this commonly used model (e.g. Franklin and Chang, 1977), earthquake acceleration influences the driving force but not the resisting moment. The consolidated-undrained soil strength parameters were used for calculating the resisting moment due to soil resistance.

The assumption of the horizontal component of motion applied in the direction of the sliding creates, in effect, a vertical component acting in-phase with the horizontal component. The vertical component of recorded earthquake motion is typically smaller than the horizontal component. Sliding-block analyses have been recently conducted on another project at the Massachusetts Institute of Technology using all the records of the Franklin and Chang study, including the actual recorded vertical motions. These analyses have shown that the vertical component of motion has a negligible effect on the amount of sliding.

Franklin, A. G. and Chang, F. K. 1977, "Earthquake Resistance of Earth and Rock-Fill Dams; Permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis," Miscellaneous Paper S-71-17, Report 5, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

E.2(d)

Comment

Seepage forces were neglected in the analyses on the grounds that they would be of negligible effect except in the core of material of the dam.

Response

Seepage forces are those resulting from frictional drag on the individual soil grains by the seeping water. In general, as the seeping water moves through the soil, energy lost from frictional resistance due to interaction with soil and rock particles results in changes in the fluid potential.

What is neglected in the analysis is only the change in kinetic energy due to changes in seepage velocity from one vertical section (slice boundary) to the next, an insignificant factor generally not included in slope stability analyses by the engineering profession. The changes in the fluid potential between any two points will be given by the differences in elevation (or distance above a given datum) to which water would rise in an open tube (piezometer) placed and sealed at each of the points. This was the way in which the seepage pressures were accounted for in the analyses.

The phreatic surface slopes across the width of each slice, due to frictional drag on the individual soil grains. The water forces acting on each slice of the trial failure mass were assumed to vary linearly with depth vertically below the saturation (phreatic) line. An outward component of seepage pressure thus occurs due to the sloping phreatic line and was included in the analyses for its contribution to the driving moment.

The pore pressure acting on any location on the trial failure arc was (conservatively) assumed to be equal to the distance of the phreatic surface vertically above that location multiplied by the unit weight of water. This conservatively ignores the reduction in uplift (at depth below the phreatic surface) caused by the frictional losses associated with the vertical component of seepage. The lateral seepage force was accounted for as previously described.

E.2(e)

Comment

The peak seismic force applied to the rigid soil section is determined by applying an acceleration amplification factor to the peak bedrock acceleration estimated for the site. The amplification factor used is based, apparently, upon natural period estimates of the dam obtained from other elastic analyses.

Response

The fundamental natural period of Jocassee Dam was estimated by extrapolation from the results of natural periods by shaker tests determined for the reasonably similar El Infiernillo Dam (as reported by Prince, et. al., 1980). The extrapolation was done using theoretical two-dimensional shear beam

theory (Abdel-Ghaffer and Scott, 1979) accounting for the height, height-to-length ratio, and shear wave velocity of the dams. The shear wave velocity ratio of Jocassee to El Infiernillo was estimated by comparison of the unit weights, void ratios and average stresses in the dam materials.

The fundamental period estimate of the dikes was obtained using results of one-dimensional shear beam analysis (Sarma, 1979) and estimates of shear wave velocity of the embankment and foundation materials based on cross-hole shear wave velocity measurements made in compacted fill and undisturbed soil at Oconee Nuclear Station.

The non-rigid embankment and foundation materials above rock respond to the earthquake accelerations, and thus the embankment section is not considered rigid for this part of the calculations. The amplification factors used in the analyses were obtained from Sarma (1979), who obtained a closed-form solution for a shear-beam model of a system consisting of an embankment underlain by a foundation layer or equivalent layer. The earthquake motions are considered to be in the rock underlying the foundation layer and it is assumed that all motions are horizontal (the shear-beam model). Sarma (1979) plotted amplification factors against the fundamental period of the embankment-foundation system for nine strong motion earthquake records and for dimensionless parameters representing a range of geometry and material properties of the embankment-foundation system. [Amplification values obtained from finite element analyses on other projects are generally lower than the curves in Sarma (1979).]

Abdel-Ghaffar, Ahmed M., and Scott, Ronald F., SHEAR MODULI AND DAMPING FACTORS OF EARTH DAM, Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 105, No. GT12, December, 1979.

Prince, J., Navarro, I., and Alonso, L., "Strong-Motion Accelerograms of the 14 March 1979 Earthquake Recorded at the Dams," in Performance of El Infiernillo and La Villita Dams Including the Earthquake of March 14, 1979, Commission Federal de Electricidad, Mexico, Feb., 1980, pp. 33-50.

Sarma, S. K., 1979. "Response and Stability of Earth Dams During Strong Earthquakes," Miscellaneous Paper GL-79-13, U. s. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss., 1979.

E.2(f)

Comment

Estimates of the probability that the peak displacement of the soil block will exceed a critical value are apparently based on simple rigid body estimates for the case where safety factor is less than 1. These are included in an attempt to yield information on probability of cracking of the dam core material.

Response

The question of how much deformation is tolerable (the critical value) depends on such factors as the size and geometry of the dam, the zonation, the location of the sliding surface, and the amount of freeboard available. About 1 m (more precisely, 36 inches) of permanent displacement was chosen as the critical displacement or tolerable upper limit. This amount of critical displacement was chosen for Jocassee Dam based on consideration of the thickness of the filter materials; and for the dikes, the critical displacement value was chosen arbitrarily and is consistent with the thinking expressed by other engineers in the technical literature. Such a deformation would be considered as serious damage, but it is our opinion it would be tolerated in the dam and dikes without immediately threatening the integrity of the reservoirs.

The method for estimating the peak displacement of the failure block of a dam or dike was first presented by Professor Newmark in his 1965 Rankine lecture and has since been studied and explained by several investigators, and coherent procedures have been developed. It is assumed that the crest displacement will be in the average direction of the failure surface. The method is quite valuable in that, at some point, the computed displacements will increase quite rapidly with increasing peak acceleration, an indication that large deformations may then occur in the field and allowing evaluation of the dam or dike.

Inherent in the method is the assumption of a sliding failure surface along which the displacement occurs. At shaking levels below the point of development of a failure surface (safety factor greater than 1) crest deformations result from any overall distortion of the embankment and from compaction of rockfill. This latter performance experience at El Infiernillo Dam (during the Mexican earthquake of March 14, 1979) at shaking levels of up to but not exceeding the failure level, was incorporated into the displacement data and extrapolated by lines parallel to the theoretical displacement trend lines, thus effectively adding pre-slip surface displacements to the theoretical post slip-surface displacements for use in the evaluation of the dam and the dikes. This adds displacements at safety factors greater than 1 to those calculated at safety factors less than 1, resulting in combined displacements for determining when the critical displacement occurs. This is believed to be particularly conservative for the dikes, since at safety factors greater than 1 they are much less likely to distort or compact during earthquake shaking than is the dam, which has a rockfill shell.

D.3(a)

Comment

It is stated on page J-7-22 of the PRA report that

"...the Simplified Bishop Method is generally regarded as an accurate procedure of slope stability analyses. . . Errors have been estimated to be of the order of 10%; consequently the factor lambda-sub-m has been taken to have mean value 1 and standard deviation 0.1. . . ."

It should be pointed out that the error estimates mentioned above refers to the Simplified Bishop Method (used in the PRA report), as compared to the more

complete "Method of Slices", including internal rigid body, circular failure surface analyses, with the simplified Bishop Method merely reducing the calculation requirements by simplifying assumptions. However, it would be realized that actual estimates of errors in safety factor are much greater than the 10% mentioned above. The basic assumptions common to both methods lead to errors much greater than 10%. That is the reason for the use of safety factors 1.5 and greater being typically used for slope stability analyses of even the simples configurations.

Response

The Simplified Bishop analysis enjoys wide adoption and long usage in engineering practice. For evaluation purposes, it is considered one of the most suitable procedures; numerous comparisons in the technical literature (such as Wright and Kulhawy, 1973) have indicated it gives about the same numerical results as other more involved or more "complete" procedures of the Method of Slices. The justification for using the Simplified Bishop analysis is thus largely empirical, since the published comparisons show that it almost invariably is no more than a few percent different from the more rigorous procedures for computed values of overall safety factor. The referenced estimated error on the order of 10% is conservative based on the technical literature and relates to the computational accuracy of the method when compared to more involved analytical methods, not the accuracy of the actual overall safety factor. The other sources of uncertainty concerning the safety factor were included and involve the soil strength parameters and reduction in resisting moment with slope movements.

Wright, Stephen G., and Kulhawy, Fred H., "Accuracy of Equilibrium Slope Stablity Analyses," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 99, No. SM10, Proc. Paper 10097, October, 1973, pp. 873-791

D.3(b)

Comment

The circular failure surface analyses (of which the Bishop Method is only one) is based upon the primary assumption of rigid block behavior. Its applicability is clearly to static problems wherein the time frame of interest is very large as compared to the time of propagation of stress waves through and around the dam. The method was modified in previous decades by the simple assumption of including a horizontal seismic pseudo-static inertia force. This was done because it was simple and could be handled in the precomputer age. With the advent of large finite element computer programs, more detailed analyses are now performed to try to ascertain estimates of stability of earth slopes.

Response

The Newmark approach to a deformational analysis has been well studied and contributions to a coherent procedure using this approach have been made by Professors Ambraseys and Sarma, Imperial College, London (e.g. Ambraseys ad Sarma, 1967; Sarma, 1975, 1979), the Berkeley group (Goodman and Seed, 1966, Makdisi and Seed, 1977), and the U. S. Army Engineer Waterways Experiment

Station (WES) (Franklin and Chang, 1977). The method thus is well described in the current engineering literature, and is a significant improvement over the precomputer "horizontal seismic pseudo-static inertia force" approach for indicating the probable behavior of dams and dikes such as those in the Oconee PRA study.

The method involves two steps. The first is to obtain the dynamic response of the dam or dike to earthquake ground motion and the second is to make the displacement calculations on potential sliding masses. Engineering judgment is required in applying the method and assessing the results, but because of its relative simplicity, the method is adaptable to probabilistic expression as has been done for the PRA study.

On the other hand, the finite element method, while also well-described in the current engineering literature, involves many more steps than the Newmark approach, and the application of engineering judgment in applying the method and assessing the results is much more involved due to the many steps and complexity of analysis and output data.

The finite element method is not currently adapted to a probabilistic approach to the degree which the Newmark method has been for this study. Therefore, the Newmark approach for assessing the behavior of the dams and dikes was selected as being more suitable than the finite element method for use in the PRA study.

D.3(b) References

Ambraseys, N. N. and Sarma, S. K., 1967. "The Response of Earth Dams to Strong Earthquakes," Geotechnique, Vol 17, No. 2, pp 181-213.

Franklin A. G. and Chang, F. K., 1977. "Earthquake Resistance of Earth and Rock-Fill Dams; permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis," Miscellaneous Paper S-71-17, Report 5, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

Goodman, R. E. and Seed, H. B., 1966. "Earthquake-Induced Displacements in Sand Embankment," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol 92, No. SM2, pp 125-146.

Makdisi, F. I. and Seed, H. B., 1977. "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol 104, No. GT7, pp 849-867.

Sarma, S. K., 1975, "Seismic Stability of Earth Dams and Embankments," Geotechnique, Vol 25, No. 4, pp 743-761.

Sarma, S. K., 1979, "Response and Stability of Earth Dams During Strong Earthquakes," Miscellaneous Paper GL-79-13, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

D.3(c)

Comment

By looking at the nonuniform configurations of both the dam and dikes, it is not clear that the potential circular failure surface is most critical.

Response

The computer program was used to search for failure surfaces. As indicated in the report, only those failure surfaces that, if activated, would induce flooding because they would lower the crest or fracture the filter to a significant degree, were considered critical in the results. The potential failure surfaces shown in the report are those meeting the above criteria.

D.3(d)

Comment

In all the analyses performed, seepage effects were neglected. Yet, it is well known that they will have significant effects on safety factor, particularly for configurations similar to the dikes. For the dam, pore pressure and liquefaction effects under seismic loadings are extremely important in the core material. These have not been evaluated.

Response

See the response to E.2(d) concerning clarification of how seepage effects and pore pressure were included in calculations for safety factor.

Pore pressure (changes from those existing under static conditions) under seismic loading of the soil comprising the dikes and the core of the dam were assumed to be reflected in the values of soil shear strength from the consolidated-undrained expressions of the triaxial shear tests. This is the approach commonly used in applying the Newmark deformation analysis.

See the response to D.3(g) concerning liquefaction effects under seismic loading.

D.3(e)

Comment

In the analyses, seismic effects were not included in evaluating resisting moments (page J-7-9 of the PRA report). Yet, in evaluating the soil shear strength along the failure surface (eq. 2), the seismic coefficient will clearly impact the intergranular stress term.

Response

The lack of seismic effects on the resisting moments were discussed in the response to E.2(c).

D.3(f)

Comment

The effects of vertical seismic earthquake coefficients have not been mentioned in the analyses, although it clearly will have an impact on the calculations.

Response

The lack of seismic effects on the resisting moments were discussed in the response to E.2(c).

D.3(g)

Comment

Other failure modes developed by pore pressure and soil liquefaction both through and under the dam have not been addressed. These are clearly important considerations for any dam as they perenially lead to catastrophic dam failures. This is particularly true for configurations similar to the Oconee Dikes.

Response

Past experience of embankment dams shaken by earthquakes has shown the importance of examining the performance of embankments actually shaken by earthquakes as a guide in making earthquake safety evaluations of other embankments. The following are some of the major conclusions by Seed, et al. (1977) concerning embankment dam performance under earthquake loading:

(1) Virtually any well-built, rolled fill dam can withstand moderate earthquake shaking, with peak accelerations of 0.2g and more, with no detrimental effects.

(2) Dams constructed of clay soil, on clay or rock foundations, have withstood extremely strong shaking ranging from 0.35 to 0.8g from a magnitude 8.25 earthquake with no apparent damage.

(3) Rockfill dams have withstood moderately strong shaking with no significant damage.

In summary, experience has shown that well-compacted, impervious rolled-fill dams are resistant to earthquake forces provided they are constructed on rock or overburden foundations resistant to liquefaction. The same is true of well-drained, compacted rockfill dams, although some surface deformation can be expected.

The rock shells of Jocassee Dam are not considered vulnerable to liquefaction due to their permeable nature.

Several factors contribute to the ability of the core soils of the Jocassee dam to withstand seismic loading. The core soils form a low-permeability mass under confinement by the rockfill shells. Measurements during construction of the dam indicated load transfer from the granular shells onto the soil core,

thus increasing the confining stresses in the core. Furthermore, the soils have significant fines (passing the No. 200 sieve) content making them more resistant to liquefaction than if the fines content was very low.

The Oconee dikes are composed of rolled fill of fine-grained soil supported on residual soils. The residual soil profile generally is very stiff to hard sandy silty clay and sandy clayey silt overlying saprolite from granite gneiss - firm to very dense silty sand having standard penetration resistances of about 20 to 100 blow per foot, all in turn underlain by partially weathered rock and rock. Such foundation soils are not of the type generally susceptible to liquefaction, nor is the rolled fill of the dikes.

Seed, H. B., Makdisi, F. I. and de Alba, P., "The performance of Earth Dams during Earthquakes," UCB/EERC-17/20, August, 1977.