Nebraska Public Power District
Nebraska's Energy Leader

NLS2002120
September 27, 2002

U.S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, D.C. 20555-0001

Subject: Response to Request for Additional Information Related to Nebraska Public Power District's Seismic Reevaluation Proposed to Address Cooper Nuclear Station License Condition 2.C.(6)
Cooper Nuclear Station, NRC Docket No. 50-298, DPR-46

References:
1. Letter to David L. Wilson (Nebraska Public Power District) from U.S. Nuclear Regulatory Commission dated August 6, 2002, Request for Additional Information Related to Nebraska Public Power District's Seismic Reevaluation Proposed to Address Cooper Nuclear Station License Condition 2.C.(6) (TAC No. MB4654)

2. Letter to U.S. Nuclear Regulatory Commission (NLS2002014) from David L. Wilson (Nebraska Public Power District) dated February 26, 2002, License Condition 2.C.(6) Seismic Evaluation

3. Letter to U.S Nuclear Regulatory Commission (NLS2002073) from Michael T. Coyle (Nebraska Public Power District) dated June 9, 2002, Supplemental Information Related to License Condition 2.C.(6) Seismic Evaluation

Attachment 1 provides Nebraska Public Power District's (NPPD's) response to the Nuclear Regulatory Commission (NRC) Request for Additional Information (RAI) transmitted by letter dated August 6, 2002 (Reference 1). The RAI requests additional information regarding the Cooper Nuclear Station (CNS) Main Steam Isolation Valve Leakage Pathway seismic evaluation submitted in compliance with CNS License Condition 2.C.(6) (Reference 2), and supplemental information submitted in Reference 3. The attached responses also address follow-on issues discussed with the NRC staff during a teleconference held on September 5, 2002.

Attachment 2 identifies the commitments contained within this letter.

A001

Cooper Nuclear Station
P O Box 98 / Brownville, NE 68321-0098
***Telephone:** (402) 825-3811 / **Fax:** (402) 825-5211*
http //www nppd com

Should you have any questions concerning this matter, please contact Mr. Paul Fleming at (402) 825-2774.

Sincerely,

David L. Wilson
Vice President- Nuclear

/wrv

Attachment

cc: Regional Administrator w/attachment
USNRC - Region IV

Senior Project Manager w/attachment
USNRC - NRR Project Directorate IV-1

Senior Resident Inspector w/attachment
USNRC

NPG Distribution w/o attachment

Records w/attachment

STATE OF NEBRASKA)
)
NEMAHA COUNTY)

David L. Wilson, being first duly sworn, deposes and says that he is an authorized representative of the Nebraska Public Power District, a public corporation and political subdivision of the State of Nebraska; that he is duly authorized to submit this correspondence on behalf of Nebraska Public Power District; and that the statements contained herein are true to the best of his knowledge and belief.

David L. Wilson

David L. Wilson

Subscribed in my presence and sworn to before me this 27th day of September, 2002.

GENERAL NOTARY - State of Nebraska
LUANN BRAY
My Comm. Exp. May 11, 2006

LuAnn Bray

NOTARY PUBLIC

ATTACHMENT 1

The following discussion provides the responses to the questions contained in the NRC Staff's Request for Additional Information (RAI) sent to the Nebraska Public Power District (NPPD) by letter dated August 6, 2002 (Reference 1). Note that the reference citations in the following questions correspond with those references included in the NRC's RAI. All references cited in the responses correspond with those listed at the end of this attachment. Where appropriate, reference citations have been inserted in the questions for clarification; these are identified by bracketed non-italicized text.

Question 1: *In your submittal (Reference 1), you indicated that the 2.0×SSE [safe shutdown earthquake] ground response spectrum (GRS) envelopes the floor response spectra (FRS) at elevation 932'-6" in both the Control Building (CB) and Reactor Building. However, Figure 4.5 shows that the FRS at elevation 932'-6" in the CB is higher than the 2.0×SSE GRS. Explain the discrepancy. Also, provide a figure, which confirms that the 2.0×SSE GRS envelopes the FRS at elevation 903'-6" in the CB.*

Response: As discussed in the June 9, 2002 letter (Reference 3), NPPD has developed a Turbine Building (TB) FRS that replaces NPPD's use of the more conservative 2.0xSSE GRS as input to the Cooper Nuclear Station (CNS) Main Steam Isolation Valve (MSIV) Leakage Pathway seismic evaluation.

Notwithstanding this change, Figure 4.5 from EE 01-147, Rev. 0, submitted previously with NPPD's February 26, 2002 letter (Reference 2), has been revised to show the FRS for CB 903'-6" and is provided as Figure 1 to this response. Figure 1 shows that 2.0×SSE GRS completely envelops the FRS for CB 903'-6".

As noted in the Question 1, 2.0×SSE GRS does not completely envelop the FRS for CB 932'-6". However, 2.0×SSE GRS was judged to be an adequate estimate of the FRS at Turbine Building (TB) 932' because:

- Figure 1 shows that the points where the CB 932'-6" FRS is higher than 2.0×SSE GRS are relatively small, and occur in the low frequency regions, below about 2 Hz,
- Figure 2 shows that 2.0×SSE GRS is a conservative envelope of the RB 931'-6" FRS, and
- The CB and RB were developed using the Individual Plant Examination for External Events (IPEEE) Review Level Earthquake (RLE) which, as shown in Figure 3, has substantially more low frequency content than either the Cooper Nuclear Station (CNS) SSE or the Regulatory Guide 1.60 GRS. Even so, these conservatively calculated FRS are enveloped by the 2.0×SSE GRS in all but the low frequency region.

As discussed in Reference 3, NPPD has developed new Turbine Building-specific FRS to replace the more conservative use of 2.0×SSE GRS. The Turbine Building-specific FRS has been calculated following the guidance in NUREG-0800 (Standard Review Plan) Sections 3.7.1 and 3.7.2, using the Soil-Structure Interaction analysis method and the Regulatory Guide 1.60 ground response spectrum anchored to the CNS SSE peak ground acceleration of 0.2g. As requested by the NRC Staff in a September 5, 2002 teleconference, the inputs, assumptions, and methodology used in this analysis have been provided in this response (see Appendix). This analysis has been completed and is undergoing NPPD review and approval.

Question 2: *You indicated that the methodology described in NUREG/CR-6240 (Reference 2) was used to determine the seismic capacity of welded and non-welded (e.g., threaded pipe) steel piping. Indicate whether NRC has reviewed and accepted the methodology as an acceptable approach to determine the seismic capacity of the steel piping.*

Response: EE 01-147 (Enclosure to Reference 2) will be revised to remove reference to NUREG/CR-6240, and credit and reference BWROG Report NEDC-31858P, Appendix D for evaluation of welded and threaded steel piping, and the associated NRC Safety Evaluation (SE), which accepted that approach.

The criteria (capacity and allowable spans) established in NEDC-31858P, Appendix D and subsequent BWROG submittals are based primarily on welded steel pipe. The experience data contains examples of non-welded fittings (threaded, friction, etc.) and non-ductile materials (cast iron) which have not performed as well as welded steel pipe in strong motion earthquakes. To address these situations in the experienced based evaluation of the piping in the CNS MSIV Leakage Pathway, a one-third (1/3) reduction in the allowable unsupported spans was applied to piping systems containing threaded fittings. Systems containing friction fittings or cast iron components would be classified as outliers and require a more detailed review and evaluation.

During a September 5, 2002 teleconference, the NRC Staff requested further explanation of the basis for using this 1/3 reduction in the piping allowable spans for piping systems containing threaded fittings. This ratio is based on a review and assessment of experience data and was used in the MSIV Leakage Pathway qualification effort. The NRC reviewed and accepted this 1/3 reduction as documented in Paragraph 5 of Section 4.5 of the NRC's SE for the Monticello MSIV Leakage Pathway submittal. This previous NRC acceptance was the primary basis for the selection of the one-third (1/3) reduction in the allowable unsupported spans for piping systems containing threaded fittings.

Question 3: *You indicated in Reference 1 that the seismic demand for outlier resolution will be 2 times the GRS in the horizontal direction and 2/3 the GRS in the vertical direction for all piping systems. The 2/3 the GRS in the vertical direction is based on an* *assumption that there is no amplification of the vertical seismic input ground motion by the Turbine Building (TB). Justify the TB is perfectly rigid in the vertical direction.*

Response: EE 01-147 (Enclosure to Reference 2) indicated that 2/3 the GRS will be used as the vertical seismic demand. The basis for this position was:

- This was consistent with the vertical seismic demand stated in the CNS USAR for all Class I structures (including the Control and Reactor Buildings), and
- The Turbine Building is a substantial reinforced concrete shear wall structure from the foundation mat (El. 877') to the operating deck (El. 932'), and it was reasonable to assume that it would not amplify the vertical ground motion.

As stated in Reference 3 and in the response to Question 1 above, multiples of the GRS are no longer being used as an estimate of the Turbine Building FRS. Instead, Turbine Building specific FRS have been calculated following the guidance in NUREG-0800 (Standard Review Plan) Sections 3.7.1 and 3.7.2. In addition, the preliminary Turbine Building specific analyses show the first vertical structural mode of the building to be in excess of 20 Hz, which is above the frequency regions in which significant amplification of the seismic ground motion occurs. These preliminary results support the previous use of 2/3 of the GRS in the vertical direction.

Question 4: *You indicated in Reference 1* [EE 01-147] *that the anchor bolt capacities of Appendix C of the Seismic Qualification Utility Group-Generic Implementation Procedure (SQUG-GIP) (Reference 3) will be used for the pipe support evaluations. However, if anchor bolts exist that are not given in the SQUG-GIP, then the manufacturer's capacities will be used with a factor of safety 3.0. Discuss your justification for not using the manufacturer's recommended factor of safety.*

Response: NPPD has revised its criteria for evaluation of existing concrete anchors. No "non-GIP" concrete expansion anchors (CEAs) were encountered in evaluating the pipe supports at CNS. Most of the CEAs are Phillips "Self-Drilling" expansion anchors; a few Phillips "wedge" anchors were also encountered. Both of these CEAs are specifically addressed by the SQUG-GIP. Accordingly, NPPD will use the SQUG-GIP, Appendix C allowable capacities for CEAs.

Some of the pipe supports (particularly the larger Main Steam Line supports) are anchored using the original cast-in-place Richmond Inserts, or a combination of Richmond Inserts and CEAs. SQUG-GIP addresses certain cast-in-place anchors; however, none of those evaluated were similar to Richmond Inserts. NPPD evaluated the Richmond Inserts using the capacity values specified in CNS procedures, which were developed using the manufacturer's recommended factor of safety. NPPD will revise EE 01-147 to reflect the above positions.

Question 5: *You used Equation 5.9 in Reference 1* [EE 01-147] *for determining the adequacy of the anchor bolt capacity. Discuss how Equation 5.9 is more conservative than the bilinear formulation given in the SQUG-GIP (Reference 3).*

Response: NPPD has revised its methodology for evaluation of anchor bolt capacity. NPPD is now using the Bilinear Relationship employed in the SQUG GIP. NPPD will revise EE 01-147 to reflect this change.

The interaction curves for the Bilinear Relationship and the Power Law Relationship (EE 01-147, Equation 5.9) are shown in Figure 4 for comparative purposes. When the shear ratio (V/V_{all}) is less than about 0.4, the Power Law relationship, with a = b = 5/3 is more conservative than the Bilinear Relationship. When the shear ratio is greater than about 0.4, the Bilinear Relationship is more conservative. The CNS MSIV Leakage Pathway support anchor bolts were determined to be acceptable using both methods. However, NPPD will revise its methodology to use the Bilinear Relationship that has already been reviewed and approved by the NRC as part of the SQUG GIP, and update EE 01-147 accordingly.

Question 6: *Equations 5.1a through 5.3 in Reference 1* [EE 01-147] *are similar to the equations contained in the ASME Boiler and Pressure Vessel Code, Section III, Division 1 for Class 3 piping systems. If the ASME type equations are used for a piping evaluation, then the appropriate i factor (stress intensification factor) from the version of ASME Code where those equations appear should be used in the evaluation.*

Response: See the response to Question 7.

Question 7: *You stated that the basis for the establishment of Equation 5.3 in Reference 1* [EE 01-147] *is that "... S_A for carbon steel pipe is approximately 1.5 S which is approximately 5/8 S_y. The majority of the piping is A-106B GR. B CS with S = 15000 psi and S_y = 36000* [sic] *psi. 2.5 S_A = (2.5 × 1.5 × 15000) = 56250 psi and, therefore, 2.5 S_A is approximately 1.6 S_y. The applied stresses are secondary; limiting the range of applied stress to less than 2 S_y insures that elastic shakedown will occur, no significant membrane stress rupture will occur, and the accumulated cyclic damage will be elastic. Therefore, given the limited number of cycles of strong motion in a Design Basis SSE (10 to 20 cycles) and that elastic cycling below the 2.0 S_y will occur, a fatigue failure due to the SAM's from one SSE would not occur. Therefore, the 1.6 S_y secondary stress range limit used is significantly less than the upper bound limit of 2 S_y and with this limit no fatigue failures due to one SSE event would be anticipated."*

However, the NRC staff has a different view on Equation 5.3. Equation 5.3 specifies the use of ½ the range of SSE anchor moments. This justification implies that the range of anchor motions is held to less than 2 S_y. Your statement is not accurate *unless Equation 5.3 considers the full range of SSE. Provide your discussion with respect to the staff's view.*

Response: The reference to ASME Boiler and Pressure Vessel Code, Section III, Division 1 was made to provide a technical basis for the 2.4 S limit being used. The reference to ASME Section III will be deleted from this discussion in EE 01-147. The revised basis for Equation 5.1 and 5.2 is as follows:

Equation 5.1a and 5.1b are the standard deadweight, thermal, and pressure allowable stress equations per the B31.1 Power Piping Code. In equation 5.2, S is the allowable material stress per the B31.1 Power Piping Code, which is the lesser of 5/8 S_y (2/3 S_y in the later code editions) or $S_u/4$. The majority of the piping under review is A-106B carbon steel pipe, which has S = 15,000 psi, S_y = 35,000 psi, and S_u = 60,000 psi. Therefore equation 5.2 limits the pressure + deadweight + SSE seismic inertial stress combination to less than 1.03 S_y, which ensures elastic behavior. Further, it ensures the validity of the linear elastic analysis techniques used in the static and dynamic analyses that were conducted.

EE 01-147 will be revised to reflect the above discussion.

In addition to equation 5.3, for all dynamic and static analyses, the following equation was checked:

$$i[(2 \times M_{bsam)} / z] \leq 2.5S_A;$$

where i, M_{bsam}, z, S_A are as defined in EE 01-147

This was done to address the case where the amplitude (½ range) of seismic anchor motion (SAM) stress is larger than the thermal stress. This equation (in conjunction with equation 5.3) ensures that the worst secondary stress range is limited to approximately 1.6 S_y, which is less than 2.0 S_y, ensuring that elastic shakedown will occur. Therefore, given the limited number of strong motion SSE cycles (10 cycles to 20 cycles), there will be no fatigue failures due to one SSE event. This equation and the above discussion will be added to EE 01-147, with the following basis.

In equation 5.3, S_A for carbon steel pipe is approximately 1.5 S, which is approximately 22,500 psi, and, therefore, 2.5 S_A is approximately 1.6 S_y. These stresses are secondary in nature, if limited to less than 2.0 S_y (per ASME criteria document, "Criteria of the ASME Boiler and Pressure Vessel Code for Design by Analysis in Section III and Section VIII, Division 2"), ensures that elastic shakedown will occur, no significant membrane stress rupture will occur, and accumulated cyclic damage will be elastic. The 1.6 S_y limit used here is significantly less than the upper bound 2.0 S_y limit, and is an acceptable secondary stress limit.

Question 8: *You stated in Reference 1 that "...Recent criteria and studies including Regulatory Guideline 1.61 [Damping Values for Seismic Design of Nuclear Power Plants], the ASME Boiler and Pressure Vessel Code Section III, Division 1, Appendix N, and NUREG/CR-0098 specify levels of damping for the SSE analysis of piping systems. In all the aforementioned documents, the basis of the determination of damping values is primarily the stress level in the component, not the basis or methodology used for response spectrum generation. That is, once a response spectrum is selected, the specified damping is based on the response of the structure under analysis in terms of fabrication methods and member stress levels. Newmark and Hall in NUREG/CR-0098, specify damping values of 2% to 3% for piping stressed to no more than ½ S_y and 5% to 7% for piping stressed to approximately the yield point. The ASME Boiler and Pressure Vessel Code, Section III, Division 1, Appendix N, currently specifies 5% damping for the evaluation of the piping systems at both the Level B and Level D conditions. The Level D condition corresponds to the SSE event under evaluation here."*

The NRC staff does not agree with your statement. The basis for staff acceptance of 5 percent damping is the conservatism in the spectra generation. This position has been previously stated in the NRC endorsement of Code Case N-411 in Regulatory Guide 1.84 [Design and Fabrication Code Case Acceptability-ASME Section III Division 1].

Response: As stated in Reference 3, the median-centered estimate of the floor response spectra (2.0×SSE GRS) is no longer being utilized. Instead, specific floor response spectra have been calculated following the guidance in NUREG-0800 (Standard Review Plan) Sections 3.7.1 and 3.7.2, using a Regulatory Guide 1.60 spectral shape as the input ground response spectrum. Therefore, the new spectra being used are conservative. This, in addition to the reasons previously cited in EE 01-147, Rev. 0, justifies the use of 5% damping. However, it should also be noted that in Section 4.3.1 (a) of the NRC's SE for the Monticello MSIV Leakage Pathway, the staff accepted the use of 5% damping in conjunction with an approximate median-centered spectra for the seismic ruggedness evaluation of piping systems.

Question 9: *You indicated in Reference 1 that an approach called the "collapsed beam" approach is used for localized evaluation of piping systems. The NRC staff is not aware of the "collapsed beam" approach and did not endorse the approach previously. Justify the reasons why the "collapsed beam" approach is equivalent to or more conservative than the analysis methods discussed in Sections 3.9.1 and 3.9.2 of the NRC Standard Review Plan.*

Response: The terminology "Collapsed Load Method" means the use of classical beam theory with conservatively established spans and end conditions to conduct a static stress analysis of local portions of piping using manual methods. This method is the same as the Equivalent Static Load Method permitted in Section 3.9.2 of the NRC Standard Review Plan (SRP) (NUREG-0800). The terminology "Collapsed Beam Method" will be replaced with the terminology "Equivalent Static Load Method," in EE 01-147.

In a September 5, 2002 teleconference, the NRC Staff requested the basis for applying a factor less than 1.5 to the peak acceleration of the amplified floor response spectra. In applying the "Equivalent Static Load Method," SRP Section 3.7.2 recommends a factor of 1.5 be applied to the peak acceleration of the amplified floor response spectra, unless a lower factor can be justified. For the CNS MSIV Leakage Pathway, the equivalent static seismic ruggedness evaluations of piping systems utilized a factor of 1.0 applied to the peak acceleration of the amplified floor response spectra.

The use of this factor is based on the work conducted in references 7.17 and 7.22 of EE 01-147, Rev. 0, submitted to the NRC in Reference 2. These studies demonstrated that for equivalent static seismic analyses (when using a conservative amplified floor response spectra as was developed for the CNS evaluation), a factor of 1.0 applied to the peak of the two orthogonal horizontal FRS and the vertical FRS enveloped the results as predicted by the Response Spectra Modal Analysis Method for piping stresses and pipe support loads. In addition, this work demonstrated that factors lower than 1.0 could be justified for low frequency piping systems, similar to those systems found in the main steam drain lines at CNS. However, it was conservatively decided to limit the factor to a lower bound value of 1 for the CNS MSIV Leakage Pathway analyses.

These studies discussed above formed the basis for the Equivalent Static Load Method implemented in Appendix N, Article N-1225, of the ASME Boiler and Pressure Vessel Codes, Section III, Division 1 in the early 1990's. This method was reviewed extensively by the ASME Code Committee working groups and subgroups prior to its acceptance in the ASME Code. The use of the Equivalent Static Load Method with a factor less than 1.5 was also previously accepted by the NRC in Paragraph 3 of Section 4.6.1 of the NRC's SE for the Monticello MSIV Leakage Pathway submittal.

<u>Question 10</u>: *During the teleconference held on May 8, 2002, the licensee indicated that the piping support components at Cooper Nuclear Station (CNS) are designed in accordance with the requirements in MSS-SP-58, "Pipe Hangers and Supports - Materials, Design, and Manufacture." In Reference 1, the licensee indicated that the capacities of the piping support components for the Level D load case should not exceed 2.0 times the capacities specified in MSS-SP-58 based on the ASME Boiler and Pressure Vessel Code Case N-500-1. The NRC staff requests response to the following:*

<u>Question 10(a)</u> *The ASME Boiler and Pressure Vessel Code Case N-500-1 specified other requirements (e.g., materials, quality assurance program, etc.) in order to use 2.0 times the capacities specified in MSS-SP-58 for the Level D load case. Indicate whether the piping support components at CNS meet the pertinent requirements of the*

ASME Code that would permit an increase in the load capacity by a factor of 2.0 times at the load Level D.

Response: ASME Code Case N-500-1 was used as a technical basis for the use of Service Level D capacities for standard support components of 2 times the MSS-SP-58 capacities. The technical basis of this Code Case is independent of the material traceability requirements. The reference to ASME Code Case N-500-1 will be deleted, and the technical basis for the use of the factor of 2.0 will be directly provided in EE 01-147. The basis is as follows:

Section 4 of MSS-SP-58, (1967 edition)[1] requires that in establishing pipe support component capacities, the maximum stress shall not exceed the allowable stresses in Table 2 of MSS-SP-58. Furthermore, for materials not listed in Table 2, the maximum allowable stress shall be $S_u/5$. The materials in Table 2 include several grades of cast and malleable iron. These materials were screened out during the walkdown as "non-ductile" support components requiring special consideration and evaluation.

A review of the allowable stress (for temperatures up to 650 °F) for steel bars, plates, bolts, straps and castings in Table 2, shows that the minimum ratio of the yield stress to the allowable stress is 2.0. Therefore, using the rated loads for the MSS-SP-58 components maintains the working stresses in the component to no higher than ½ S_y. This is applicable for the minimum ratio of the yield stress to the Table 2 allowable stress. The average ratio for all materials is 0.4 S_y. Therefore, multiplying the MSS-SP-58 rated capacities by a factor of 2 results in support components that are an average of 0.8 S_y, and, in the limiting the case, support components that are at the material yield stress. Multiplying the MSS-SP-58 rated capacities by a factor of 2 maintains elastic behavior for the SSE event and no significant deformation of the supports. Further, it ensures the validity of the linear elastic analysis techniques used in the static and dynamic analyses that were conducted.

The approach of keeping the support component allowable stresses at or slightly below the material yield stresses is consistent with the criteria and capacities the NRC accepted in Section 4.6.2 of the SE for the Monticello MSIV Leakage Pathway submittal.

Question 10(b): *In Reference 4* [NPPD's June 9, 2002 submittal], *the licensee indicated that CNS Updated Safety Analysis Report specifies the use of 0.9 S_y as the stress limit for the piping support components for the Level D load case. This limit exceeds 2.0 times the capacities specified in MSS-SP-58. Provide justification for suggesting to use an even higher limit than those permitted in the ASME Boiler and Pressure Vessel Code Case N-500-1. Also, indicate whether NRC had reviewed and accepted your use of 0.9 S_y as a stress limit for the piping support components at CNS for the Level D load case.*

---

1. With respect to this Question, MSS-SP-58 (1967 edition) is consistent with the B31.1.0-1967 edition that is the code of record for CNS.

Response: NPPD's original submittal (Reference 2) specified a criterion of 2.0 × the MSS-SP-58 specified allowable stresses for the faulted condition. In Reference 3, NPPD documented its intention to pursue a course of action to establish higher capacities for some component standard supports based on the greater of either:

1. 0.9 S_y, or
2. 2.0 × the MSS-SP-58 specified allowable stresses for the faulted condition (with SSE loading).

As discussed in the response to Question 10 (a), the rated loads for the MSS-SP-58 components limit the working stresses in the component to no higher than ½ S_y (i.e., $S_{allowable} = 0.5\ S_y$). This value corresponds to the materials with the minimum ratio of the material yield stress to the Table 2 allowable stress (i.e., $S_y / S_{allowable} = 2$). The average ratio of the material yield stress to the Table 2 allowable stress for all materials is approximately 2.5 which equates to an average allowable stress of 0.4 S_y. Therefore, multiplying the MSS-SP-58 rated capacities by a factor of 2 results in support component members that are on average at about 0.8 S_y and in the limiting case support component members are at the material yield stress (1.0 S_y). For example, for Grade 55, A663-82 carbon steel (for rods & bars), 2 times the listed allowable stress is 0.996 S_y [(2 × 13.7) / 27.5] (Reference: MSS-SP-58, 1983 edition). Therefore, the use of an allowable stress capacity of 0.9 S_y for the detailed evaluation of component support items is consistent with the 0.8 S_y to 1.0 S_y capacity achieved with the use of 2.0 times the MSS-SP-58 rated loads.

The CNS USAR (as supported by design calculations) addresses the usage of 0.9 S_y for Class I pipe support structural members, Class I building structural steel, structural steel rebar used in CNS Class I concrete structures, Class I conduit and cable tray supports, etc. when subjected to the SSE load cases. The USAR does not explicitly address the stress limit for the catalog component standard supports for the Level D load case.

Accordingly, NPPD's licensing basis, as approved by the NRC, for using 0.9 S_y for the specified SSCs is that no "loss of function" would occur for the SSE load cases. The criterion for "no loss of function" is that stresses remain in the elastic range.

Some of the applicable CNS catalog component standard supports are attached to and supported directly by a structural steel pipe support structure that is governed by an allowable stress limit of 0.9 S_y. Therefore, this criterion was also applied to the catalog component standard support.

The use of 0.9 S_y is consistent with previously approved criteria at CNS and provides sufficient margin against failure of the applicable pipe supports. As stated in its June 9, 2002 letter (Reference 3), and as further supported by the preceding discussion, NPPD requests explicit NRC approval of the above criteria for the pipe supports within the scope of the MSIV Leakage Pathway seismic qualification project. This approach meets the intent of the NRC's "Safety Evaluation of GE Topical Report, NEDC-31858P, Revision 2, 'BWROG Report for Increasing MSIV Leakage Limits and Elimination of Leakage Control Systems,' September 1993" to "provide reasonable assurance that the main steam line...will maintain structural integrity and operability during and following an SSE." This criterion also ensures "no loss of function" as previously defined. The approach of keeping the support component allowable stresses at or slightly below the material yield stresses is also consistent with the criteria and capacities the NRC accepted in Section 4.6.2 of the SE for the Monticello MSIV Leakage Pathway submittal.

Question 11: *In Reference 4* [NPPD's June 9, 2002 submittal], *the licensee indicated that a numerical technique (i.e., finite element analysis) will be used to establish the capacities of the pipe support components. Discuss your rationale for concluding that a finite element analysis, which relies on approximation of the geometry, can be considered to provide a more realistic estimate of the load carrying capacity of the analyzed component than the actual testing performed by the vendor for such component.*

Response: The use of finite element analysis (FEA) to establish higher capacities of pipe support components was proposed in NPPD's June 9, 2002 submittal (Reference 3). This methodology was proposed to reduce the number of pipe support modifications that had been indicated by the analyses completed at that time. MSS-SP-58 (1967 edition) requires that support components be load rated such that the allowable material stress remains less than the allowable stresses given in Section 4 of MSS-SP-58. There is no explicit or implied requirement in MSS-SP-58 that these load ratings be established by test. They can be, and in many cases are, established by calculation or analysis. The use of a FEA analysis is simply a more refined method of analysis that can be used to establish a support component load rating consistent with MSS-SP-58. However, the development of the Turbine Building specific floor response spectra coupled with reductions in other conservative pipe system modeling assumptions has rendered the proposed use of FEA for support component qualification unnecessary. Use of this methodology is not required and will not be used.

Question 12: *In Reference 4* [NPPD's June 9, 2002 submittal], *the licensee indicated that it will use the concrete anchor bolt capacities used in IE Bulletin 79-02 [Pipe Support Base Plate Designs Using Concrete Expansion Anchor Bolts] with a factor of safety of 4. However, IE Bulletin 79-02 requires a factor of safety larger than 4 for certain types of anchor bolts. Provide your technical justification for using only the factor of safety of 4.*

Response: As discussed in the response to Question 4 above, NPPD has revised its methodology for evaluation of existing concrete anchor bolts. NPPD is using the SQUG-GIP,

Appendix C capacities for existing concrete expansion anchors (CEAs), and existing CNS procedures and the manufacturer's recommended factor of safety for evaluating existing cast-in-place anchors. NPPD will revise EE 01-147 to reflect this position.

In its June 9, 2002 submittal (Reference 3), NPPD had intended to use, as an alternate evaluation methodology, the anchor bolt capacities identified in its response to IEB 79-02, adjusted for a concrete compressive strength of 5,000 psi based on actual compression test results, with a factor of safety of 4. While it is NPPD's position that use of this criteria is technically justified, NPPD will evaluate existing concrete anchors as specified in the response to Question 4 above.

REFERENCES

1. Letter to David L. Wilson (Nebraska Public Power District) from U.S. Nuclear Regulatory Commission dated August 6, 2002, Request for Additional Information Related to Nebraska Public Power District's Seismic Reevaluation Proposed to Address Cooper Nuclear Station License Condition 2.C.(6) (TAC No. MB4654).

2. Letter to U.S. Nuclear Regulatory Commission (NLS2002014) from David L. Wilson (Nebraska Public Power District) dated February 26, 2002, License Condition 2.C.(6) Seismic Evaluation.

3. Letter to U.S Nuclear Regulatory Commission (NLS2002073) from Michael T. Coyle (Nebraska Public Power District) dated June 9, 2002, Supplemental Information Related to License Condition 2.C.(6) Seismic Evaluation.

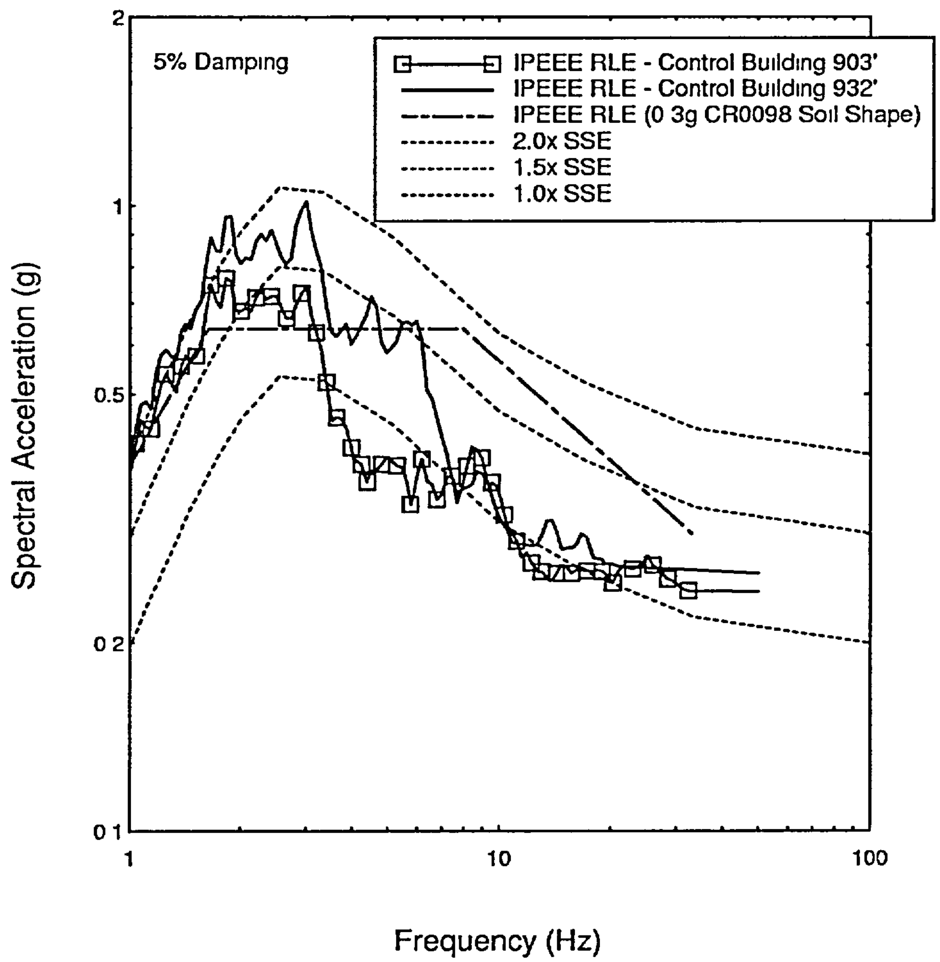

Figure 1. **IPEEE RLE Floor Response Spectra for the Control Building**

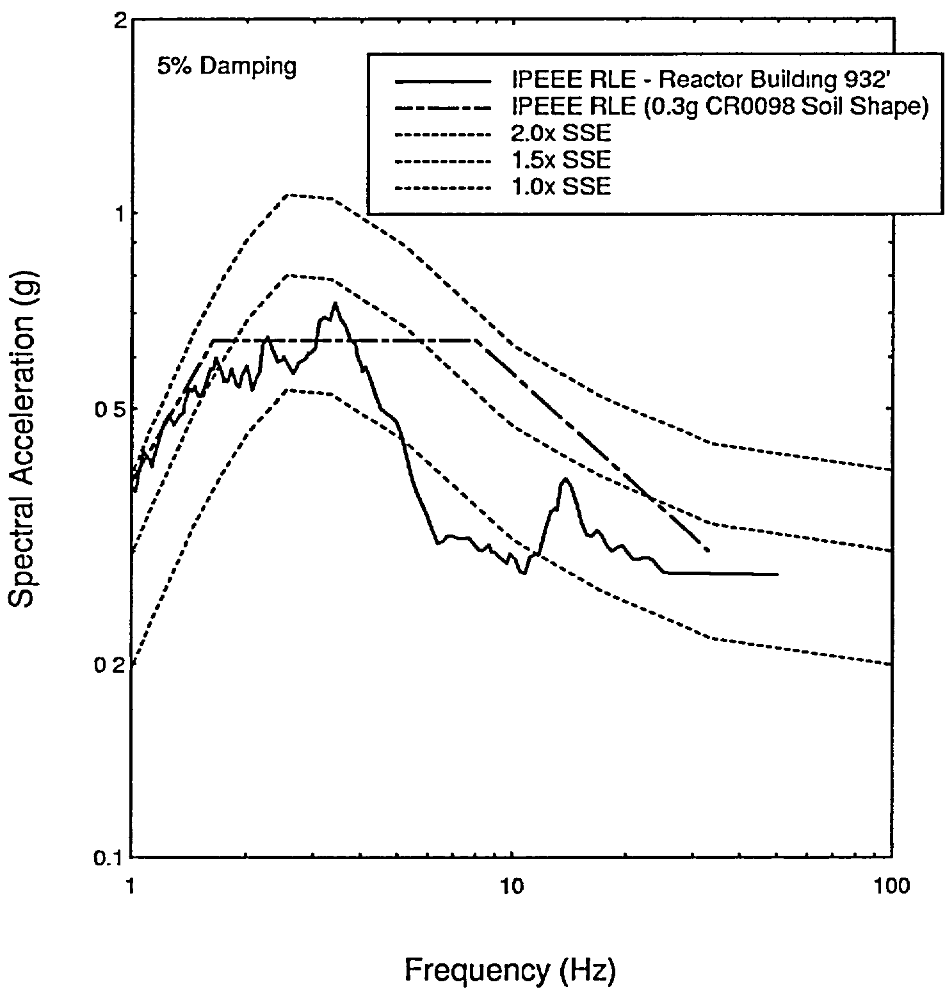

Figure 2. **IPEEE RLE Floor Response Spectra for the Reactor Building**

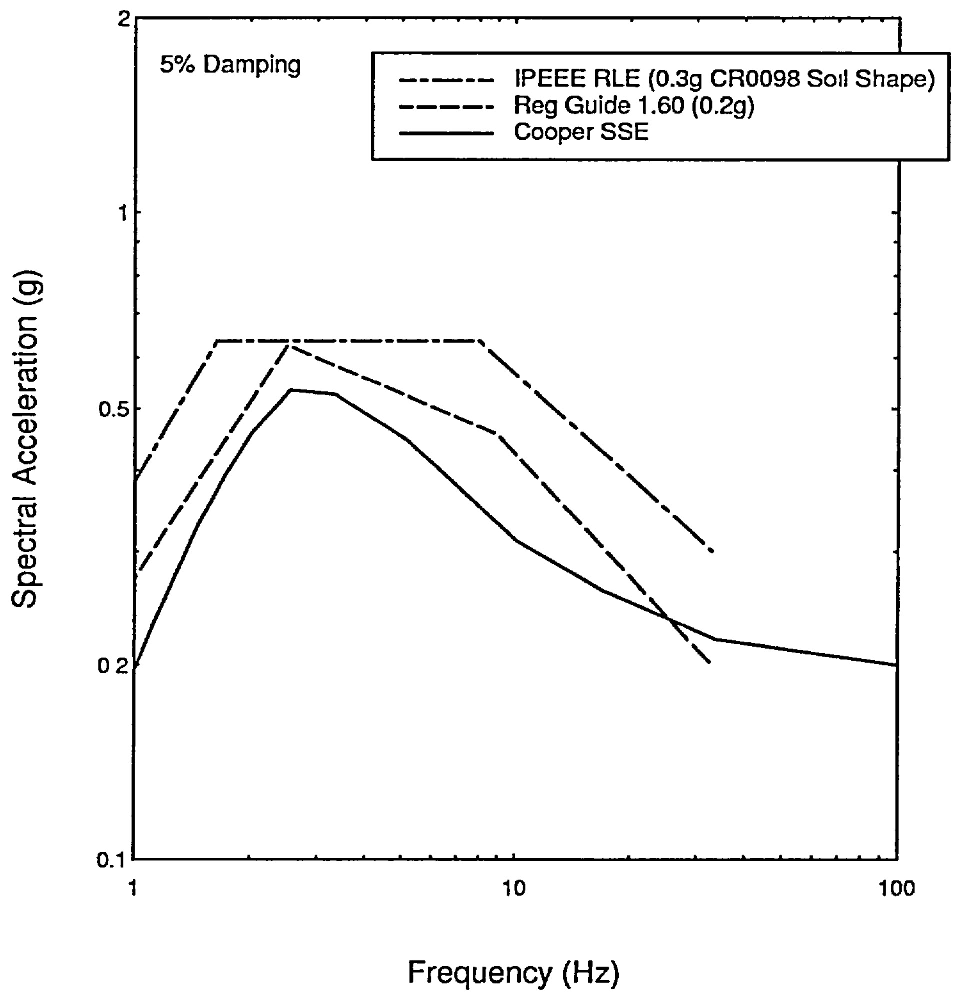

Figure 3. Comparison of Ground Response Spectra

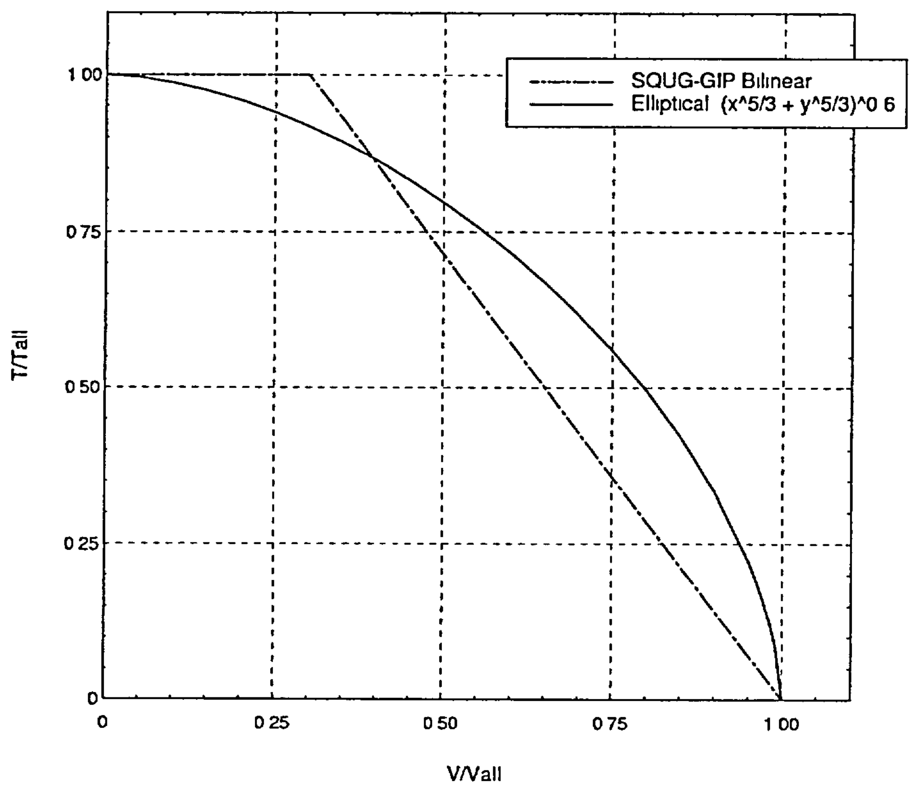

Figure 4. **Comparison of Shear (V) / Tension (T) Interaction Curves**

APPENDIX

Stevenson & Associates
Calculation 02Q4295-C-001
Design Inputs and Methodology

4. Design Inputs

4.1 Materials

Concrete

Normal density = 0.150 kcf [3].

High density = 0.300 kcf [2]. Used for the walls on turbine building 932'-6" designated as high density on drawing 4066 [44].

Compressive strength
Turbine Building:	f_c' = 5000 psi [21]
Reactor Building (excluding the drywell pedestal):	f_c' = 5000 psi [14, Appendix C, Section 2.3]
Reactor Building, drywell pedestal:	f_c' = 2000 psi [18]

Young's Modulus: $E = 57000(fc')^{0.5}$ [1]
Turbine Building:	E = 4.03E6 psi = 5.80E5 ksf
Reactor Building (excluding the drywell pedestal):	E = 4.03E6 psi = 5.80E5 ksf
Reactor Building, drywell pedestal:	E = 2.55E6 psi = 3.67E5 ksf

Poisson's Ratio: v = 0.2 [4]

Shear Modulus: G = E / 2(1 + v)
Turbine Building:	G = 1.68E6 psi = 2.42E5 ksf
Reactor Building (excluding the drywell pedestal):	G = 1.68E6 psi = 2.42E5 ksf
Reactor Building, drywell pedestal:	G = 1.06E6 psi = 1.53E5 ksf

Damping = 7% [12]

Steel

Density = 0.490 kcf [5].

Young's Modulus: E = 29E6 psi [5] = 4.18E6 ksf

Poisson's Ratio: v = 0.25 [5]

Shear Modulus: G = E / 2(1 + v) = 1.16E7 psi = 1.67E6 ksf

Damping = 4% [12]

Soil

Per Section XII.2.4.4 of the USAR [14], Contract E67-38 [16], the boring drawings [22, 23], and the excavation and backfill drawings [24-27], the site was excavated to within 9' of bedrock, and the remaining in-situ soil (which is predominately sand) was compacted to a relative density of 85%. A sand or sand/gravel fill was then placed in lifts of no more than 12" in height and compacted to an average relative density of 85% . The plant grade elevation (top of fill) is 902'-6".

The groundwater elevation is established at 885'-0" based on the information in the original soils report [15]. During normal conditions, the groundwater elevation on the landside of the levee is no more than 3' below the normal river elevation of 880'-0". During rapidly rising river levels, the groundwater elevation on the landside of the levee is 3' to 10' lower than the river level. Per Section 4.2.2 of the USAR [14], the maximum river level is approximately 900'-0". For this calculation, the groundwater elevation is established as the average of the normal river level and 10' below the maximum river level: (880' + 900' - 10') /2 = 885'.

The specific soil parameters are as follows:

Soil Type:	Sand, sand/gravel mix [16]
Soil Relative Density:	85% [16]
Soil Weight:	116 pcf dry [6, Table 3.2, 85% of maximum density] 134 pcf saturated [17] 71.6 pcf submerged [17]
Soil Poisson's Ratio:	0.30 dry [7, Table 4.1] 0.47 submerged (controlled by water, set close to the theoretical limit of 0.5)
Soil Coefficient of Pressure at Rest (K_0):	0.38 [17]
Bedrock Weight:	150 pcf [6, Table 3.1, specific gravity of 2.4]
Bedrock Shear Modulus:	20,000 ksf [17]
Bedrock Poisson's Ratio:	0.30 [17]
Grade Elevation:	902'-6" [24-27]
Groundwater Elevation:	885'-0" (see discussion above)
Bedrock Elevation:	822'-0" [22, 23]

The low strain soil shear modulus is calculated based on the following equation from Seed & Idriss [8]:

$$G_{soil} = 1000(k_2)(\sigma_m')^{1/2}, \text{ where}$$

k_2 = an empirical constant based on the soil type and relative density. A value of 67 was used based on sand at a relative density of 85%.

σ_m' = effective mean pressure. Calculated, as a function of elevation, based on the weight of the soil above. Saturation effects are included based on a water table elevation of 885'.

In order to calculate the high strain soil properties (i.e., the soil properties during the seismic event), the soil shear stiffness and damping as function of shear strain is required. These data are obtained from Figure 5.10 in Reference 9, which is reproduced here in Figure 1. The curves labeled Geomatrix 1990, 50' < H <150' are used.

4.2 Turbine Building Structure and Major Equipment

The weight and stiffness of the Turbine Building is calculated based on the information contained in the structural drawings listed as References 28 through 59.

Major equipment weights are obtained from the drawings listed as References 60 to 68, These are summarized as follows:

- Turbine-Generator: 4,460 kips distributed along the top of pedestal as shown in Reference 61. As stated in Reference 60, this value includes only the weight of the turbine, generator and associated equipment, and does not include any forces exerted by the condenser or the exhaust connections. These values are consistent with those used in the original design calculations [19, Sheet 13].
- Condenser: 3,139 kips, both shells, operating [53]
- Moisture Separator: 108 kips, per separator, operating [62]
- Feedwater Heaters: Varies, 82 kips -> 233 kips per heater, full [64 - 68]

4.3 Reactor Building Structural Model

The reactor building structural model is based on the original design basis model from Reference 18. The model is shown in Figure 2, and the model parameters are listed in Table 1. The following changes are made to the model:

- The soil springs (K_l, K_ϕ) are removed. These are replaced by the soil impedance functions (see Section 6.7)
- The node representing the torus (node 10 in Figure 1) is removed. The mass of the torus (280 k-s^2/ft based on the weight tabulation contained in the reference) is added to the node 9 mass of 1562 k-s^2/ft, resulting in a total mass at node 9 of 1842 k-s^2/ft. The original mass moment of inertia for node 9 (3.05E6 k-s^2-ft) is re-calculated by transforming the original mass moment or inertia at node 10 (3.22E5 k-s^2-ft) from elevation 871'-7" to 854'-9":

  3.05E6 + 3 22E5 + 280 * (871.58 - 854.75)^2 = 3.45E6 k-s^2-ft

- In order to capture the vertical response due to rocking, four nodes are added to each elevation in the model. The four nodes are located at the extreme corners of each elevation and are connected to the central (mass) node at that elevation by rigid links.
- The concrete stiffness (E, G) in the original model was based on concrete compressive strengths of 4000 psi for the exterior walls below grade (nodes 6 to 8 and 8 to 9 in the model), 3000 psi for all walls above grade (nodes 1 to 6), and 2000 psi for the drywell pedestal (nodes 6 to 7 and 7 to 9). As specified in Section 4.1, for this calculation the drywell pedestal stiffness is based on a concrete strength of 2000 psi, and the concrete stiffness in the rest of the reactor building is based on a concrete strength of 5000 psi.

4.4 Ground Response Spectra

The horizontal and vertical ground response spectra are the spectral shapes specified in Regulatory Guide 1.60 [11], anchored to the "hypothetical maximum design earthquake" peak ground acceleration of 0.2g specified in Section 5.2.3 of the USAR [14]. These spectra are shown in Figure 3. The control point values are listed below.

Horizontal (5% damping)		Vertical (5% damping)	
Frequency (Hz.)	Acceleration (g)	Frequency (Hz.)	Acceleration (g)
0.25	0.094	0.25	0.063
2.50	0.626	3.50	0.596
9.00	0.522	9.00	0.522
33.0	0.200	33.0	0.200

Note that the 5% damping values are used. These spectra are used to generate artificial time histories for use in the calculation. Per paragraph I.1.b of SRP Section 3.7.1 [10]:

> *"The response spectra obtained from such an artificial time history of motion should generally envelop the design response spectra for all damping values to be used."*

In a soil-structure interaction analysis the damping used is a combination of the structural damping (4% for steel and 7% for concrete in this analysis) and the damping inherent in the soil impedance functions, which varies with frequency. As the "actual" damping value is difficult to ascertain, 5% damping was arbitrarily selected to develop the artificial time histories and perform the required enveloping checks.

5. Methodology

The methodology consists of the steps outlined below. The detailed calculations, organized according to these steps, are provided in Section 6. The coordinate system used in the calculations is X=NS, Y=EW, Z=Vertical.

1. For the turbine building, the weight parameters of each elevation and the inter-elevation stiffness parameters are calculated. The elevations are·

   877'-6" (Basement)
   903'-6" (Mezzanine, including the heater bay elevation at 909'-6")
   932'-6" (Operating Deck)
   1000' (Roof)

   Weight parameters include the mass, center of mass location, and the mass moments of inertia about all three axes. Stiffness parameters include the translational stiffnesses in all three directions, the center of stiffness, and the rotational stiffnesses around all three axes.

2. For the turbine pedestal, a structural model is constructed and the modal properties for the first three modes are calculated. These modes represent the translational response in the X (NS) and Y (EW) directions, and the rotational response about the Z (vertical) axis. Based on the modal frequencies and the calculated weight distribution, a stick model of the pedestal is constructed consisting of a single elevation representing the operating deck at 932'-6" anchored to the foundation mat.

3. Based on the results of the previous two steps, a "two-stick" structural model of the turbine building and turbine pedestal is constructed, and the modal properties are calculated. Each elevation in the model includes a node representing the center of stiffness, a node representing the center of mass, and up to four additional nodes representing the limits of the main steam piping at that elevation.

4. Fixed-base modal properties are calculated for the reactor building using the structural model described in Section 4.3.

5. Time histories are developed corresponding to the free-field (Reg Guide 1.60) ground response spectra. The requirements of SRP Sections 3.7.1 and 3.7.2 [10] are verified.

6. The best estimate low strain soil properties are calculated. The corresponding best estimate high strain soil properties are then calculated based on the soil property/strain curves shown in Figure 1. Per the requirements of SRP Section 3.7.2 [10], upper bound and lower bound high strain soil properties are calculated by doubling and halving the low strain soil properties and re-calculating the high strain properties. The SRP requirements on minimum upper bound high strain soil properties and maximum soil damping values are verified.

7. Three sets of soil impedance functions are calculated for each of the turbine building and the reactor building using the best estimate, lower bound and upper bound high strain soil properties.

8. Three sets of base input time histories are calculated for each of the turbine building and the reactor building using the best estimate, lower bound and upper bound high strain soil properties.

9. The "two-stick" turbine building structural model and the soil impedance functions are combined to form the soil-structure model. The model is excited using the base input time histories, and response time histories are calculated at the various nodes. This calculation is performed three times, using the best estimate, low bound, and high bound soil impedances and base input time histories. The resulting nodal time histories are converted to response spectra, and, for each

elevation and direction in the turbine model, the response spectra for all three soil estimates and all nodes are enveloped to produce three response spectra (EW, NS and Vertical) for each elevation.

10. Step 9 is repeated for the reactor building. Response spectra are calculated only for elevations 903'-6" and 931'-6".

11. The relative displacements between turbine building elevation 932'-6" and reactor building elevation 931'-6" are calculated. The relative displacements between turbine building elevation 932'-6" and turbine pedestal elevation 932'-6" are calculated.

ATTACHMENT 3 LIST OF REGULATORY COMMITMENTS

Correspondence Number: NLS2002120

The following table identifies those actions committed to by Nebraska Public Power District (NPPD) in this document. Any other actions discussed in the submittal represent intended or planned actions by NPPD. They are described for information only and are not regulatory commitments. Please notify the NL&S Manager at Cooper Nuclear Station of any questions regarding this document or any associated regulatory commitments.

COMMITMENT	COMMITTED DATE OR OUTAGE
EE 01-147 will be revised, as necessary, to reflect the information contained in this letter.	7/30/03