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May 24, 1982

Mr. James G. Keppler, Regional Administrator
Directorate of Inspection and
Enforcement - Region III
U.S. Nuclear Regulatory Commission
799 Roosevelt Road
Glen Ellyn, IL 60137

Subject: Dresden Station Units 2 and 3
Quad Cities Station Units 1 and 2
Information Relevant to I.E.
Bulletin 79-14 Hanger Seismic
Operability and IR Nos.
50-237/82-01, 50-249/82-01,
50-254-82-01 and 50-265/82-01
NRC Docket Nos. 50-237/249/254/265

Reference (a): W. L. Stiede letter to J. G. Keppler
dated April 16, 1982.

Dear Mr. Keppler:

In a May 20, 1982, meeting with Messrs. D. Danielson and I. Yin of your staff, Commonwealth Edison committed to formally transmit certain information related to I.E. Bulletin 79-14 and our Reference (a) inspection report response. This information was previously summarized in a May 17, 1982, telephone call with Mr. C. Norelius.

What is the probability of a DBE Earthquake between now and December 1983?

The DBE or Safe Shutdown Earthquake (SSE) is .20g for Dresden Units 2 and 3 and .24g for Quad Cities Units 1 and 2. To estimate the probability of an earthquake, the Zion Probabilistic Risk Assessment (PRA) study was used. Figure 11 of the Attachment 1 DAMES & MOORE report entitled "Seismic Ground Motion Hazard at Zion Nuclear Power Plant Site for Pickard, Lowe, and Garrick" dated July 2, 1980, provides a plot of the composite annual probability of exceedance versus peak acceleration. This report is very similar to section 7.9.1 of the Zion PRA report.

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Using a value of .20g, which yields a conservative result for Quad Cities Station, Figure 11 predicts a probability of approximately 6×10^{-5} per year. Considering that there is approximately 1.5 years between now and December 1983, results in a total probability of approximately 9×10^{-5}. We believe that the seismicity curves employed for Zion represent a very reasonable approximation for the Dresden and Quad Cities sites because the seismogenic zones employed for Zion would also apply to Dresden and Quad Cities.

It should be noted that even if an earthquake of this magnitude should occur, the likelihood of piping or hanger loss of function during such an event is very small, based on the Zion PRA studies. The Zion fragility studies in Attachment 2, (Section 7.2 of the Zion PRA) specifically in Table 7.2-2, show that the median ground acceleration to failure for piping (which includes supports) is typically 10 or more times the SSE acceleration. Applying this safety factor to Figure 11 of Attachment 1 (probability curve) demonstrates the vanishingly small probability of an SSE induced piping/hanger loss of function. These safety factors reported for Zion are similar to those employed on a generic basis for other studies, and have in large part been confirmed by Lawrence Livermore in the Seismic Safety Research Margin Program.

Attachment 3, from section 7.9.2 of the Zion PRA, provides a further discussion of piping and support seismic capabilities, and is attached for your information.

We are assessing the unit-specific IEB 79-14 completion schedules discussed in the May 22, 1982, meeting, and will provide them to your office by the week of June 1, 1982.

Please address any questions you may have concerning this matter to this office.

Very truly yours,

Wayne L. Stiede

Wayne L. Stiede
Assistant Vice-President

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cc: Attachments

cc: Region III Inspector - w/o att.

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Assuming the allowable continuous duty load to be at least a -3β capacity for short term loading, the logarithmic standard deviation is calculated to be

$$\beta_C = 0.23$$

The estimated random and uncertainty variability associated with bearing load capacity are:

$$\beta_R = 0.15$$

$$\beta_U = 0.17$$

5.2.3 Generic Structural Capacities Derived from Design Criteria

In the majority of cases in risk studies, all detailed information regarding resulting stresses, deflections, bearing loads, etc., for safety related equipment is not readily available to the risk analyst. Classes of equipment must then be treated generically and the fragility descriptions derived from knowledge of design criteria and methods, service experience, etc. In this section, fragility descriptions are developed for those items of equipment whose failure modes are structural and for which design reports were not provided or summarized.

5.2.3.1 Piping and Supports

Piping and support designs provided by the architect/engineer were conducted to the requirements of the 1967 ANSI B31.1 Power Piping Code, Reference 32. That code required that the combined stress due to axial pressure loading, deadweight bending and bending due to the OBE loading be equal or less than 1.20 times an allowable stress S_h where S_h is typically set at 5/8 of yield or 1/4 of ultimate at temperatures below the creep regime for carbon steel and from 5/8 to 0.9 times the yield or 1/4 the ultimate for stainless steels. At temperatures near the reactor outlet temperature, the allowable S_h for stainless steel was typically 0.9 times the yield strength.

Piping supports were governed by the ANSI B31.1 Code and typically the stress allowable was S_h for materials listed in the code (piping materials) or 1/4 the ultimate strength for materials not listed. In addition, a 20% overload factor is allowed for short term loads and a 25% reduction in allowable is required for threaded supports which is a typical connection detail for seismic supports.

In order to determine the most probable failure mode for piping, we must examine design margins inherent for various pipe fittings when designed to code and compare these margins to those for supports designed to the applicable codes.

Our fragility description for piping systems will be based upon the single component type most likely to fail, i.e., pipe fitting, straight pipe, pipe support, etc. Failure of a pipe support does not necessarily mean failure of a piping system pressure boundary; however, the scope of this study does not permit side studies to determine the increased probability of a piping system failure given a support failure. Consequently, we are assuming that a support failure results in a failure of the piping system. In this comparison and in the development of fragility descriptions for pipe and supports, we will reference everything to the OBE design loading since that loading generally governs piping design plus, for Zion, the supports were designed for OBE loading only as DBE loading would always result in stresses less than yield if the OBE criteria were met.

5.2.3.1.1 Piping Failure Modes - References 46 and 50 compared pipe fitting collapse loads to code allowable loads for Class 1, 2 and 3 piping for Service Levels C and D. Both studies used almost identical data bases and both studies were based on current code criteria which for Class 2 and 3 piping are very similar to the ANSI B31.1 criterion used in Zion. The only significant difference in criteria for OBE loading is a slightly more liberal stress acceptance equation in current criteria. Equation 9 for Class 2 and 3 piping (Reference 9) is:

$$\frac{P_{max} D_o}{4t_n} + \frac{0.75i\ M_i}{Z} \overline{<} 1.2\ S_h$$

The ANSI B31.1 code equation used for Zion design was

$$\frac{P_{max} D_o}{4t_n} + \frac{i\ M_i}{Z} \overline{<} 1.2\ S_h$$

Both equations account for the axial stress due to pressure and the bending stress due to deadweight and earthquake induced moment, M_i. Current criteria only take credit for 75% of the combined deadweight and earthquake moment; however, the combination of 0.75i cannot be less than 1.0, where i is a stress intensification factor. This can have a slight effect on the most critical type of pipe fitting selected.

Reference 46 ranked pipe fittings in order of least to most conservative design as:

- Straight pipe
- Elbows and bends
- Branch connections
- Tees

Review of the data base revealed, however, that at room temperature the elbows were slightly less conservative in design than straight pipe. The same conclusion can be drawn from Reference 50. However, two factors must be considered in the ranking. First, for elevated temperatures, in the vicinity of the reactor outlet operating temperature, the straight pipe becomes slightly less conservative in design than elbows. This is due to a change in the governing criterion for establishing the allowable S_h as temperature increases, i.e., S_h is based upon yield strength instead of ultimate strength. Secondly, the Zion design equation more heavily penalizes pipe fittings with a stress intensification factor, i, significantly greater than 1.0 since the full value of $i \cdot M_i$ is used instead of 75% of the value. Since i for straight pipe and butt welds is 1.0, and for elbows is usually quite a bit larger than 1.0, the least design margin for Zion pipe fittings, if stressed to the code allowable, would be in straight pipe or butt welds. Butt weld joints are the most likely candidate for deriving the fragility description since they occur at terminal points in piping (anchors) where large seismic induced moments are likely to occur and they are more likely to contain flaws than wrought material.

5.2.3.1.2 Support Failure Modes - Supports for restraint of seismic inertial loads can be in the form of snubbers or rigid rod type supports and can be both horizontal and vertical. Vertical rigid rod type supports must also carry deadweight; thus, would carry proportionally less seismic load than theoretically allowed for lateral supports or vertical snubbers. If we assume then that the resulting stresses in each support type were at code allowable, a larger seismic margin would exist for vertical rigid supports than for lateral rigid supports or vertical snubbers. Thus, our fragility description for supports will be based on supports that carry only seismic load. In the case of snubbers, the snubbers themselves would be less likely to fail under the seismic loading than the attachments to the pipe or the building.

5.2.3.1.3 Piping Fragility Description - In order to establish a median factor of safety on piping capacity, we must,

1. Establish a range of piping capacity,

2. Estimate range of loading on piping due to weight, pressure and seismic,

3. Estimate range of ductility,

4. Estimate piping system collapse vs individual pipe element collapse.

Our range of piping capacity is estimated from a summary of test data contained in Reference 46. Collapse moment test results were reported for a variety of straight pipe sections of carbon steel and stainless steel for D/t ratios up to 50. For heavy wall pipe with $D/t < 25$, the median shape factor was about 1.5 for stainless steel and 1.67 for carbon steel. For D/t between 25 and 50, which is representative of the standard weights of pipe (Schedule 40), the median shape factor for both stainless steel and carbon steel is about 1.4. Since we are covering a broad range of pipe sizes, materials, and schedules we have selected a shape factor of 1.4 as a median value. Our median pipe capacity is then 1.4 times the yield moment. The logarithmic standard deviation on the shape factor is estimated from the test data to be about 0.15, of which the random portion is estimated to be about 0.1, with the uncertainty equal to 0.11. Considering that the median yield strength is about 1.25 times the code specified yield strength, the median moment capacity is about 1.75 times the yield moment determined from code yield properties. The variability in yield properties expressed as a logarithmic standard deviation is about 0.14 with $\beta_R = \beta_U = 0.1$.

The code allowable design stress expressed as a function of yield moment varies with material and temperature from about 0.43 to 0.87. The median value is about 0.62 so that the allowable stress of 1.2 S_h is about 0.75 times the yield moment determined from code properties. The ratio of static collapse load to allowable design load is then 1.75/0.75 or 2.33.

Equation 5-4 can be used to estimate the strength factor range by looking at the ranges of normal load and OBE load. Equation 5-4 in a slightly different form can be expressed as:

$$F_S = \frac{\frac{P_L}{P_D} - \frac{P_N}{P_D}}{\frac{P_{OBE}}{P_D}}$$

where P_L is the limit moment stress, P_D is the allowable stress used in the design, P_N is the stress due to deadweight and pressure and P_{OBE} is the stress due to the operating basis earthquake. We have already established a ratio for P_L/P_D along with a variability in the value. We now need to establish limits for P_N/P_D and P_{OBE}/P_D.

Typically, the deadweight stresses are 10-20% of P_D and pressure stresses are 20-40% of P_D resulting in a P_N/P_D ratio of 0.3 to 0.6. OBE stresses are typically 20-50% of P_D. Combining these assumptions, the total stress on a critical element is then assumed to vary from 50 to 110% of the allowable stress.

Using Equation 5-4 in the form above, minimum and maximum strength factors can be computed. If the range from minimum to maximum is considered to be approximately a $\pm 2\beta$ range, the median factor on strength is 5.9 relative to the OBE event. The logarithmic standard deviation for the assumptions made on loading is computed to be 0.27 which is considered to be all uncertainty. Combining this variability with the variability established for material yield and shape factor, the variability on strength is:

$$\beta_S = 0.34$$

$$\beta_R = 0.14$$

$$\beta_U = 0.31$$

The ductility for safety related piping is considered equivalent to that for the primary coolant piping which was stated to range from a minus two logarithmic standard deviation value of 1.5 to a median of 3.0. The ductility factor is then

$$F_\mu = 2.24$$

The variability expressed as a logarithmic standard deviation and accounting for variability in ductility plus the uncertainty in deriving the ductility factor from Equation 5-5 is:

$$\beta_{\mu} = 0.30$$

$$\beta_R = 0.15$$

$$\beta_U = 0.26$$

In order for a piping system to completely collapse, usually more than one collapse mechanism must form in the system; thus, basing fragility on the moment capacity of one fitting is conservative. A lower threshold of collapse could be likened to a simple beam where only one hinge is necessary for collapse. An upper threshold of collapse could be likened to a fixed-fixed beam where three hinges must form and the elastically calculated maximum moment is 1.5 times the pipe element collapse moment. If this is approximately a $\pm 2\beta$ range, the median system collapse factor is computed to be 1.22 and the logarithmic standard deviation, which is all uncertainty, is approximately 0.1

Combining all the factors and variabilities results in a median factor of safety on the OBE and variability, expressed in terms of logarithmic standard deviations, of:

$$F_C = 16.1$$

$$\beta_C = 0.46$$

$$\beta_R = 0.20$$

$$\beta_U = 0.42$$

5.2.3.1.4 Support Fragility Description - We stated previously that for supports stressed to their design limit under OBE conditions, the minimum margin would occur if the load were all seismic, i.e., the support carried no normal load. We will develop our fragility description for supports on this basis.

In order to apply Equation 5-4 to compute a strength factor or upper and lower bound factors, we must establish the range of material properties, the allowable design load and a range of applied load for the OBE event.

We will first assume the supports to be made of carbon steel equivalent to SA 675-Gr 50 with a code specified yield and ultimate strength of 25 and 50 ksi, respectively. The average yield and ultimate strengths will be considered to be 1.25 times the minimum. The allowable design load per the ANSI B31.1 code is 1/4 the ultimate strength increased by 20% for short term loading and decreased by 25% for threaded connections resulting in a design load of 0.225 times the code ultimate or for the material considered here, 0.45 times the code yield. The median yield to design allowable load is then 2.78 with an associated variability based upon the variability in material properties.

The applied load for the OBE event can vary widely from one support to another; thus, we assumed that a $\pm 3\beta$ load range could be 0.2 to 1.1 times the code design load. Using $\pm 3\beta$ material properties and the load range above in Equation 5-4, the $\pm 3\beta$ strength factors were computed and from the range of strength factors, the median factor was computed to be:

$$F_S = 5.9$$

The variability in the factor expressed as a logarithmic standard deviation was computed to be 0.42, of which the random portion representing randomness in material properties and other factors is estimated to be 0.14, with the uncertainty, primarily made up of loading range assumptions, being 0.40.

For threaded connections in tension, columns in compression and considering the possibility of anchor pullout in concrete walls, the ductility was estimated to range from 1.25 to 5.0. The median ductility factor, F_μ, is then about 1.9 with a logarithmic standard deviation, β_μ, of 0.30 of which $\beta_R = 0.1$ and $\beta_U = 0.28$.

Combining the strength and ductility factors and their variability, the resulting capacity factor and its variability are computed to be:

$$F_C = 11.2$$

$$\beta_C = 0.51$$

$$\beta_R = 0.17$$

$$\beta_U = 0.48$$

5.2.3.1.5 Governing Criterion for Piping - From the previous fragility descriptions, it appears that Zion piping has a higher factor of safety than supports. However, in a visit to the Zion plant, it was observed that the supports for small diameter piping appeared to be much larger in size relative to supports for larger pipe. We would conclude from a visual examination that supports would be more critical for large pipe than for small pipe. Also, it would appear that loss of support for very large piping would be more likely to cause pipe failure than loss of support for small piping. We, therefore, have based the fragility for piping eight inches in diameter or less on the pipe butt weld fragility description and the fragility for piping ten inches and larger on the pipe support fragility description.

The capacity factors for piping, listed in Table 5-6, reflect the above criterion.

5.2.3.1.6 Piping Subjected to Relative Building Motion - In Section 4.2.1, it was determined that soil failure beneath the containment building could occur at approximately 0.73g due to base slab uplift. The base slab uplift begins to occur at about 0.15g. A plot of uplift vs peak ground acceleration is shown in Figure 4-7. The effect of uplift and soil failure is to greatly increase the relative motion between the containment building and connecting structures. This has the effect of increasing the loading on pipes and pipe supports at or near points where they penetrat[illegible]tainment. Safety related pipelines that are connected between the containment building and other structures include:

- 34" and 35.5" main steam
- 16" main feedwater
- 10" containment spray
- 3" and 4" safety injection
- 12" and 14" residual heat removal
- 3" coolant charging
- 3" auxiliary feedwater
- 2", 3", 4" and 6" component cooling water

In addition, there are some underground pipelines connecting the recirculation and cavity flood sumps in the containment building with the auxiliary building equipment drain collection tank. At the point of soil failure under the reactor building, we could reasonably expect the buried piping to fail. The other non-buried piping systems do not necessarily fail, however. The increase in loading on these connecting pipes due to relative building motion is highly dependent on the stiffness of the pipe and the distance from containment to the first support in adjacent structures. An evaluation was made of one piping system in an attempt to quantify the consequences of base mat uplift and soil failure under the reactor building. A very stiff relatively short span of pipe was evaluated for the effect of relative building displacement. The 34 inch diameter main steam line was selected for this evaluation. The main steam line, upon leaving containment, travels about 20 feet through the safety valve room and then into the pipe tunnel. The steam pipe is supported just outside the safety valve room about 20 feet from the containment penetration. For this model, the pipe support reaction and pipe bending moment at the containment penetration were computed as a function of displacement of the containment building.

In order to develop a fragility relationship for the steam piping model, for various acceleration levels, the probability of occurrence of uplift displacements, δ_i, must be combined with the frequency of failure, given a displacement, and the products summed over the range of possible displacements.

$$\left[P_f | A_i\right] = \sum_{i=1}^{n} \left[P_{oc_{\delta i}} | A_i\right] \left[P_f | \delta_i\right] \qquad \text{where}$$

$\left[P_f | A_i\right]$ is the conditional frequency of failure given an acceleration level, A_i

$\left[P_{oc_{\delta i}} | A_i\right]$ is the probability of occurrence of one of several possible displacements, δ_i, for a given acceleration, A_i

$\left[P_f | \delta_i\right]$ is the frequency of failure for each δ_i

The summation is repeated for possible peak ground accelerations, A_i, and the resulting conditional frequencies of failure define the fragility curve (Cumulative Distribution Function). Figure 5-4 is the resulting fragility relationship for the 34-inch diameter steam line. Note that the CDF is not lognormally distributed in this case and is defined point by point even though capacity is treated as a lognormal function.

The capacity factor of the steam pipe, given an acceleration level, A_i, and displacement, δ_i, was computed as the product of Equations 5-4 and 5-5 as was done for all other equipment that fails in a structural mode. In the case of the steam line the treatment was more specific, thus the uncertainty is narrowed down from that derived for generic piping covering all pipe sizes, schedules, materials and operating temperatures. The resulting uncertainty on capacity defined as a logarithmic standard deviation about the median CDF is, $\beta_U = 0.33$. After establishing the capacity factor for each A_i and δ_i, the frequencies of failure were determined using Equation 2-12 and normal distribution tables and assuming that the capacity is lognormally distributed.

In Figure 4-7, displacement vs peak ground acceleration is shown with displacements beyond 2 inches undefined. While there is a finite limit to the displacement, a definition of the displacement vs peak ground acceleration beyond the relationship shown in Figure 4-7 is not possible without extensive nonlinear analyses. Since the slope of the displacement vs acceleration function is nearly vertical after soil failure is indicated, it was assumed that displacements beyond the limits of Figure 5-7 were sufficient to result in 100% frequency of failure. The probability of occurrence of displacements greater than about 2 inches for a given peak ground acceleration is, consequently, a major contributor to the frequency of failure for a given ground acceleration. Frequency of failure is then, a strong function of displacements greater than about 2 inches and a much weaker function of seismic inertial effects combined with finite displacements less than 2 inches. In other words, for the assumptions made, the frequency of failure of the steam piping is dominated by the probability that the displacement will exceed the values shown in Figure 4-7, (about 2 inches) and is much less affected by variability in pipe strength or the seismic inertial load.

From the above conclusion based on the assumption that displacements greater than 2 inches will be large enough to fail the pipe, one could postulate that all interconnecting piping would have a fragility curve very similar to that shown in Figure 5-4 for the 34 inch diameter steam line. This is a conservative conclusion but, lacking a definition of displacement vs acceleration beyond the bounds shown in Figure 4-7 there is no other practical choice.

In the event that the displacement vs acceleration function of Figure 4-7 were to be extended via multiple time history nonlinear analyses, each interconnecting pipe could be evaluated individually accounting for the pipe size, material, temperature, normal loads, distance from the containment penetration to the first rigid vertical support, containment building uplift (δ_1) and probability of occurrence of δ_1. Treatment of individual interconnecting piping individually is, however, not warranted at this time without further definition of uplift vs peak ground acceleration beyond the description in Figure 4-7.