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7. Abstract Structures and equipment, which are qualified for the design basis earthquake (DBE) and have anchorage designed for the DBE loading, do not require an evaluation of the near-field earthquake (NFE) effects. However, safety class 1 acceleration sensitive equipment such as electrical relays must be evaluated for both NFE and DBE since they are known to malfunction when excited by high frequency seismic motions.		
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Evaluation of Near-Field Earthquake Effects

August 1994

Prepared by: H. P. Shrivastava
H. P. Shrivastava, Principal Engineer
Structural Assessments

Date: 8/5/94

Reviewed by: J. T. Baxter
J. T. Baxter, Principal Engineer
Structural Assessments

Date: 8-8-94

Approved by: T. J. Conrads
T. J. Conrads, Manager
Structural Assessments

Date: 8/10/94

Westinghouse Hanford Company
for the U.S. Department of Energy
Richland Operations Office (RL)

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TYPICAL CHECKLIST FOR INDEPENDENT REVIEW

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Author H. P. Shrivastava

Yes No N/A

☒	☐	☐	Problem completely defined.
☒	☐	☐	Necessary assumptions explicitly stated and supported.
☐	☐	☒	Computer codes and data files documented.
☒	☐	☐	Data used in calculations explicitly stated in document.
☒	☐	☐	Data checked for consistency with original source information as applicable.
☒	☐	☐	Mathematical derivations checked including dimensional consistency of results.
☒	☐	☐	Models appropriate and used within range of validity or use outside range of established validity justified.
☐	☐	☒	Hand calculations checked for errors.
☐	☐	☒	Code run streams correct and consistent with analysis documentation.
☐	☐	☒	Code output consistent with input and with results reported in analysis documentation.
☒	☐	☐	Acceptability limits on analytical results applicable and supported. Limits checked against sources.
☒	☐	☐	Safety margins consistent with good engineering practices.
☒	☐	☐	Conclusions consistent with analytical results and applicable limits.
☒	☐	☐	Results and conclusions address all points required in the problem statement.

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J. T. Baxter
Reviewer

8-10-94
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EVALUATION OF NEAR-FIELD EARTHQUAKE EFFECTS

1.0 HANFORD PLANT STANDARDS EARTHQUAKE DESIGN REQUIREMENTS

The most recent revision to the Hanford Plant Standards Civil-Structural Design Criteria, SDC-4.1, Revision 11 (Reference 12), was issued September 5, 1989. Revision 11 included several major modifications, the most important of which was incorporation of new Department of Energy (DOE) natural hazards design and evaluation requirements from UCRL-15910 (Reference 13). This included new design basis loadings (earthquake and wind) based on Hanford site-specific hazards studies.

The revised earthquake design basis loadings included both a design basis earthquake (DBE) event and a new design requirement for a near-field earthquake (NFE). The NFE events are characterized in SDC-4.1 as a response spectra resulting from small magnitude, high frequency, low energy earthquakes close to the facilities on site. Currently all safety class 1 (SC1) equipment which must operate both during and after an earthquake must be qualified for both the DBE and NFE events. Questions have been raised regarding interpretation and use of the new NFE seismic design requirement by both Hanford area contractor engineering personnel and DOE subcontractors involved in the design of capital projects.

2.0 SUMMARY AND CONCLUSIONS

The purpose of this "white paper" or plan is to clarify the use of the NFE event in current and future Hanford Site facility seismic engineering designs and evaluations.

This paper reports the results of a literature search and subsequent evaluation to assess industry experience and select design analysis methods for addressing NFE events. Information sources used for this evaluation include both past and current seismic design research efforts in the commercial nuclear reactor field in the following areas:

- o Engineering characterization of seismic ground motion.
- o Industry studies to develop experience based seismic qualification methodologies.
- o Recent efforts to establish exceedance criteria for commercial nuclear facility "operating basis earthquakes (OBE)."

Analysis efforts for this report included review of recent sensitivity studies (Weiner, 1989) and the efforts reported herein to evaluate both damage and functionality thresholds for small near-field earthquakes.

Results of the investigation completed to date indicate that the use of the NFE event in seismic design analyses can be easily minimized, but not totally eliminated for the Hanford Site non-reactor nuclear facilities. Structures and equipment may in general be qualified for the DBE with anchorage designed for the DBE loading without further consideration of the NFE based on the discussion in Section 8.1. This conclusion, however, is not applicable to SC1 acceleration sensitive equipment and relays, contactors, motor starters and switches that require functional operability during a seismic event. For example, some electrical relays are known to chatter when excited by high frequency seismic motions. Thus, SC1 equipment in these categories will have to be seismically qualified for both DBE and NFE ground motions during design evaluations. The report prepared by Kennedy for the Westinghouse Hanford Company under purchase order MJG-SCV-267607 supports this conclusion (see Appendix A).

One acceptable method for seismic qualification of SC1 relays is the use of floor response spectra for the facility in question with the EPRI generic ruggedness spectral (GERS) database (applicable only to items contained in the database). The EPRI GERS data and seismic margin reports can be obtained through Dr. Bob Murray of the Lawrence Livermore National Laboratory. Dr. Murray has acquired all EPRI A46 and seismic margins reports for use by all DOE contractors.

3.0 BACKGROUND INFORMATION ON THE HANFORD SITE SEISMIC DESIGN BASIS

Reference 1 indicates that Woodward Clyde Consultants (WCC) performed a probabilistic seismic hazard study for the Hanford Site in 1989. This study was based upon the data and methodology similar to those used in the WNP-2 hazard assessment performed by WCC and, therefore, underestimated the earthquakes of magnitude 4.0 to 5.7 and overestimated the earthquakes of higher magnitude. A detailed study of the recorded seismic data was performed in Reference 2 using an improved recurrence model. The data and seismic criteria provided in Reference 1, which provides the technical basis for Revision 11 of SDC 4.1, are based on this revised seismic recurrence model.

The Hanford Site is within an area of primarily micro-earthquakes ($M=1.0$ to 3.0) and small-magnitude earthquakes ($M=3.0$ to 5.0). As the response spectra for the small-magnitude earthquakes are not represented by a broad band response spectra, Reference 1 provided a set of spectra for 100 and 200 Areas and a different set for the 300 and 400 Areas of the Hanford Site. The 100 and 200 Area spectra have higher peak acceleration as they are anchored to a higher peak ($0.2g$) horizontal free-field ground acceleration due to their proximity to the Umtanum-Gable Structure. Figures 1 and 2 show the 5 percent damping NFE spectra.

4.0 EQUIPMENT DAMAGE

From the seismic damage viewpoint, SCl components can be classified into the following two categories:

- i) Components which are ductile and can sustain small inelastic deformation without loss of safety function. Structural damage is of prime concern for this type of equipment.
- ii) Acceleration sensitive equipment which may malfunction due to high frequency motions.

Applicability of the NFE to design of these two categories of equipment is discussed below.

5.0 LITERATURE REVIEW - DAMAGE POTENTIAL OF EARTHQUAKES

5.1 STRUCTURAL DAMAGE

Reference 3 (NUREG 3805, Vol. 1, Kennedy et. al, NRC 1984) discusses and analyzes the engineering characterization of ground motions of several earthquakes including four small-magnitude earthquakes. It concludes that the ground motions defined in terms of instrumental peak accelerations do not provide a reliable measure of damage potential of earthquakes. Because structure or equipment damage is measured by its inelastic deformation, the earthquake damage potential depends upon the time duration of motion, number of strain cycles, energy absorption capacity of structure or equipment, and the energy content of the earthquake. Small-magnitude earthquakes with higher peak accelerations have been found to be less damaging than the lower peak acceleration long duration earthquakes. The study (Reference 3) clearly indicates that the earthquake duration and the energy content have significant effects on damage potential.

The small-magnitude earthquakes of 1978 and 1979 near Virgil C. Summer nuclear power plant (NPP) in South Carolina and of 1986 near the Perry NPP in Ohio caused the nuclear industry to formulate a criterion, documented in Reference 4 (EPRI Report NP-5930, Reed et. al, 1988), for determining OBE exceedance. Based on the nonlinear analyses of single-degree-of-freedom 7 percent-damped models subjected to 39 earthquake time histories, spectral acceleration and cumulative absolute velocity (CAV) were selected as the most appropriate parameters for earthquake damage prediction. A two-level criterion was formulated for determining when an earthquake ground motion has exceeded the OBE. Another 263 earthquake records were analyzed to verify the recommended OBE exceedance criteria given below :

1. Response Spectrum Check : OBE is exceeded if 5 percent damped ground response spectra exceeds OBE spectra between 2 and 10 hz or 0.2g whichever is greater. This provides a quick check of OBE exceedance.

A cutoff of response spectrum check is specified above at 10 hz. It is shown, in Reference 4, that the high frequency motions above 10 hz are non-damaging.

2. CAV Check : This parameter is the area under the absolute accelerogram. The OBE is exceeded if the CAV value for an earthquake record exceeds 0.3 g-sec. This CAV damage threshold being erroneously high has been lowered to 0.16 g-sec. in the draft NRC Regulatory Guide DG-1017 (Reference 11). CAV is defined as follows :

$$CAV = \int |a(t)| dt$$

where :

$a(t)$ = acceleration time history

t_{max} = duration of record

A correlation of damage data for buildings of "good design and construction" indicates that the above criterion provides a minimum safety margin of 1.5. The criterion was conservatively formulated to ensure that the ground motions greater than Modified Mercalli Intensity (MMI) VI are identified as exceeding the OBE.

A study (Reference 5) on lower-bound earthquake magnitude was completed under EPRI sponsorship in 1989. In this study, a total of 99 MMI VII and 13 MMI VIII with moment magnitudes less than 5.0 and 150 MMI VII and 17 MMI VIII events with moment magnitudes between 5.0 and 5.5 have been reviewed. Generally, damage to equipment is confined to intensities MMI VIII or greater. Reference 5 concludes that the earthquake magnitudes lower than M5.0 have a very small probability of causing MMI VIII damage and, therefore, need not be considered in the seismic hazard assessment of the nuclear power plants.

It may be noted that both Reference 4 and 5 address only the structural damage issue and both assume that the equipment is appropriately anchored. The earthquake induced relay chatter and operability of contactors, motor starters, and switches are not discussed in these studies.

5.2 FUNCTIONAL DAMAGE

In response to U.S. Nuclear Regulatory Commission unresolved safety issue (USI) A-46, the electric utilities formed the Seismic Qualification Utility Group (SQUG) to develop a technical basis for establishing seismic adequacy of certain classes of equipment without performing detailed seismic qualification. The study, sponsored by SQUG in 1982, provided guidelines for seismic qualification of eight classes of equipment on generic basis. The methodology was based upon earthquake experience data which indicated that the classes of equipment studied under the SQUG program were functional and undamaged after experiencing earthquakes. The SQUG methodology was later extended to nearly twenty equipment classes.

In 1984, EPRI sponsored research projects to support equipment seismic qualification efforts of the nuclear utilities. The EPRI programs developed generic ruggedness spectra (GERS) for many equipment classes (Reference 6), and the anchorage guidelines (Reference 7) which are based on the seismic qualification test data accumulated over years of nuclear power plant design. The GERS defines the maximum input motion at the base or support point for which a class of equipment has been demonstrated, by tests, to remain functional. These GERS are used by the utilities to evaluate the USI A-46 equipment. The application of this methodology requires an utility to prepare a list of safe shutdown equipment and to compare the site spectra with the bounding spectra applicable to different classes of equipment. A walkdown by a team of qualified engineers is required to evaluate the anchorage. The GERS for the following equipment classes have been developed by EPRI :

Electrical Equipment

- Batteries on Racks
- Battery Chargers
- Inverters
- Electrical Penetration Assemblies
- Distribution Panels
- Motor Control Centers
- Transformers
- Switches
- Transmitters
- Relays (several types)
- Switchgear
- Control Panels
- Instrument Rack Components
- Automatic Transfer Switches

Mechanical Equipment

- Motor Valve Operators
- Air-Operated Valves
- Solenoid-Operated Valves
- Safety Relief Valves

Almost all of the GERS reported in Reference 6 are for equipment functionality for both during and after an earthquake motion. It may be noted that a GERS for functionality does not assure structural integrity.

For a 5 percent spectral damping, the GERS for electrical equipment have spectral peaks between 1.5 to 10g, for mechanical equipment between 9 to 20g, and for many relays used in nuclear plants between 4.5 to 14g. The EPRI Report NP-5223 states (page S-3) "it is expected that these GERS would be sufficient to qualify almost all equipment within these classes in all plants except those with very high floor spectra".

In Appendix A, Kennedy has conservatively developed a NFE design spectrum to be used for NFE evaluation of structures and equipment. To obtain this spectra, he has used the 5 percent damping NFE spectra, for 100 and 200 Areas, given in SDC-4.1; an inelastic energy absorption correction factor; and high-frequency incoherency factors for the ground motion response spectra. The inelastic energy absorption correction factor is conservatively based upon an inelastic anchorage distortion of 0.01 inch, since all anchorage types including even the most brittle ones are capable of undergoing such a small distortion. The incoherency factors account for variation of the ground motion at different points on a large size basemat. This reduction factor is frequency dependent and varies from unity at 5 Hz to approximately 0.8 at 25 Hz and greater. The NFE design spectra, developed by Kennedy, exceeds the DBE spectra by only a small amount and over a narrow frequency range. This exceedance has no seismic safety consequences for structures and equipment as the inelastic distortion permitted is only 0.01 inch.

Acceleration sensitive equipment and relays, contactors, motor starters and switches may require an evaluation as they are known to malfunction at high frequency small magnitude motions.

6.0 SEISMICALLY QUALIFIED SAFETY CLASS 1 EQUIPMENT AT HANFORD

The DOE non-reactor nuclear facilities typically have a small inventory of SC1 components that are seismically qualified. These facilities usually have much smaller radionuclide inventories than operating reactor cores. In addition, the high energy sources needed to disperse the inventories are seldom available within the facility building envelopes.

Typical configurations for canyon type processing facilities and mill type industrial structures rely solely on the following elements:

- o passive building envelope,
- o HEPA filters and filter frames,
- o ducting from the building envelope to the HEPA filters, and
- o an elevated stack

for post-DBE isolation of the radionuclide inventory from the on-site workers and the off-site public. Many existing facilities do not have the documentation demonstrating compliance to current design rules. Current design rules also require seismically qualified stack monitoring equipment such as:

- o alarm CAMS,
- o isokinetic samplers,
- o CAM flowmeters,
- o associated support equipment, power, annunciators, wire, relays, etc.

7.0 COMPARISON OF HANFORD SITE DBE AND NFE SPECTRA

Figures 3 to 6 show the velocity, displacement, and energy plots for the DBE. Similar plots for the NFE are shown in Figures 7 to 10. These plots are taken from Reference 9. Some of the important parameters of these earthquakes are listed below :

Parameter	DBE	NFE
Max. Acceleration, 100 & 200 areas - 5 percent damping	0.42g	0.52g
Max. Acceleration, 300 & 400 areas - 5 percent damping	0.42g	0.39g
Max. Velocity, in/sec	13.017	2.36
Max. Displacement, in	7.048	0.29
Time Duration T_n , sec	17.5	1.0
Energy, in^2/sec^3	13239	858

Figure 11 shows the cumulative energy versus frequency plots for both DBE and NFE. Figure 12 shows the PSDs for both DBE and NFE. It is obvious from these plots that the energy content of NFE is significantly lower than the DBE for all frequencies of interest.

As shown in Figure 1, the peak (at 12.5 hz) of NFE spectra for 300 and 400 areas exceeds the DBE spectral acceleration at 12.5 hz by 0.05g (approximately 15 percent higher).

Figure 2 shows the DBE and NFE for the 100 and 200 Areas. The NFE spectra clearly has peak acceleration higher than the DBE peak acceleration. The NFE peak acceleration at 12.5 hz exceeds the DBE acceleration at this frequency by 0.19g (58 percent higher). It is not apparent what affect the NFE exceedance over the DBE spectral accelerations, in the high frequency range, would have on the equipment. However, a sensitivity study of small-magnitude earthquakes on buildings mainly in the 200 Area with an exception of the Fast Flux Test Facility, performed in Reference 10, indicates that the small-magnitude earthquakes may be important from the vertical motion viewpoint.

8.0 BASIS FOR EXEMPTING EQUIPMENT FROM NFE EVALUATION

8.1 DUCTILE EQUIPMENT

The structures and equipment excluding acceleration sensitive equipment such as electrical relays may be exempted from the NFE evaluation on the following basis :

- i) The NFE is a low energy earthquake. The total energy is approximately 6.5 percent of that for the DBE.
- ii) The NFE strong-motion duration is just over one second. In comparison, the DBE strong-motion duration is 17.3 seconds. These time durations are based on the cumulative energy from 5 percent to 95 percent of the total energy.
- iii) The CAV for the NFE is approximately 0.08 g-sec as shown in Fig. 15. This is significantly lower than the 0.16 g-sec limit set in Reference 11.
- iv) Figure 13 shows a comparison between the cracking threshold spectrum (3 percent damping) for unreinforced masonry basement wall given in Figure 4-19 of Reference 4 and the NFE spectra (2 percent damping) for the 100 and 200 areas. The NFE spectra above 2.5 hz falls well below the threshold cracking spectrum. As the threshold cracking spectrum is known to be a conservative indicator of damage to heavy industrial structures and equipment, this indicates that the damage potential of NFE is insignificant. It is also interesting to note that the bounding spectra given in Reference 8 nearly envelopes the NFE spectra as shown in Figure 14.
- v) As shown in Appendix A, the design NFE spectra, which is corrected for the acceptable inelastic distortion of the anchorage and the incoherency of the ground motion, exceeds the DBE spectra only by a small magnitude and over a narrow frequency range. This exceedance has no significant seismic safety consequences and, therefore, need not be considered in the design and evaluation of SC1 structures and equipment.

Since all SC1 equipment and associated anchorage either have been or will be evaluated for the DBE loading, no separate anchorage evaluation is needed for the NFE.

8.2 ACCELERATION SENSITIVE EQUIPMENT

As stated on page 2.3 of Reference 4, the OBE exceedance criteria does not address the relay chatter issue. Similarly, Appendix A does not provide any guidelines for the evaluation of these types of equipment. The SC1 acceleration sensitive equipment and relays, contactors, motor starters and switches that require functional operability during a design basis seismic event must be evaluated for the NFE effects. For this purpose, the EPRI A46 and GERS data base may be utilized in the following manner:

- The EPRI data base identifies the equipment types which have unsatisfactory performance. For example, the relays falling under this category may be replaced by relays which have been proven to operate satisfactorily under high frequency motions.
- Obtain the floor seismic response spectra and utilize the GERS to evaluate the equipment. The GERS can only be used if a seismic similarity of the equipment in question with the equipment for which GERS is available can be demonstrated.
- In lieu of using the above method of association as a qualification technique, equipment can be qualified by test.

9.0 REFERENCES

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Figure 1 RESPONSE SPECTRA FOR NEAR-FIELD AND DESIGN BASIS EARTHQUAKES
(All areas except 100 and 200 areas)

Figure 2 RESPONSE SPECTRA FOR NEAR-FIELD AND DESIGN BASIS EARTHQUAKES
(100 and 200 areas only)

FIGURE 3 DBE VELOCITY TRACE

FIGURE 4 DBE DISPLACEMENT TRACE

FIGURE 5 DBE ENERGY vs. TIME

FIGURE 6 DBE ENERGY VS. FREQUENCY

FIGURE 7 NFE VELOCITY TRACE

FIGURE 8 NFE DISPLACEMENT TRACE

FIGURE 9 NFE ENERGY VS. TIME

FIGURE 10 NFE ENERGY VS. FREQUENCY

FIGURE II DBE AND NFE ENERGY VS. FREQUENCY

FIGURE 12 DBE AND NFE POWER DENSITY SPECTRA

3% DAMPING
THRESHOLD CRACKING OF
UNREINFORCED MASONRY WALLS
EPRI NP-5930, Fig 4.19

FIGURE 13 RESPONSE SPECTRA FOR SMALL-MAGNITUDE, NEAR-FIELD EARTHQUAKE, 0.20g
(100 AND 200 AREAS ONLY)

FIGURE 14 RESPONSE SPECTRA FOR SHALL-HIGHITUDE, NEAR-FIELD EARTHQUAKE, 0.20g, & SQUG BOMBING SPECTRA (100 AND 200 AREAS ONLY)

FIGURE 15 DBE AND NFE CUMULATIVE ABSOLUTE VELOCITY

DEVELOPMENT OF SEISMIC DESIGN RESPONSE SPECTRUM
FOR STRUCTURES AND EQUIPMENT SUBJECTED TO
HIGH FREQUENCY SEISMIC MOTIONS

By

Robert P. Kennedy
RPK Structural Mechanics Consulting
Yorba Linda, California, USA

August 1992

Prepared For

Westinghouse Hanford Company
Richland, Washington

1. Background

The results of recent seismic hazard analyses conducted for the Hanford Site indicate that a small magnitude Near Field Earthquake (NFE) has relatively large high-frequency spectral acceleration content above approximately 10 Hz. Because of this high frequency content, the NFE response spectrum exceeds the Design Basis Earthquake (DBE) response spectrum by as much as a factor of 1.58 at 13 Hz and over frequencies from 9 Hz to 45 Hz at the 100 and 200 areas (see Figure 1). Exceedances are less at other areas, so the following discussion will concentrate on the 100 and 200 areas.

The NFE spectrum is in sharp contrast to the DBE spectrum, which has its peak spectral accelerations at frequencies less than 10 Hz and returns to the peak ground acceleration at frequencies between 30 and 40 Hz. The shape of this DBE design spectrum is based upon the characteristics of larger magnitude western North America (WNA) earthquake ground motions which have been of sufficient amplitude to be potentially damaging to engineered structures and/or equipment.

Two questions which must be addressed are:

1. What are the potential seismic safety consequences to facilities from this NFE spectrum which substantially exceeds the facility design spectrum at frequencies above about 10 Hz?
2. How should the NFE spectrum be modified in order to produce a design spectrum to be used by engineers in their evaluation of structures and equipment?

Over the last few years, these two questions have received considerable attention (for example, see Refs. 1 through 6). An extensive analytical study on the effects of high-frequency seismic motion has recently been completed for the Electric Power Research Institute (EPRI). The report on this study (Ref. 1) is expected to be published in 1993. However, the conclusions of this study have been summarized in Refs. 2 and 3 (copies

attached). Herein, I intend to summarize only the most salient points with regard to the first question. Most of this write-up addresses the second question. For more background, it is suggested that the reader review Refs. 1 through 6.

With regard to the first question, two points should be emphasized. First, I know of no case in which a well-engineered, reinforced concrete or steel structure has ever been significantly distressed by earthquake ground motion, except when the 5% damped spectral accelerations exceeded 0.20g or the spectral velocity exceeded 5 inches/second (13 cm/sec) at frequencies well below 10 Hz. This statement is applicable even when such structures had little or no seismic design. Furthermore, significant distress refers to significant visible cracking of concrete or yielding of steel and is well short of failure. Damage correlates best with spectral accelerations below 4 Hz. These statements are also applicable to anchored industrial (commercial) grade equipment typical of that in power plants, even when neither the anchorage nor the equipment were designed for seismic-induced forces. Conscientious searches for exceptions to these statements have been conducted by several investigators with no success to date. Correlation of damage to industrial facilities and engineered structures with various ground motion parameters are reported upon in Refs. 4 and 5. In fact, Ref. 5 defines *potentially damaging* ground motion as being ground motion which produces 5% damped spectral accelerations in excess of 0.20g at frequencies less than 10 Hz. In addition, the ground motion must have adequate duration.

Second, this empirical evidence--that damage to a well-engineered structure or anchored equipment will occur only if the ground motion produces substantial spectral accelerations at frequencies below 10 Hz--is supported by analytical studies. For a concrete or steel structure with a lateral load-carrying system (shear walls, braced frame, etc.) to be severely distressed, it must undergo significant interstory drifts in at least one story. In most cases, interstory drifts must exceed 0.4% of the story height before one might reasonably expect severe distress. Thus,

for a 10-foot (3-meter) story height, interstory drift must exceed about 0.5 inches (12 mm). As the structure drifts nonlinear, its effective frequency is lowered. Within my experience, any structure undergoing at least 0.4% interstory drift will have an effective frequency of less than 5 Hz by the time such a drift is reached. Unless the input ground motion produces sufficient spectral acceleration below 5 Hz to overcome the spectral acceleration capacity of the structure, it will be impossible for the structure to develop at least 0.4% interstory drift. The issue of the effective frequency of structures being lowered as the structure drifts nonlinear has been extensively studied, with the results presented in Refs. 7 and 8.

Two hundred nonlinear time-history analyses were conducted on a dynamic model of the Diablo Canyon Turbine Building using 25 different ground motion time histories, with each being scaled to 3 different amplitudes (Ref. 8). One purpose of this study was to define the characteristics of the ground motion which were required for this ground motion to be potentially damaging to the turbine building. It was found that to be potentially damaging (interstory drift in excess of 0.4%), the ground motion had to produce 5% spectral accelerations S_A which exceeded all three of the following limits:

High Frequency Limit

$S_A > 1.6g$ within 8.6 to 9.5 Hz range

Mid Frequency Limit

$S_A > 3.2g$ within 2.8 to 6.0 Hz range

Low Frequency Limit

$S_A > 2.75g$ within 2.4 to 3.5 Hz range

Unless the high frequency limit was exceeded at some frequency in the 8.6 to 9.5 Hz range, the structure remained elastic and did not begin to soften due to nonlinear drift. Unless the mid frequency limit was exceeded, nonlinear drifts stopped at very low interstory drift levels. Last, unless the low frequency limit was exceeded within the 2.4 to 3.5 Hz range, interstory drifts of 0.4% could not be reached. The conclusion is that

there must be substantial spectral acceleration content over a broad frequency range and that this frequency range must extend to frequencies below 3.5 Hz for the ground motion to be potentially damaging to this structure.

Most industrial facility type structures have fundamental frequencies in the 1 to 10 Hz range and have sufficient interstory drift capacity to drop their effective frequency below 4 Hz before severe distress will develop. Based upon the nonlinear studies presented in Refs. 7 and 8 and other similar studies, I believe that it is widely accepted among knowledgeable engineers that ground motion must have substantial spectral acceleration content in the 1 to 4 Hz range to be potentially damaging to industrial facility type structures, and that higher spectral accelerations at frequencies in excess of about 10 Hz are of little or no interest in the design of such structures, unless a very severely poorly designed and constructed link exists within the structure.

Thus, for industrial facility type structures, the NFE spectrum shown in Figure 1 is incapable of damaging any structure which has been properly designed to the DBE spectrum. No potential seismic safety consequences should exist for the structures due to the NFE spectrum substantially exceeding the design spectrum at frequencies in excess of 10 Hz.

Most equipment within an industrial facility/nuclear power plant has fundamental frequencies in the 5 to 25 Hz range. Some analysts compute equipment fundamental frequencies substantially in excess of 25 Hz. However, within my experience, this situation occurs only when the analyst has either ignored or mistakenly estimated the flexibility of the attachment of the equipment to the structure. Within my experience, it is nearly impossible to mount a piece of equipment sufficiently rigidly in a structure to produce a fundamental frequency substantially in excess of 25 Hz at high shaking levels, unless the equipment has trivial mass and thus is not vulnerable to seismic-induced distress (relay chatter may be an exception). Thus, for

equipment, we are primarily concerned with spectral accelerations up to about 25 Hz.

However, all but the most brittle equipment has sufficient distortion capability to quickly drop its effective frequency to well below 25 Hz before it is severely damaged. In most cases, the attachment of the equipment to the structure must undergo a minimum distortion of at least 0.06 to 0.12 inches (1.5 to 3 mm) before its load transfer capability is fully mobilized.

A simplistic but reasonable way to look at the damage capability of input motion to equipment follows. No element within the load transfer path will undergo an intra-element distortion in excess of the spectral displacement content of the input motion at the effective frequency of the equipment when it undergoes this distortion. Assuming an input spectral acceleration of 0.6g, the spectral displacements corresponding to various frequencies are:

Frequency	Spectral Displacements Corresponding to 0.6g Spectral Acceleration
5 Hz	0.24 inch (6 mm)
10 Hz	0.06 inch (1.5 mm)
25 Hz	0.01 inch (0.24 mm)

Even if all of the distortion is concentrated into a single element, this element will not fail if its distortion capability exceeds the spectral displacement of the input motion at the effective natural frequency of the equipment. Since the weak-link elements of most items of rugged industrial equipment can accommodate distortions of at least 0.1 inch (2.5 mm), spectral accelerations of less than 1g at frequencies of 10 Hz and greater are unlikely to result in equipment failure. Therefore, it is reasonable to conclude that the damaging frequency range for most items of equipment is less than 10 Hz, with the exception of relay chatter during the shaking.

A more critical potential exception occurs when a very strong piece of equipment is attached to the supporting structure through a stiff but weak anchorage. In this case the entire load path except for the anchorage remains elastic, and all of the nonlinear distortion must be accommodated within the weak anchorage. Based upon a review of common anchorage schemes, it was concluded that fillet welds loaded transverse to the welds represented the anchorage approach with the least distortion capability. See Figures B-8 through B-12 of Ref. 3 (attached) for a comparison of the distortion capability of various anchorages. Based upon Ref. 9, the distortion capability of a fillet weld loaded transverse to the weld is about 5.6% of the weld size. Other investigators have reported somewhat greater distortion capability, but this capacity from Ref. 9 will be used herein. The smallest weld that would be used to anchor a structural base of a piece of equipment would be 3/16 inch (5 mm). Such a weld would have a minimum distortion capability of at least 0.01 inch (0.25 mm). Somewhat smaller welds might be used to anchor a sheet metal cabinet, but in this case the sheet metal will undergo more than 0.01 inch (0.25 mm) distortion prior to weld failure. It is concluded that the least credible distortion capability of any anchorage is 0.01 inch (0.25 mm).

The effect of nonlinear anchorage distortion is to lower the effective frequency of the equipment and to increase the energy dissipation capability (see Ref. 7). Both effects are commonly defined in terms of an inelastic energy absorption factor F_u by which the elastic computed demand (spectral acceleration demand) can exceed the ultimate capacity (spectral acceleration capacity) without failure. Then, to maintain a consistent margin of safety, the elastic spectral accelerations $S_A(f, \beta)$ associated with a frequency f and damping β should be reduced by F_u to obtain an effective design spectral acceleration $S_{AD}(f, \beta)$. Thus:

$$S_{AD}(f, \beta) = \frac{S_A(f, \beta)}{F_u(f, \beta)} \quad (1)$$

For a 5 Hz piece of equipment subjected to ground motion, the lowering of the effective frequency and increase in energy dissipation due to a 0.01 inch (0.25 mm) anchorage distortion is negligible, because the elastic distortion of the 5 Hz piece of equipment is many times greater than 0.01 inch. Thus, at 5 Hz, $F_u \approx 1.0$ and there is no benefit from inelastic anchorage distortion. Therefore, we have come to treat anchorages as "brittle" and have traditionally defined our design spectral acceleration S_{AD} as being equal to the elastic spectral acceleration S_A .

However, for a 25 Hz piece of equipment, the lowering of the effective frequency and increasing of energy dissipation due to a 0.01 inch (0.25 mm) nonlinear anchorage distortion is substantial because such a nonlinear distortion is no longer small when compared to the elastic distortion of the equipment. For an NFE spectrum such as that in Figure 1, both the lowering of the effective frequency and the increasing of energy dissipation associated with a 0.01 inch (0.25 mm) nonlinear distortion are substantial. The net effect is that F_u is significantly greater than unity, and the design spectral acceleration at 25 Hz should be much less than the elastic NFE spectrum at 25 Hz. However, for WNA ground motions which produce spectra that are similar in shape to the DBE design spectrum in Figure 1, the lowering of the effective frequency below 25 Hz actually increases the spectral acceleration input, which counters the benefit gained from the increased energy dissipation, and F_u is again close to unity.

Because, in the past, design response spectra have always been low-frequency content spectra similar to the DBE design spectrum shown in Figure 1, and because small nonlinear distortions such as 0.01 inch (0.25 mm) do not produce an F_u significantly greater than unity for such spectra even at 25 Hz, we have typically not modified such spectra. However, when dealing with a high-frequency content NFE spectrum, this spectrum should be modified by the appropriate F_u associated with at least some minimal nonlinear distortion such as 0.01 inch (0.25 mm). Making such a modification will produce spectra more similar in

shape to the design spectra which have been used in the past. In the next section I will describe a method for estimating F_u .

2. Inelastic Energy Absorption Factor Correction

In a recent study sponsored by the Electric Power Research Institute, a large number of nonlinear time history analyses were performed on the component sliding and rocking models shown in Figures 2 and 3. In these models, the component was represented by two masses consisting of a dynamic (sprung) mass defined as $(1 - \alpha)M$ and a rigid mass αM at the cabinet base. The component was given sufficient strength to remain elastic in all analyses. The cabinet mass M and stiffness K_s was varied to produce fixed base frequencies ranging from 2 to 25 Hz. This fixed base frequency represents the frequency of the cabinet at low shaking levels, i.e., when sliding is resisted by friction and rocking is resisted by the cabinet weight. The anchorage was represented as an elasto-perfectly plastic system with a weld capacity F_w and a stiffness K_w . However, consistent with the properties of welded anchorage, K_w was large so that the yield drift δ_y of the anchorage when the capacity F_w was reached was only 0.001 inch (0.025 mm), which is insufficient to shift significantly the frequency of even a 25 Hz component. In these analyses, the anchorage was allowed to undergo an ultimate distortion δ_u of 0.0105 inch (0.27 mm), which represents essentially the lowest distortion capability that one might expect from any anchorage. A range of weld capacities F_w were studied. A large number of actual and artificial ground-motion time histories with a wide variety of frequency contents were used. These time histories were scaled to produce the ultimate anchorage drift δ_u . The results of these studies will be presented in Ref. 1.

It was found that the nonlinear responses from these analyses could be accurately estimated using a pseudo linear elastic model with an effective frequency f_e and an effective damping β_e in which the effective frequency and damping are directly related to the elastic fixed base frequency f_f and damping β_f and the ultimate distortion δ_u and yield capacity F_y

of the anchorage. For sliding, the yield capacity of the anchorage is:

Sliding

$$F_y = F_w + \phi (Wt) \quad (2)$$

where F_w is the anchor capacity, ϕ is the coefficient of sliding friction, and (Wt) is the cabinet weight. For rocking, the yield capacity of each anchorage is:

Rocking

$$F_y = F_w + 0.5 (Wt) \quad (3)$$

The conclusion--that the nonlinear response of the sliding and rocking models shown in Figures 2 and 3 can be accurately estimated using a pseudo linear elastic model with an effective frequency f_e and effective damping β_e --is consistent with the conclusions of Refs. 7 and 8, which showed that such models can be used to predict the response of structures. These same conclusions were previously presented in Refs. 10 through 12, and Refs. 7 and 8 simply represent an extension of this previous work.

Tables 1 and 2 of Ref. 2 present the necessary equations for the effective frequency f_e and effective damping β_e and the resulting inelastic energy absorption factor F_p for the sliding and rocking models of Figures 2 and 3, respectively. These equations are complicated by the presence of the two masses (two modes) in the sliding and rocking models of Figures 2 and 3. Frankly, I am not in favor of this added complexity, because I believe it tends to hide the fundamental ideas involved and does not provide an appreciable improvement in the estimation of F_p . Therefore, I will present the derivation and necessary equations for the case when $\alpha = 0$, i.e., the rigid masses in Figures 2 and 3 are set to zero. First, I will go through the necessary derivation for sliding.

With $\alpha = 0$, the sliding case shown in Figure 2 simplifies down to the situation shown in Figure 4. The first step in the evaluation is to define the yield capacity F_y of the anchorage (from Equation 2, if one wishes to include the influence of friction) and the permissible ultimate anchorage distortion δ_u . Then, the spectral acceleration capacity S_{AC} of this weak anchorage link is:

$$S_{AC} = \frac{F_y}{M} = \frac{F_y}{(Wt)}g \quad (4)$$

where g is gravity.

Next, one defines the maximum drift of the mass center δ_m when the anchorage distorts δ_u :

$$\delta_m = \left(\frac{F_y}{K_f}\right) + \delta_u \quad (5)$$

where the first term represents the elastic deformation of the component in which K_f is the fixed base component stiffness. Given δ_m , the secant stiffness K_s is defined by:

$$K_s = \frac{F_y}{\delta_m} \quad (6)$$

Defining X as the nondimensional ratio of the secant stiffness to the fixed base stiffness (or alternately as the square of the ratio of secant frequency to fixed base frequency), one obtains:

$$X = \left(\frac{K_s}{K_f}\right) = \left(\frac{f_s}{f_f}\right)^2 = \frac{F_y}{K_f \delta_m} \quad (7)$$

Combining Equations 5 and 7, one obtains:

$$X = \frac{1}{1 + A} \quad (8)$$

where:

$$A = \frac{K_f \delta_u}{F_y} \quad (9)$$

Alternately, A may be expressed in terms of the fixed base frequency f_f and spectral acceleration capacity S_{AC} by noting that:

$$(2\pi f_f)^2 = \frac{K_f}{M} \quad (10)$$

and combining Equations 4, 9, and 10. Thus:

$$A = \frac{(2\pi f_f)^2 \delta_u}{S_{AC}} \quad (11)$$

Equation 11 tends to be more useful than Equation 9 to define A for parametric studies. Given A, one may then estimate the secant stiffness K_s and secant frequency f_s from Equations 7 and 8.

The effective stiffness will lie between the fixed base stiffness and the secant stiffness. Defining X_e as the ratio of effective stiffness K_e to fixed base stiffness, then:

$$X_e = \left(\frac{K_e}{K_f}\right) = \left(\frac{f_e}{f_f}\right)^2 \quad (12)$$

where f_e is the effective frequency. Then:

$$X \leq X_e \leq 1.0 \quad (13)$$

From the nonlinear time-history studies of anchorage which will be presented in Ref. 1 and which were briefly described previously, an excellent estimate of X_e has been found to be provided by:

$$X_e = 1 - (1 - X)^{1.6} \quad (14)$$

Thus, given X , one may estimate X_e from Equation 14 and then K_e and f_e from Equation 12.

The relationship between X_e and X provided by Equation 14 is very similar to that presented by Kennedy (Ref. 7), Iwan (Ref. 12), and Sozen (Refs. 10 and 11) for somewhat different nonlinear problems, as can be seen from the following table.

X	X_e			
	Eqn 14	Kennedy	Iwan	Sozen
0.9	0.97	0.99	0.97	0.9
0.75	0.89	0.91-0.95	0.92	0.75
0.6	0.77	0.74-0.85	0.85	0.6
0.45	0.62	0.52-0.70	0.76	0.45
0.3	0.43	0.38-0.48	0.62	0.3
0.15	0.23	0.23	0.38	0.15

The values given by Kennedy are a function of the duration of strong motion, with the lower values applying to long-duration motion and the higher values applying to short-duration motion. The X_e values from Equation 14 lie close to the center of the range of values given by Kennedy for a somewhat different nonlinear problem. Also, the X_e values from Equation 14 lie between the values given by Iwan and Sozen for somewhat different nonlinear problems. Thus, Equation 14 seems to be consistent with previous research study results.

Next, one must estimate the effective damping β_e following the approach described in Appendix C of Ref. 7. The first step is to obtain the secant damping β_s associated with the nonlinear model producing a drift δ_m at a capacity F_y . This secant damping is the combination of the effective elastic damping β_{fp} of the

component plus hysteretic damping β_H needed to approximate the average hysteretic energy loss during nonlinear cycles. Thus:

$$\beta_S = \beta_{FP} + \beta_H \quad (15)$$

If the component has a fixed base elastic damping β_F associated with the fixed base frequency f_f , then the effective elastic damping β_{FP} associated with the secant drift δ_m and capacity F_y becomes:

$$\beta_{FP} = \left(\frac{f_S}{f_f}\right) \beta_F = (X)^{1/2} \beta_F \quad (16)$$

The hysteretic damping term β_H was approximated by an empirical fitting to the results of the nonlinear time-history analyses using an equation form based on the maximum possible hysteretic loop times an empirically defined coefficient b . As shown in Table 1 of Ref. 2:

$$\beta_H = b \left[\frac{2}{\pi} - \beta_{fp} \left(\frac{1 - G\Omega}{1 - G} \right) \right] \left(1 - \frac{X}{D} \right) (1 - G) \quad (17)$$

where

$$b = 0.3 \quad (17a)$$

is an empirical coefficient indicating that the effective average hysteretic damping is 30% of the maximum possible hysteretic damping, and G , Ω , and D are terms defined in Table 1 of Ref. 2. Since Equation 17 is essentially an empirical equation due to b being empirically defined, and since a 20% variation in β_H has little effect on the finally derived S_{AD} or F_p , I consider Equation 17 to be far more complicated than is warranted and suggest that it be greatly simplified.

The term D is primarily related to α . With $\alpha = 0$ (i.e., no rigid mass):

$$D = 1.0 \quad (18)$$

and will never exceed 1.3 for practical values of α and X . Therefore, I suggest that Equation 18 be conservatively used to define D . Similarly, an upper bond which is never excessively conservative for G is:

$$G \leq \frac{\delta_y}{\delta_u} \quad (19)$$

where δ_y is the yield distortion of the anchorage and δ_u is the ultimate permissible weld distortion. For welds, which are considered to be the most brittle anchorage, based on the data in Ref. 9, $\delta_y/\delta_u \leq 0.095$. Thus:

Welds

$$G \leq 0.095 \quad (20)$$

should be used. Last, G will always be close to and slightly greater than unity. Thus, it is conservative to assume:

$$G \geq 1.0 \quad (21)$$

Incorporating the conservative approximations of Equations 18, 20, and 21 into Equation 17, one obtains:

$$\beta_H = 0.173(1 - X) \quad (22)$$

The degree of conservatism introduced by the use of Equation 22 in lieu of Equation 17 is negligible, and Equation 22 fits the nonlinear results essentially as well as Equation 17. I strongly suggest the use of Equation 22 because of its simplicity.

Once the secant damping is defined using Equations 15, 16, and 22, the secant damping must be converted to an effective damping β_e to be used with the effective stiffness K_e and

effective frequency f_e . To maintain the same total energy dissipation when the effective model drifts to δ_m :

$$\beta_e = \left(\frac{K_s}{K_e}\right) \beta_s = \left(\frac{X}{X_e}\right) \beta_s \quad (23)$$

The values of β_e versus X provided by the combination of Equations 15, 16, 22, and 23 are in the same range as those suggested by Kennedy (Ref. 7), Iwan (Ref. 12), and Sozen (Refs. 10 and 11) for somewhat different nonlinear problems. Assuming that $\beta_f = 5\%$, the following β_e are obtained by each of these approaches for various X values:

X	β_e (%)			
	Recommended Herein	Kennedy	Iwan	Sozen
0.9	6.0	5.1 - 5.9	7.6	6.0
0.75	7.3	5.3 - 7.1	8.9	7.7
0.6	8.4	6.1 - 8.3	10.1	9.5
0.45	9.3	7.5 - 9.6	11.3	11.6
0.3	10.4	7.9 - 11.6	13.0	14.0
0.15	10.9	7.7 - 15.2	16.2	17.3

The values given by Kennedy are for in-plane nonlinear response of shear-wall and braced-frame structures that exhibit severely pinched hysteretic behavior, at least for longer strong motion durations. The lower values from Kennedy are for long-duration motions, while the upper values are for short-duration motions for which this pinching is not severe. One would expect β_e for nonlinear anchorage behavior to be similar to the upper values from Kennedy. The differences between the β_e values recommended herein and the upper values from Kennedy will typically have less than a 4% effect on computed response, and so these differences are negligible. Even the differences between the damping values recommended herein and values midway between those recommended by

Iwan and Sozen will typically have less than an 10% effect on computed response, and so these differences are also small. Thus, the β_e approach recommended herein seems consistent with previous research study results.

For any given time history with its corresponding elastic response spectrum, the spectral acceleration response S_{ARE} is defined by:

$$S_{ARE} = S_A(f_e, \beta_e) \quad (24)$$

However, corresponding to a total drift δ_m , the capacity F_{ye} of the pseudo elastic effective model is:

$$F_{ye} = \left(\frac{K_e}{K_s}\right) F_Y = \left(\frac{X_e}{X}\right) F_Y \quad (25)$$

Thus, in spectral terms, the effective capacity S_{ACE} becomes:

$$S_{ACE} = \left(\frac{X_e}{X}\right) S_{AC} \quad (26)$$

The drift δ_m is reached when the ground motion is scaled by a factor F_I sufficiently that S_{ARE} equals S_{ACE} . Thus:

Inelastic

$$S_{AC} = \left(\frac{X}{X_e}\right) S_A(f_e, \beta_e) F_I \quad (27)$$

whereas based on elastic analysis the capacity is reached when the ground motion is scaled by a factor F_E so that the fixed base elastic spectral acceleration equals S_{AC} . Thus:

Elastic

$$S_{AC} = S_A(f_f, \beta_f) F_E \quad (28)$$

The inelastic energy absorption factor F_{μ} is simply the ratio of F_I to F_E or:

$$F_{\mu}(f_f, \beta_f) = \frac{F_I}{F_E} = \left(\frac{X_e}{X}\right) \frac{S_A(f_f, \beta_f)}{S_A(f_e, \beta_e)} \quad (29)$$

Then, from Equation 1, the effective design spectral acceleration S_{AD} to use at a frequency f_f and β_f becomes:

$$S_{AD}(f_f, \beta_f) = \left(\frac{X}{X_e}\right) S_A(f_e, \beta_e) \quad (30)$$

When one knows the spectral acceleration capacity S_{AC} of the component anchorage, the permissible drift δ_u , and has a specific input response spectrum, then the process of estimating margin (scale factor F_I) and the inelastic energy absorption factor F_{μ} is straightforward, using the following steps:

- Step 1: Determine A (Equation 11)
- X (Equation 8)
- X_e (Equation 14)
- f_e (Equation 12)
- β_{FP} (Equation 16)
- β_H (Equation 22)
- β_S (Equation 15)
- β_e (Equation 23)

Step 2: Determine $S_A(f_e, \beta_e)$ from the input response spectrum.

Step 3: Find the margin (F_I) from Equation 27, F_{μ} from Equation 29, and S_{AD} from Equation 30.

As an example, it will be assumed that a component has a fixed base frequency of 20 Hz, fixed base damping of 5%, an anchorage spectral-acceleration capacity against sliding of 0.70g, and a permissible anchorage distortion of 0.01 inch

(0.25 mm). The NFE spectrum from Figure 1 will be used as input. Then, following the above steps, it is found that $A = 0.583$, $X = 0.632$, $X_e = 0.798$, $f_e = 17.9$ Hz, and $\beta_e = 8.2\%$. From the input spectrum of Figure 1, which is plotted at 5% damping, I find that $S_A(17.9 \text{ Hz}, 5\%) = 0.41g$. For $\beta_e = 8.2\%$, I estimate that $S_A(17.9 \text{ Hz}, 8.2\%) = 0.344g$. Thus, from Equation 27, $F_I = 2.57$. At the fixed base frequency of 20 Hz, and damping of 5%, the $S_A(20 \text{ Hz}, 5\%) = 0.38g$; and based on elastic analysis, the elastic scale factor F_E from Equation 28 would be $F_E = 1.84$. Thus, from Equation 29, $F_\mu = 1.40$. For elastic analysis to have predicted the appropriate margin, the effective design spectral acceleration S_{AD} at 20 Hz and 5% damping would have to be reduced using Equation 30 to $S_{AD}(20 \text{ Hz}, 5\%) = 0.27g$. In other words, the effective design spectral acceleration at 20 Hz should be only 0.27g rather than the elastic spectral acceleration of 0.38g. This example shows the dramatic reduction achieved at 20 Hz for a nonlinear distortion of only 0.01 inch (0.25 mm).

Now, if this same example component were reduced to a fixed base frequency of 5 Hz, very little benefit would accrue from a 0.01 inch (0.25 mm) anchorage distortion. In this case, $f_e = 5.0$ Hz, $\beta_e = 5.4\%$, and $(X/X_e) = 0.970$. From Figure 1, I estimate $S_A(5.0 \text{ Hz}, 5\%) = 0.24g$; and thus, $S_A(5.0 \text{ Hz}, 5.4\%) = 0.235g$. Therefore, $F_I = 3.07$ versus $F_E = 2.92$, from which $F_\mu = 1.05$, which is negligibly small. There is essentially no reduction in the elastic spectrum at 5% from a 0.01 inch (0.25 mm) nonlinear distortion.

Unfortunately, F_μ and thus S_{AD} is a function of the spectral acceleration capacity S_{AC} . To illustrate, the capacity of the 20 Hz model will be doubled to $S_{AC} = 1.40g$. Then, $f_e = 19.0$ Hz, $\beta_e = 7.1\%$, and $(X/X_e) = 0.853$, from which $F_I = 4.65$, whereas F_E doubles to $F_E = 3.68$, thus yielding $F_\mu = 1.26$ versus 1.40 when $S_{AC} = 0.70g$. With this lesser F_μ , the effective design spectral acceleration to be used in an elastic evaluation to produce the same computed margin as F_I becomes $S_{AD} = 0.30g$. Although the effect is not dramatic, doubling S_{AC} increases S_{AD} at 20 Hz by 11%.

When one has an input elastic response spectrum such as the NFE spectrum shown in Figure 1 and wishes to define a reasonable effective design spectrum $SAD(f,\beta)$ for design, one does not typically know SAC. In this case, one must establish a minimum permissible margin FI between SAC and SAD. This margin must cover the following two issues:

1. The amplification of the high frequency (greater than 10 Hz) spectral accelerations within a structure, i.e., the amount that the input spectral acceleration at the base of a component mounted within the structure will exceed the ground spectral acceleration at the same equipment frequency. For NFE spectra with shapes similar to that in Figure 1, fundamental structure frequencies below 10 Hz, and at equipment fundamental frequencies greater than about 1.5 times the structure fundamental frequency, these amplifications should be less than 2.0. Often, they will be much less than 2.0.
2. A minimum desired safety margin. For DOE facilities, this minimum desired safety margin is typically in the range of 1.5 to 2.0.

Therefore, I suggest that the minimum permissible margin FI to cover both of these two issues be slightly conservatively set at 4.0. Thus:

$$SAC(f,\beta) = 4 SAD(f,\beta) \quad (31)$$

The process to define $SAD(f,\beta)$ for a given elastic input spectrum then becomes iterative. First, a trial SAC is selected and the previously defined Steps 1 through 3 are followed to find SAD. Then Equation 31 is used to establish a new trial SAC and Steps 1 through 3 are repeated. The process is repeated until the chosen SAC and resultant SAD closely satisfy Equation 31. The process converges rapidly and typically only two or at most three trials are necessary.

So far, this write-up has explained the process to develop S_{AD} using the sliding anchorage failure model of Figure 2. The rocking anchorage failure model of Figure 3 was also studied and could be equally well-predicted using a pseudo linear elastic model with an effective frequency f_e and effective damping β_e . The necessary equations for predicting f_e , β_e , and F_u for the two-mass rocking model shown in Figure 3 are given in Table 2 of Ref. 2.

However, as shown in Ref. 2, the rocking model will nearly always produce a larger F_u and thus a lesser S_{AD} than the sliding model. In fact, the rocking model cannot produce a lesser F_u (higher S_{AD}) than the sliding model except at high fixed base frequencies f_f , for which the sliding model predicts an effective frequency f_e sufficiently above the frequency at which the input spectrum peaks; so that $S_A(f_e, 10\%)$ is significantly less than the peak 10% damped input spectral acceleration $S_{AP}(10\%)$. The rocking model always produces an effective frequency less than the sliding model and β_e greater than for the sliding model. For the NFE spectrum shown in Figure 1, this situation of the rocking model controlling could occur only when the sliding model produces an effective frequency in excess of 21 Hz. As will be shown subsequently, for a fixed base frequency of 25 Hz and this NFE spectrum, the effective frequency for the sliding model is only 21 Hz at an anchor distortion of 0.01 inch (0.25 mm). For this NFE spectrum, the rocking model can control only for fixed base frequencies in excess of 25 Hz.

It has also been found that the rocking model can never control so long as S_{AD} does not fall below the following asymptotic limit or the stiff side of the peak of the input spectrum:

$$S_{AD\text{MIN}} = \frac{S_{AP}(10\%)}{1.6} \quad (32)$$

It is recommended that S_{AD} not be taken below the limit of Equation 32 or the elastic spectral acceleration, whichever is

less at frequencies greater than that at which the elastic spectrum peaks.

3. Spatial Variation and Incoherence of Ground Motion

The ground motion response spectrum estimated from seismological studies represent the elastic response of simple oscillators mounted on a small lightweight pad on the free ground surface. These are not the motions "seen" by a large massive structure founded on a substantial size basemat. Comparison of response spectra obtained from motions measured on the basemat of large structures with those obtained from motions measured on small lightweight pads show substantial differences, particularly at frequencies in excess of 5 Hz (for instance, see Ref. 13). At frequencies of 10 Hz and greater, the spectral accelerations obtained on the basemat of large structures are often more than a factor of two less than those obtained on a small pad on the free ground surface.

These reductions in the spectral accelerations at higher frequencies input to large structures appear to be due to two causes:

1. Vertical spatial variation of the ground motion, wave scattering effects, and soil-structure interaction.
2. Horizontal spatial variation and incoherence of the ground motion.

On soil sites, the first cause is likely to be the greatest source of these high-frequency reductions. However, on a stiff rock site, this cause is not likely to result in much reduction (probably less than a 10% reduction, unless the structure is deeply embedded). Furthermore, when appropriate, these reductions have generally been taken into account in the seismic design or evaluation of stiff structures. Therefore, I will not further discuss herein the first cause.

Horizontal spatial variation and incoherence of the ground motion will reduce the input at higher frequencies to structures

with large plan dimensions. This reduction occurs on rock sites as well as soil sites. It has seldom been considered in the seismic design or evaluation of stiff structures because the collection of observational data concerning this reduction is of recent vintage. Based on studies conducted for the U.S. Nuclear Regulatory Commission, Ref. 14 presents some recommendations concerning the reductions of higher frequency spectral accelerations input to structures to account for their large plan dimensions. These recommendations are based on a conservative interpretation of a small data set of observed variations of ground motions recorded over short separation distances. These recommendations have been adapted for use in seismic margin reviews of existing nuclear power plants in the U.S. (Ref. 15). Guidelines recommended by the U.S. Department of Energy (DOE) for use in the seismic design and evaluation of DOE facilities also provide these recommendations (Ref. 16), which state:

Horizontal spatial variations in ground motion result from nonvertically propagating shear waves and from incoherence of the input motion (i.e., refractions and reflections as earthquake waves pass through the underlying heterogeneous geologic media). The following reduction factors may be conservatively used to account for the statistical incoherence of the input wave for a 150-foot plan dimension of the structure foundation:

Fundamental Frequency of the Soil-Structure System (Hz)	Reduction Factor
5	1.0
10	0.9
25	0.8

For structures with different plan dimensions, a linear reduction proportional to the plan dimension should be used: for example, 0.95 at 10 Hz for a 75-foot dimension and 0.8 at 10 Hz for a 300-foot dimension (based on 1.0 reduction factor at 0-foot plan dimension). These reductions are acceptable for rock sites as well as soil sites. The above reduction factors assume a rigid base slab. Unless a severely atypical condition is identified, a rigid base slab condition may be assumed to exist for all structures for purposes of computing this reduction.

Since the publication of Ref. 14 (1986), considerable additional observational data has become available on the incoherency of high-frequency ground motion over short horizontal plan dimensions. This data has been collected primarily from research sponsored by the Electric Power Research Institute using a closely spaced instrument array in Lotung, Taiwan (Ref. 17). Based on this research, Dr. Norm Abrahamson has recently provided an estimate of the coherency of the ground motion at short horizontal separations for a Western U.S. nuclear power plant rock site (see Figure 5 solid lines). From Figure 5 one can note that the incoherency of the ground motion at frequencies above about 12 Hz is substantial, even at separation distances as short as 66 feet (20 meters). Based on this estimate of the coherency, an analysis has been conducted using a dynamic model of the auxiliary building at this plant to compare floor spectra obtained assuming incoherent versus coherent motion. The results of this unpublished analysis suggest that the incoherency reduction factors presented above may be slightly more conservative than necessary.

At this time, I recommend use of the incoherency reduction factors of Reference 16 presented above. However, I believe that ongoing research may result in greater reductions being suggested in the future, particularly for structures in the 66-foot (20-meter) plan dimension range.

These high-frequency incoherency reduction factors should be applied in addition to the inelastic-energy-absorption correction factor described in the previous section. In fact, the incoherency reduction factor should be applied first, since it represents a reduction in the motion input to the structure and to equipment mounted therein. Then, the inelastic-energy-absorption correction factor should be applied to this reduced motion.

For illustration, I have applied to the NFE spectrum (shown in Figure 1), the incoherency correction factors recommended herein for a 150-foot (45-meter) plan dimension

building foundation in order to obtain 5% damped effective-coherent-input spectral accelerations. The results are presented in Table 1.

4. Design or Evaluation Response Spectrum Corresponding to NFE Spectrum

A design or evaluation response spectrum S_{AD} can be produced by applying the inelastic energy absorption factor correction F_u as defined in Section 2 to the scaled effective coherent spectral accelerations S_A (see Table 1). Then the derivation of $S_{AD}(f, 5\%)$ for various frequencies f and 5% damping is presented in Table 2. The effective coherent spectral accelerations S_A are defined in Table 1 at 5% damping only. To obtain S_{AD} , it was necessary to define S_A at various damping values ranging from 5% to 11%. The following approximation was used for this purpose:

$$\frac{S_A(\beta)}{S_A(5\%)} = 1.81\beta^{-0.37} \quad (33)$$

Thus,

β	$S_A(\beta) / S_A(5\%)$
5%	1.00
6%	0.93
8%	0.84
10%	0.77

As described in Section 2, the first step is to assume a minimum permissible spectral capacity S_{AC} at any frequency f , and then to evaluate S_{AD} . Then, in accordance with Equation 31, S_{AC} should be approximately four times S_{AD} . If not, another trial is performed. As shown in Table 4, S_{AD} is only slightly sensitive to S_{AC} , so that only one or two trials are needed at each frequency.

In addition, Section 2 suggests that S_{AD} not be allowed to fall below a lower-bound asymptotic limit defined by Equation 32. Based on this limit and a 5% damped effective coherent peak spectral acceleration $S_{AP}(5\%)$ of 0.458g:

$$S_{AD\text{MIN}} = \frac{0.458g (0.77)}{1.6} = 0.22g \quad (34)$$

Comparison of this limit with the S_{AD} values from Table 2 indicates that this limit is reached at about 21 Hz.

The 5% damped Proposed Design and Evaluation Spectrum for the NFE based on Table 2 and the above limit, is shown in Figure 6. This Proposed Design and Evaluation (D&E) Spectrum for the NFE only exceeds the large magnitude DBE spectrum by a maximum factor of 1.19 at 12 Hz, and exceedances occur only over the narrow frequency range from about 10 to 17 Hz.

5. Conclusion

This brief report presents an approach for converting an NFE spectrum into a Design and Evaluation (D&E) Spectrum to be used by engineers in the design or evaluation of structures and equipment. This approach consists of two steps:

- Step 1: Apply an incoherence reduction factor to higher-frequency NFE spectral accelerations. This reduction factor is likely to range from unity at 5 Hz to about 0.8 at 25 Hz and greater as shown in Table 1. (See Section 3 for further discussion.)
- Step 2: Reduce the effective coherent NFE spectral accelerations resulting from the application of Steps 1 and 2 by an inelastic energy absorption factor F_u associated with small but acceptable inelastic distortions. Herein, it is recommended that this F_u factor be based on an inelastic distortion of about 0.01 inch (0.25 mm), since even

the most brittle anchorage is capable of undergoing this distortion. (See Section 2 for further discussion.)

For the NFE spectrum of Figure 1, these steps have been applied in Section 4 to conservatively develop the proposed D&E Spectrum shown in Figure 6. Note that the exceedances of the large magnitude DBE spectrum are small and only over a narrow frequency range. As stated earlier, higher spectral accelerations at frequencies greater than 10 Hz are non-damaging. Therefore, the higher NFE accelerations shown in Figure 6 have no seismic safety consequences for equipment and structures except for some electrical relays which have been found to chatter at high-frequency motions.

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Table 1
Conversion of High-Frequency NFE Spectrum to
Effective Coherent Input Spectrum
(5% Damping)

Frequency Hz	NFE Spectrum $S_A(5\%)$, g	Incoherency Reduction Factor	Effective Coherent $S_A(5\%)$, g
5	0.24	1.00	0.24
8	0.36	0.94	0.338
10	0.45	0.90	0.405
12	0.515	0.89	0.458
14	0.50	0.87	0.435
18	0.41	0.85	0.348
20	0.38	0.83	0.315
22	0.35	0.82	0.287
25	0.32	0.80	0.256
30	0.27	0.80	0.216
40	0.21	0.80	0.168
50	0.20	0.80	0.160

Table 2
Design Spectral Acceleration SAD(5%) for NFE

f (Hz)	Assumed SAC (g)	A	X	X _e	$\frac{X}{X_e}$	f _e (Hz)	β _e (%)	Coherent		
								SA(f _e , 5%) (g)	SA(f _e , β _e) (g)	SAD(5%) (g)
80	.47	13.9	.0670	.1050	.638	25.9	11.1	.248	.184	.117
50	.51	5.01	.166	.252	.659	25.1	10.9	.255	.191	.126
30	.63	1.461	.406	.566	.718	22.6	9.7	.280	.219	.157
25	.74	0.864	.536	.708	.758	21.0	8.9	.300	.242	.183
20	.94	0.435	.697	.852	.818	18.5	7.7	.339	.288	.236
15	1.40 1.35	0.164 0.170	.859 .854	.956 .954	.898 .895	14.7 14.7	6.4 6.4	.417 .417	.380 .380	.341 .340
12	1.60	0.0921	.916	.981	.934	11.9	5.8	.455	.431	.403
10	1.40 1.45	0.0731 0.0706	.932 .934*	.986 .987	.945 .946	9.9 9.9	5.7 5.7	.402 .402	.383 .383	.362 .362
8	1.30 1.25	0.0504 0.0524	.952 .950	.992 .992	.960 .958	8.0 8.0	5.5 5.5	.338 .338	.326 .326	.313 .312

FIGURE 1. RESPONSE SPECTRA FOR SMALL-MAGNITUDE, NEAR-FIELD EARTHQUAKE, 0.20g, 4 DBE
(100 AND 200 AREAS ONLY)

Figure 2 Model for Component Sliding

Figure 3 Model for Component Rocking

Figure 4
One Degree of Freedom Sliding Model
Force-Displacement Diagrams

HORIZONTAL - 50 m

Figure 5: Coherency of Ground Motion

FIGURE 6. RESPONSE SPECTRA FOR SHALLOW-MAGNITUDE, NEAR-FIELD EARTHQUAKE, 0.20g, 4 DBE (100 AND 200 AREAS ONLY)

ANALYSIS OF HIGH-FREQUENCY SEISMIC EFFECTS

John W. Reed¹
Robert P. Kennedy²
Bahman Lashkari¹
Robert P. Kassawara³

ABSTRACT

The effects of seismic motions rich in high-frequency energy content are discussed for both high- and low-frequency equipment. Based on the study discussed in the paper it was shown for high-frequency equipment subjected to earthquakes with high-frequency energy content that the effects are significantly less than those predicted by conventional static analysis procedures. The presence of even limited ductile energy absorbing capacity for so-called brittle anchorages such as welds can be used to develop a conservative simplified procedure to reduce an input ground response spectrum for the purpose of evaluation or design of nuclear power plant components. This procedure was calibrated based on detailed non-linear analyses of realistic equipment models for a large set of earthquakes with significant energy content which spanned frequencies between 2 and 25 Hz. The procedure and steps to use the simplified methodologies are given along with representative results.

INTRODUCTION

This paper presents preliminary results of a project conducted for the Electric Power Research Institute (EPRI) to investigate the effects of high-frequency seismic ground motions. The study was performed to develop a rational procedure for reducing input to high-frequency components which are being evaluated for earthquake response spectra rich in high-frequency energy content. The final results from the study will be documented in Reference 1 in the near future and are intended for use in the Individual Plant Examination for External Events (IPEEE) to resolve NRC Severe Accident Policy (SAP) issues (2).

This paper is organized to present first the background on high-frequency seismic effects which motivated the need to perform the study. Next is the formulation of the study to address high-frequency seismic motions and the development of a methodology for reducing input ground response spectra. This is followed by the findings and recommendations of the study for reducing input ground response spectra. The analytical

¹ Jack R. Benjamin and Associates, Inc.
² RPK Structural Mechanics Consulting
³ Electric Power Research Institute

procedures for reducing response spectra and example results are given at the end of the paper.

Readers familiar with the high-frequency seismic issue and general nature of the EPRI study might wish to skip to the section entitled: FINDINGS AND RECOMMENDATIONS. Also, the reader may want to turn first to Figure 5 which shows example reduced response spectra developed for a rock uniform hazard spectrum (UHS). These reduced spectra are representative of reductions which can be expected for the eastern United States (EUS) earthquakes based on this study.

BACKGROUND ON HIGH-FREQUENCY SEISMIC MOTIONS

The results of recent seismic hazard analyses conducted for the EUS indicate that UHS have relatively large high-frequency spectral acceleration content above approximately 10 Hz. This is in contrast to typical plant design or seismic margins-type input, such as the NRC Regulatory Guide (R.G.) 1.60 or NUREG/CR-0098 response spectra. However, these same UHS have significantly lower spectral acceleration content at the low-frequency end of the response spectrum (i.e., between about 1 and 10 Hz), which indicates to the seismic engineering community that these motions are less damaging than traditional design or evaluation-type input. Figure 1 shows the relationship between the median NUREG/CR-0098 response spectrum anchored to 0.3 g peak ground acceleration (pga) and an example UHS, with a 10,000 year return period (at the 0.85 fractile), from a hazard analysis using the EPRI methodology for a EUS nuclear power plant site.

In the analysis and design of nuclear power plant structures it is widely felt that to be damaging, earthquakes must be rich in spectral content in the 1 to 4 Hz frequency range. Because most nuclear power plant structures and equipment have fundamental frequencies significantly above 1 Hz, strong amplified motions below 1 Hz caused by soft soil conditions, although potentially damaging to flexible structures such as high-rise buildings, do not affect typical nuclear power plant sites. Thus for stiff nuclear power plant structures at typical soil or rock sites it is necessary for energy to be present in the 1 to 4 Hz frequency range for earthquakes to be potentially damaging. It is believed that the damaging frequency range for equipment probably is slightly higher, but less than 10 Hz. A possible exception is anchorage of equipment at high ground motion levels, but for this case spectral displacement, not acceleration, is more important in defining damage potential. As shown in Figure 1 the NUREG/CR-0098 spectral shape has damage-potential characteristics but the example UHS does not.

It has also been observed that damage correlates well with elastic spectral acceleration in the 1 to 4 Hz range. Thus, it is likely that it is an "effective" frequency in this range as opposed to the fundamental frequency which is important to significant damage. These results support the conclusion that it is the low-frequency content of earthquake records which causes damage.

In a recent detailed study of the Diablo Canyon Turbine Building, conducted for the Diablo Canyon seismic probabilistic risk assessment (PRA),

Figure 1 Comparison of example uniform hazard response spectrum with NUREG/CR-0098 response spectrum (5 percent damped).

25 earthquake records were input to a nonlinear building model where the model material properties were varied in a probabilistically consistent manner (3). Several hundred deterministic analyses were conducted in this study, which spanned all realistic combinations of structural and ground motion parameters. It was found that for a ground motion to be damaging it must have broad frequency content. Specifically, high spectral accelerations were required at the elastic frequencies for those dynamic modes corresponding to the relatively weak building shear walls (i.e., approximately 8 Hz) to initiate substantial inelastic drift. In addition, high spectral acceleration content also was required in the approximately 2 Hz range in order to drive the shear walls to drift levels associated with the onset of severe distress (i.e., significant damage).

One example of a high-frequency earthquake which occurred near a nuclear facility was the Leroy earthquake (magnitude M_L 5.0) which took place January 31, 1986 about 11 miles from the Perry Nuclear Power Plant (4). At the time of the event Cleveland Electric Illuminating Company was within days of core load and receipt of their 5 percent power license for Perry. The staff observed no indications of damage to systems in operation at the time, which was confirmed by follow-up inspections. However, after the earthquake the analysis of the motion records indicated that the Operating Basis Earthquake (OBE) ground response spectrum had been exceeded at frequencies above 10 Hz, and the Safe Shutdown Earthquake (SSE) response spectrum had been exceeded above 15 Hz. The maximum acceleration of the ground motion at the foundation level was 0.18 g with a corresponding peak 5 percent damped spectral acceleration of about 0.8 g; however, the peak ground velocity was only 0.9 inches per second, and the maximum ground displacement was only 0.06 inches. The latter two values indicate low energy content in the 1 to 4 Hz frequency range; thus, it is not surprising that no damage occurred. In addition, the cumulative absolute velocity (CAV) value for this event was only 0.08 g-sec which is much lower than the OBE exceedance criterion limit of 0.30 g-sec (4).

Experience with high-frequency motions from other environments indicate that they are much less damaging than their low-frequency counterparts. In the development of a criterion for determining the exceedance of the OBE, earthquake experience concerning the damaging characteristics of high-frequency motions was collected and documented (4). Based on a review of ground motions caused by conventional high explosive blasts, and their effect on structures, it was concluded that no damage will occur to engineered structures and equipment for short-duration ground motions with spectral accelerations below the envelope of the OBE response spectrum and a threshold cracking spectrum (established from blast data). However, it has not been established whether these conclusions can be extrapolated to longer duration large-magnitude high-frequency events. The conclusions that damage is not caused by high-frequency motions is also supported by fragility data for ductile equipment, performance of equipment under industrial environment vibrations and code requirements for high-frequency loading.

Based on past experience, both from earthquakes and other physical phenomena which produce high-frequency motions, it is believed that high-frequency input, which is a predominant part of UHS as shown in Figure 1,

will not be damaging to ductile structures and equipment. However, there is still a potential concern that these motions may be an issue for acceleration sensitive equipment and relays, contactors, motor starters and switches. For this category of components the effects of high-frequency motions should be addressed only if it is determined from a systems perspective that functional failure (e.g., relay chatter) during strong motion is detrimental or equally that the safety function affected can not be confidently recovered after an earthquake. This study did not address the issue of relay chatter.

FORMULATION OF STUDY

Most high-frequency components in nuclear power plants are very strong. If components have high fundamental frequencies (i.e., relatively stiff compared to their mass) they also have high strength, since stiffness and strength are correlated. In addition to being strong most high-frequency components in nuclear power plants also have ample ductile capacity to resist the small displacements associated with high-frequency motions. Examples include the casing on a vertical pump motor and the exterior shell on a small heat exchanger. These types of components are also ductile, and small displacements associated with high-frequency motions can be safely accommodated if the yield capacity is exceeded.

A class of high-frequency components which have potentially limiting capacities are stiff electrical cabinets supported at their base, where the anchorage system is potentially the weak link (e.g., welded or secured with expansion anchors). An investigation was performed to find examples of real electrical cabinets which have high fundamental frequencies based on actual testing (i.e., 15 Hz and higher). However, the high-frequency cabinet examples which were found had very strong stiff supports (overstrength welds or anchor bolts).

In the study a limiting capacity case was postulated where an electrical cabinet is assumed to have a high fundamental fixed-base frequency, and the anchorage is sized exactly to resist the input earthquake. The total structural system, including the cabinet and anchorage, has a fundamental frequency lower than the fixed-base frequency, and in some cases may no longer be classified as a "high-frequency" component. In any case, the anchorage strength was no larger than required to resist the design earthquake. From this viewpoint limiting components can be thought of as having high fixed-base frequencies (i.e., 10 Hz and higher) with minimum strength anchorage. This class of components was assumed to represent the critical case for all high-frequency nuclear power plant components and was investigated in detail. In general, the models analyzed in the high-frequency study are very conservative relative to the general population of high-frequency components found in nuclear power plants.

The only source of non-linear behavior which was assumed to occur in the study was the anchorage system at the cabinet base. Other factors which would also reduce the load on the anchorage system include the following:

- Non-linear response in the cabinet (e.g., buckling in the side panels or local flexibility in the cabinet material near the anchorage)
- Non-linear response at the connection between the electrical devices and cabinet shell
- Non-linear response in the devices themselves
- Reduction to high-frequency motions in nuclear power plant structures caused by incoherence of the ground motion

These sources of reduction exist for real electrical cabinets; however, it was conservatively assumed in the study that all potential non-linear behavior is concentrated in the equipment anchorage. This assumption allowed the study to be focused on systematically investigating the various parameters that influence the results. The disadvantage of this approach is that the additional margin which exists for high-frequency components was not quantified.

It was determined early in the investigations that welded anchorages are more brittle than bolted anchorages. Expansion anchors have ultimate displacement capacities in the range of approximately 0.1 to 0.5 inches, while small fillet welds in shear were found to have corresponding capacities in the range of about 0.01 to 0.1 inches. It is clear from the results of this study, and past investigations (5), that seismic capacity is limited by the displacement capacity of the weakest element. Thus, all the analyses were performed assuming welded anchorages. This represents another source of conservatism in the study.

FINDINGS AND RECOMMENDATIONS

It is the prevailing belief in the seismic engineering community that welded anchorages are brittle. The earthquake experience which supports this conclusion is associated with low-frequency components damaged by low-frequency seismic motions. The damagability of low frequency earthquakes is also supported by recent analytical studies (3, 5).

Figure 2a shows an example case for a "low-frequency" component (i.e., frequency f_1) which is welded at its base. The lower response spectrum in this figure represents the design input to the component (i.e., the input used to size the welded anchorage) and the upper response spectrum is the scaled input corresponding to both the yield and ultimate capacities. Two important points are demonstrated by Figure 2a for low-frequency components with brittle anchorages:

1. Once the seismic input is scaled to the level corresponding to the yield capacity (i.e., the limit of elastic response) the ultimate capacity is for practical purposes at the same level.
2. The safety margin for the component is equal to the ratio of the yield capacity to the design capacity.

a. Low-Frequency Component

b. High-Frequency Component

Figure 2 Comparison of Design, Yield and Ultimate Response Spectra for Low-Frequency and High Frequency Components

Item 1 is another way of saying that the component is brittle. By definition brittle means that when the component reaches the limit of elastic capacity (i.e., yield limit) there is no additional capacity beyond that level (i.e., the ductility scale factor is unity). Item 2 implies that the factor of safety for welds (i.e., in the range of about 2 with a stress increase factor) was established by code committees with a conscious realization that additional capacity beyond yield can not be relied upon.

Based on failure tests welds do have some ductile capacity. Although the displacement limit for welds is small it has a very important effect in increasing the capacity of high-frequency components. Figure 2b shows an example case for a "high-frequency" component (i.e., frequency f_h) which is welded at its base. Again, the lower response spectrum represents the design input, but here there are two higher response spectra, which correspond to the yield and ultimate capacity levels. Two important points are demonstrated by Figure 2b for high-frequency components with "brittle" anchorages:

1. The ultimate capacity for a high-frequency can be significantly higher than the yield capacity.
2. The total factor of safety for a high-frequency component consists of two parts:
 - A safety margin equal to the ratio of the yield capacity to the design capacity, which is the same margin provided for low-frequency components.
 - An additional margin, equal to the ratio of the ultimate capacity to the yield capacity (i.e., ductility scale factor), which does not exist, or is very small, for low-frequency components.

For high-frequency components this additional ductility scale factor is a "bonus" beyond the margin provided by code allowable weld stresses. Non-linear analyses can not practically be performed for every component as part of the standard design process. Thus, code committees have established factors of safety which reflect the design practice. Recognizing the brittle behavior for welds used to anchor low-frequency components, code writers have provided allowable weld stresses that correspond to failure at the yield level. As demonstrated in Figure 2b this is conservative for high-frequency components.

The presence of this ductility scale factor provides a rational basis for reducing an evaluation (or design*) response spectrum for high-

* In this paper it is visualized that the input response spectrum will be used to evaluate existing structures and equipment. The concepts presented are equally applicable to the situation where the input response spectrum is used to design the capacity of components as discussed below.

frequency components. Figure 3 demonstrates schematically how this concept can be applied to reduce an evaluation response spectrum (e.g., a seismic margin earthquake response spectrum with high frequency energy content). An evaluation response spectrum is shown in Figure 3, below which a reduced spectrum is constructed. It is assumed here that the reduced response spectrum already has been determined and its adequacy is being checked. The reduced spectrum merges with the evaluation spectrum at low frequencies where no reduction is justified. The adequacy of the reduced response spectrum is demonstrated schematically for a component with a frequency, f . The top response spectrum shown in Figure 3 is the evaluation spectrum scaled to the yield level. For components with lower frequencies this upper response spectrum also corresponds to the ultimate level as discussed above (see Figure 2a). But at higher frequencies a reduced spectral value can be used to evaluate the adequacy of components as long as the ultimate capacity corresponding to the reduced spectral value is equal to the yield level obtained from the evaluation spectrum.

Note in Figure 3 that the ultimate S_a capacity shown is based on the reduced spectral ordinate and includes both the safety margin between the reduced spectrum and yield as well as the margin due to the additional ductile capacity (i.e., ductility scale factor). In contrast, the yield capacity shown is based on the original evaluation spectral ordinate and includes only the evaluation-to-yield safety margin. This is appropriate since at low frequencies, where there is no reduction, the total safety margin is due entirely to the ratio of the yield capacity to the evaluation capacity (i.e., the ductility scale factor is unity) as intended by the code writers.

As described in the next section non-linear time history analyses of models which represent cabinets anchored with welds were conducted to determine the additional ductility margin and to calibrate the properties of two simplified analytical methodologies developed to predict the response of equipment that could slide or rock. These methodologies can be used to determine the reduction factors to apply to an evaluation response spectrum without having to perform detailed time history analyses.

Figure 4 shows example reduced response spectra for a typical rock site UHS. The reduced spectra shown were developed using the simplified methodologies for the sliding and rocking models which are discussed in the next section. These models represent cases where the failure mode of a component is either sliding or rocking of the base which causes the connecting weld to fail. The purpose here is to concisely indicate the final results without the burden of explaining the details of the methodologies. This explanation is presented in the next section.

Two evaluation response spectra are shown in Figure 4: one anchored to 0.3 g pga and a second spectral shape anchored to 0.6 g pga. Below each evaluation spectrum are reduced spectra based on the simplified methodologies for sliding and rocking. In general, it was found that the sliding methodology reductions are less than the rocking methodology reductions. However, it is possible that the rocking case may control for some shapes of ground response spectra, particularly at higher frequencies

Figure 3 Basis for Determining Adequacy of Reduction Procedure

Figure 4 Example of Reduced Response Spectra for Rock Uniform Hazard Evaluation Spectrum Based on Models Using Simplified Methodology

Figure 5 Example Reduced Response Spectra for Rock UHS Anchored to 0.3 g PGA.

and high spectral input (see top case in Figure 4). Notice that the reduced spectra extend below the pga, and eventually they will become asymptotic to a "reduced pga" at a very high frequency (off the scale of Figure 4).

Three recommendations are made using the results from the work performed to generate Figure 4. First, the calculation to obtain the reduced response spectrum is stopped at a minimum frequency of 10 Hz and the reduced spectra are reconnected to the evaluation response spectrum at 8 Hz. Second, when developing time histories to be used to calculate in-structure response spectra the reduction factors should be based on a response spectrum equal to twice the evaluation ground response spectrum. In other words, the evaluation ground response spectrum should first be doubled, the reduced response spectrum obtained, and then the resulting reduced spectrum divided by two. The final response spectrum would then be used to develop time histories for generating in-structure spectra. For equipment mounted at grade the evaluation ground response spectrum can be used directly to determine the reduced response spectrum. Third, a minimum reduced spectral acceleration value should be utilized so that the reduced response spectrum can be based on only the simplified methodology for the sliding model. This will avoid the problem of the reduced spectra, based on the two models, crossing as shown at the top of Figure 4. The basis for these recommendations is discussed below.

From Figure 4 it is observed that the amount of reduction for the UHS response spectrum anchored to 0.6 g is less than the reduction for the spectrum anchored to 0.3 g. This occurs because the frequency shift caused by non-linear response in a weld of the same size is less at higher capacity levels. This implies that as an evaluation response spectrum is scaled up, the reduction factors go down. The fundamental natural frequencies of real nuclear power plant buildings are generally less than 10 Hz; hence, the reduction process will effect the response of the second and higher building modes. For practical situations it is expected that amplified in-structure response spectra at frequencies above 10 Hz will be less than twice the ground response spectra. This is the reason for the recommendation that in-structure response spectra should be based on input time histories developed using reduction factors obtained from a response spectrum equal to twice the evaluation ground response spectrum. Stopping the reduction at 10 Hz (and tapering to the evaluation spectrum at 8 Hz) will ensure that the reduction process does not propagate into the amplified spectral acceleration region near the fundamental frequency of the building.

In general, the reduced response spectrum to be used to develop in-structure response spectra should be based on reduction factors obtained from both the sliding and the rocking models. As a general rule, the evaluation spectrum should be scaled in small increments between a scale factor of 1 to 2, and the corresponding reduction factors determined for both models. The envelope of the resulting scale factors should be used to scale down the evaluation response spectrum in the high-frequency region. Both the sliding and rocking models also should be investigated to obtain the reduced response spectrum for use in analyzing equipment mounted at the ground. For the case of equipment mounted at the ground no scaling is required.

In lieu of this procedure a conservative alternate, that is easy to implement, can be used. Since both models are asymptotic to a common reduced p_{ga} , which is always higher than the spectral acceleration at which the curves for the two models cross (e.g., see Figure 4), the minimum value can be set at the asymptotic value. A conservative value for the asymptotic p_{ga} is always equal to the peak evaluation spectral acceleration obtained at 10 percent damping divided by a factor of 1.6. It is recommended that this value be used; although, slightly greater reductions can be obtained using the detailed incremental procedure described above.

It was found in the study that the design of welded anchorage leads to safety margins between the design and yield levels which vary depending on factors such as the design level, component mass distribution, coefficient of friction and model type (i.e., sliding or rocking). Following the procedure demonstrated in Figure 3 and discussed above higher safety margins always lead to lower spectral reduction factors. It was found for sliding models that the minimum safety margin for design is about 2.0, while the margin for rocking can be as low as 1.5 for practical cases where a component only rocks. In a seismic margin assessment (SMA) the capacity of a component is generally evaluated at the yield level. For this case the safety margin is always unity. As discussed above components up in a building should be evaluated for an in-structure response spectrum based on reduction factors obtained from a ground response spectrum equal to twice the evaluation response spectrum. This is equivalent to doubling the safety margin.

This implies for a SMA evaluation of existing equipment that a required safety margin, F_{SM} , of 1.0 should be used for components mounted at the ground and a value of 2.0 for components up in a building. Similarly, if the minimum margin for a weld in the design mode is 2.0, then the reduction factors should be based on a F_{SM} of 2.0 for components mounted at the ground and a value of 4.0 for components up in a building.

In summary, it is recommended that spectral reduction factors used to reduce an evaluation response spectrum for analyzing high-frequency components in a SMA be obtained using the simplified sliding model presented in the next section. As discussed below it is recommended that the fraction of mass at the base and the coefficient of friction both be set at zero. The following guidance is given when applying this recommendation:

1. The reduction should be performed for frequencies above 10 Hz, and the reduced response spectrum should be reconnected to the evaluation spectrum at 8 Hz.
2. The reduced response spectrum should not be reduced below a response spectrum value equal to the peak evaluation spectral acceleration at 10 percent damping divided by 1.6 unless additional cases are considered as discussed above.
3. In-structure response spectra should be based on a F_{SM} of 2.0. This is equivalent to using a F_{SM} of 1.0, but based on input time histories developed from reduced spectra obtained using a response

spectrum equal to twice the evaluation ground response spectrum. The resulting reduction factors are applied to the original ground response spectrum.

4. For equipment mounted at grade the evaluation ground response spectrum with a F_{SM} value of 1.0 can be used directly to determine the reduced response spectrum.

The analyst is cautioned, when performing a SMA evaluation using reduced response spectra as input, not to include a ductility factor in the SMA analysis for ductile components unless justified, since a ductility factor is already implied in the development of the reduced response spectrum. Any ductility factor assumed in a SMA must be an incremental factor which is justified in addition to the factor already embedded in the reduced evaluation ground response spectrum.

It is recommended that the spectral reduction factors used to reduce a design response spectrum follow the same steps given above for an evaluation ground response spectrum except that the safety margin, F_{SM} , must be increased. For example, it was found in the study that minimum margins of 2.0 and 1.5 correspond to pure sliding and rocking failure modes, respectively. These factors should be included in addition to the factor of 2.0 for developing in-structure response spectra.

Figure 5 shows example reduced ground response spectra for a rock site UHS anchored to 0.3 g pga based on the sliding model following the above recommendations. The lower and middle spectra are based on F_{SM} values of 1.0 and 2.0 and would be used for an SMA evaluation for equipment at the ground and for developing in-structure response spectra, respectively.

The top reduced spectrum is based on a F_{SM} value of 4.0. The middle and top spectra are examples of reduced spectra for use in design where the minimum design to yield margin is determined to be 2.0. The middle reduced response spectrum would be used for design of equipment at the ground and the top one would be used for developing in-structure design response spectra.

The shape of the reduced response spectra in Figure 5 are very similar to the type of ground response spectra used in the past to design and evaluate nuclear powerplants (e.g., R.G. 1.60 and NUREG/CR-0098 response spectral shapes). Thus, the UHS ground response spectra currently being obtained for Eastern U.S. sites have similar characteristics to traditional design spectra when the UHS are modified to have consistent safety margins.

SIMPLIFIED ANALYSIS METHODOLOGIES

The analytical procedure for obtaining the reduced response spectrum is described in this section. The derivations for the equations presented will be provided in Reference 1. Approximations to the equations are currently being developed to further simplify the analysis process. Note that procedures are presented for both the sliding and rocking models. It is recommended that the reduced response spectrum be based on these two models.

An electrical cabinet which is anchored at the base can respond in one of the following three general modes:

- Sliding
- Rocking
- Sliding and rocking

Only the sliding and rocking modes were investigated by non-linear time history analysis and the corresponding simplified methods. It was found that the sliding model generally produces larger reduction factors (i.e., conservative) than the rocking model. Both the sliding and rocking models load a welded anchorage in only one direction (i.e., either longitudinal or transverse). There is experimental data which provides a realistic basis for calculating the ultimate capacity of a fillet weld for a one-direction loading. In addition, a non-linear time history analysis of a combined sliding/rocking model is significantly more difficult to realistically perform. Based on the results of the sliding and rocking models it is believed that these models bound a model which permits both sliding and rocking response. On this basis it is recommended that the response spectrum reduction be obtained using these models.

Figures 6 and 7 show details for the rocking and sliding models, respectively. In both cases two-degree-of-freedom models were developed. This insures that both the first dynamic mode and the rigid body response of a cabinet are adequately modeled. In the Figure 6 sliding model the fraction of mass at the base, α , selected for the realistic calibration analyses was 0.20. This reflects the case where the component stiffness is essentially a shear beam for the first mode. This is appropriate for short squat components which would slide without rocking. In order to be realistic a coefficient of sliding friction, ϕ , at the base also was modeled. This turned out not to be a significant factor. Results with and without a coefficient of sliding friction and with α equal to 0 and 0.20 are given in the next section. In most of the calibration analyses that were conducted a ϕ of 0.4 was used, since this realistically represents the amount of friction which would be expected for steel on steel where no surface preparation is made.

The fraction of mass at the base, α , selected for the calibration analyses was 0.25 for the rocking model (see Figure 7). This reflects the case where the component stiffness is in between a shear and flexural beam behavior for the first mode (i.e., a straight line mode shape). This is appropriate for tall components which would rock without sliding. The upper mass was positioned at a distance equal to 2/3 the height of the cabinet. This correctly models the overturning moment for the first mode response. The component base width, w , was maintained in the rocking model and an incremental mass moment of inertia added to preserve overall rigid body rocking response. Note in the rocking model that two gap elements are provided at the base (one at each side of the model). These elements insure that the edges of the model do not cross the plane of the ground. This constrains the model to rock back and forth from edge to edge, which is the

Figure 6 Model for Component Sliding

Figure 7 Model for Component Rocking

response expected for a real cabinet which rock. Finally, a coefficient of restitution of unity was assumed for corner impact, which is conservative.

The properties of the fillet welds in shear are based on experimental studies by Lesik and Kennedy (6). Figure 8 shows normalized force-displacement curves for fillet welds loaded at various angles between 0 and 90 degrees. Note that 0 degrees corresponds to a longitudinal weld and 90 degrees corresponds to a transverse weld. From this figure it is seen that transverse welds fail at a much lower displacements than for longitudinal welds; however, the ultimate force capacity for transverse welds is larger. It was found that transverse welds are the most critical. The ultimate shear capacity, F_τ , for transverse fillet welds used in the analysis was assumed to be the following equation for a weld material with a nominal tensile strength, σ :

$$F_\tau = 0.75 \sigma d l \quad (1)$$

In this equation d is the weld leg size and l is the weld length. This is slightly on the conservative side compared to other weld test data.

In the non-linear analyses an equivalent elasto-plastic force-displacement diagram was used to model fillet weld inelastic behavior. Figure 9 shows the actual and elasto-plastic force deformation diagrams for a 3/16-inch transverse weld which is one inch long based on Reference 6. Note that the elasto-plastic yield level was selected to balance the area above and below the actual curve out to the ultimate displacement. The elasto-plastic yield capacity was found to be equal to 7.5 kips/inch, with a yield displacement of 0.001 inch and an ultimate displacement of 0.0105 inch for a weld with E60XX electrodes. For fillet welds with different leg sizes, the yield capacity and displacements can be scaled linearly with the leg size.

Most of the analyses were conducted using 3/16-inch fillet welds, which are the smallest practical size welds that are used in nuclear power plants. Smaller welds are used (e.g., 1/8-inch) when welding to the sides of cabinets with thin walls (in order to prevent burning through the metal). However, these cabinets have additional sources of flexibility which provide equivalent protection to high-frequency motions. For plants where minimum weld sizes larger than 3/16-inch welds are used, the evaluation response spectrum reduction factors should be based on the actual minimum weld size in order to obtain benefit for the larger displacement capacity. Otherwise, the reduction factors should be based on a 3/16-inch weld.

The simplified analytical methods for sliding and rocking are summarized in Tables 1 to 3 along with a macro flowchart for performing the analyses in Figure 10. The derivations of the equations and procedure will be documented in Reference 1. In addition, some simplifications will be given in this final report. The procedure for using the equations is discussed below. Table 1 gives the steps for the sliding model and Table 2 gives the steps for the rocking model to obtain the ductility scale factor needed to calculate the spectral reduction ratios. Both the simplified

Figure 8 Normalized Weld Curves (6)

Figure 9 Equivalent Elasto-Plastic Function For 3/16-Inch Weld Using E60XX Electrodes Loaded in Shear

Figure 10

Flow Chart for Calculating Response Spectrum Reduction Factor, Sa_r/Sa_e , Using Simplified Methodology for Sliding and Rocking Models

Table 1 Simplified Methodology Steps for Sliding Model

Step	Equation	Constant
<u>Normalized Effective Frequency</u> 1. Basic Model Parameter, A	$A = \frac{(2 \pi f_f)^2 \delta_u / g}{(\phi + C_u) / R_u}$ <p>where:</p> $f_f = \frac{1}{2 \pi} \sqrt{\frac{K_f}{(1 - \alpha)M}}$ $C_u = F_{SM} S_{a_r} R_y (1 - \alpha) - \phi$	K_f = Component fixed-base stiffness M = Component mass α = Fraction mass at base δ_u = Weld ultimate displacement capacity g = Acceleration due to gravity S_{a_r} = Reduced spectral acceleration F_{SM} = Required minimum safety margin.
2. Normalized Secant Frequency Squared, X	$X = \frac{2}{(1 + \cos \theta)(1 + A)}$ <p>where:</p> $\theta = \sin^{-1} \sqrt{2B}$ $B = \frac{2 \alpha A}{(1 + A)^2}$	R_u = Obtained from previous iteration (see Note 1) R_y = See note 2 ϕ = Coefficient of sliding friction
3. Normalized Effective Frequency Squared, X_e	$X_e = 1 - (1 - X)^a$	a = Empirical frequency constant (dimensionless). A value of 1.6 was used in study.

Table 1 Simplified Methodology Steps for Sliding Model

Step	Equation	Constant
<u>Effective Damping Ratio</u>		
4. Elastic Damping Ratio at Secant Frequency, β_{fp}	$\beta_{fp} = \beta_f \sqrt{\chi}$	β_f = Elastic damping ratio at fixed-base frequency
5. Hysteretic Damping Ratio at Secant Frequency, β_{Hl}	$\beta_{Hl} = b \left[\frac{2}{\pi} - \beta_{fp} \left(\frac{1 - G\Omega}{1 - G} \right) \right] \left(1 - \frac{\chi}{D} \right) (1 - G)$ <p>where: $G = \frac{Cu}{\mu_w(\phi + Cu)/R_u}$</p> $D = 1 + \left(\frac{\alpha}{1 - \alpha} \right) (1 - \chi)^2$ $\Omega = \frac{2}{\pi} \mu_w^2 \sin^{-1} \left(\frac{1}{\mu_w} \right) + \sqrt{(\mu_w - 1)(\mu_w + 1)}$	b = Empirical damping constant (dimensionless). A value of 0.3 was used in study $\mu_w = \frac{\delta_u}{\delta_y}$ δ_y = Weld yield displacement
6. Effective Damping Ratio at Effective Frequency, β_e	$\beta_e = \frac{\chi}{\chi_e} (\beta_{fp} + \beta_{Hl})$	

Table 1 Simplified Methodology Steps for Sliding Model

Step	Equation	Constant
<u>Ductility Scale Factor</u>		
7. Fraction of Total Mass Missing From Dynamic Model, F_M	$F_M = \frac{\alpha \chi^2}{1 + \left[\left(\frac{\alpha}{1 - \alpha} \right) (1 - \chi)^2 \right]}$	
8. Ductility Scale Factor, F_μ	$F_\mu = \left(\frac{Sa(f_f, \beta_f)}{Sa(f_e, \beta_e)} \right) \left(\frac{\chi_e}{\chi} \right) \left(\frac{1 - \alpha}{1 - F_M} \right) \left(\frac{R_y}{R_u} \right)$ <p>where: $f_e = f_f \sqrt{\chi_e}$</p>	$Sa(f, \beta)$ = spectral acceleration at frequency, f , and damping, β

Note:

- R_u determined from Table 3 using $f = f_e$ and $\beta = \beta_e$
(Note that R_u is assumed initially for the first trial and revised values are obtained from Table 3 for subsequent iterations.)
- R_y determined from Table 3 using $f = f_f$ and $\beta = \beta_f$

$$\frac{\chi}{\chi_e} = 1$$

$$F_M = \alpha$$

Table 2 Simplified Methodology Steps for Rocking Model

Step	Equation	Constant
<u>Normalized Effective Frequency</u>		
1. Basic Model Parameter, A	$A = \frac{(2 \pi f_f)^2 \delta_u (C_e + f_I)/g}{(0.5 + C_{u_t})/R_u}$ <p>where:</p> $f_f = \frac{1}{2\pi} \sqrt{\frac{K_f}{(1-\alpha)M}}$ $C_e = e^2 (1-\alpha)$ $e = \frac{\gamma h}{w}$ $f_I = 0.25 + \frac{\Delta M_o}{w^2 M}$	K_f = Component fixed-base stiffness M = Component mass α = Fraction mass at base δ_u = Weld ultimate displacement capacity g = Acceleration due to gravity C_{u_t} = $\frac{\text{Weld yield capacity* for truncated response spectrum}}{M g}$ R_u = See note 1 h = Component height w = Component width γ = Model height/component height ΔM_o = Incremental mass moment of inertia a = Empirical frequency constant (dimensionless). A value of 1.6 was used in study.
2. Normalized Secant Frequency Squared, X	$X = \frac{2}{(1 + \cos \theta)(1 + A)}$ <p>where</p> $\theta = \sin^{-1} \sqrt{2B}$ $B = \frac{2 \left(\frac{f_I}{C_e + f_I} \right) A}{(1 + A)^2}$	
3. Normalized Effective Frequency, X_e	$X_e = 1 - (1-X)^a$	
*For weld on <u>l-side</u> of model		

Table 2 Simplified Methodology Steps for Rocking Model

Step	Equation	Constant
<u>Effective Damping Ratio</u>		
4. Elastic Damping Ratio at Secant Frequency, β_{fp}	$\beta_{fp} = \beta_f \sqrt{X}$	β_f = Elastic damping ratio at fixed-base frequency
5. Hysteretic Damping Ratio at Secant Frequency, β_{Hl}	$\beta_{Hl} = b \left[\frac{2}{\pi} - \beta_{fp} \left(\frac{1 - G\Omega}{1 - G_p} \right) \right] \left(1 - \frac{X}{D} \right) (1 - G_p)$ <p>where</p> $G = \frac{Cu}{I_W(0.5 + Cu)/R_u}$ $G_p = \frac{1}{I_W}$ $D = 1 + \left(\frac{f_I}{C_e} \right) (1 - X)^2$ $\Omega = \frac{2}{\pi} I_W^2 \sin^{-1} \left(\frac{1}{I_W} \right) + \sqrt{(I_W - 1)(I_W + 1)}$	b = Empirical damping constant (dimensionless). A value of 0.3 was used in study. $\mu_W = \frac{\delta_u}{\delta_y}$ δ_y = Weld yield displacement
6. Effective Damping Ratio at Effective Frequency, β_e	$\beta_e = \frac{X}{X_e} (\beta_{fp} + \beta_{Hl})$	

Table 2 Simplified Methodology Steps for Rocking Model

Step	Equation	Constant
<u>Ductility Scale Factor</u> 7. Fraction of Total Mass Missing From Dynamic Mode, F_M	$F_M = \frac{-\left(\frac{f_I}{c_e}\right) X (1-X)}{D}$	
8. Ductility Scale Factor, F_μ	$F_\mu = \left(\frac{Sa(f_f, \beta_f)}{Sa(f_e, \beta_e)} \right) \left(\frac{x_e}{X} \right) \left(\frac{1}{1-F_M} \right) \left(\frac{1}{R_u} \right)$ <p>where: $f_e = f_f \sqrt{x_e}$</p>	$Sa(f, \beta)$ = spectral acceleration at frequency, f , and damping, β

Note:

1. R_u determined from Table 3 using $f = f_e$ and $\beta = \beta_e$. (Note that R_u is assumed initially for the first trial and revised values are obtained using Table 3 for the subsequent iterations. In general, R_u is close to unity and F_M is close to 0)

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Table 3 Determination of Ratio of Dynamic Mode Base Force to Total Base Force, R , at Frequency, f , and Damping Ratio, β (Used to Obtain R_u and R_y)

Frequency Range ¹	R	Equation
$f \leq f_p$	$R = R_p$	$R_p = \sqrt{1 + \left[\frac{F_M}{(1 - F_M)S} \right]^2}$
$f \geq f_h$	$R = R_h$	$R_h = 1 + \frac{F_M}{(1 - F_M)S}$
$f_p \leq f \leq f_h$	Sliding Model: $R = R_p + \frac{(f - f_p)}{(f_h - f_p)} (R_h - R_p)$ Rocking Model: $R = \sqrt{R_p R_h}$	where: $S = \frac{\frac{X}{X_e} Sa(f, \beta)}{a_g}$ a_g = Peak ground acceleration $F_M, X, X_e, Sa(,)$ = see Table 1 (Sliding) see Table 2 (Rocking)

Note 1

f_p : Frequency below which dynamic and rigid body responses are combined by SRSS (i.e., peak of response spectrum)

f_h : Frequency above which dynamic and rigid body responses are combined algebraically

sliding and rocking models contain theoretical and empirical parts. The basic formulation for each model (one for sliding and one for rocking) is based on parallel theoretical concepts. In each model an empirical frequency parameter, a , and an empirical damping parameter, b , were defined and calibrated using results from the non-linear time history analyses using the computer program DRAIN2D (Z).

For the calibration analyses a series of models were designed for a NRC R.G. 1.60 response spectrum anchored to 0.1 g for cabinets with fixed-base frequencies of 2, 5, 8, 13, 18, and 25 Hz and subjected to 15 different earthquakes. Figure 11 shows a plot of the 5-percent damped response spectra for the 15 earthquakes used in the analyses. The peak spectral ordinate for each record is scaled to 0.8 g in Figure 11. Note that significant spectral content is covered by the records between 2 and 25 Hz. High-frequency, low-frequency and broad-banded real earthquake time histories are included in the set of 15 records.

Realistic calibration analyses were conducted to find the sliding and rocking model capacities at the yield and ultimate levels. It was found that an "a" value of 1.6 and a "b" value of 0.3 fit both models very well. At the ultimate level the mean capacity ratio of the DRAIN2D results to the results from the simplified model was 1.02 and 0.98 with corresponding coefficients of variation of 0.11 and 0.16, respectively for the sliding and rocking models.

It should be noted that the theoretical model for rocking assumed a hysteretic loop which does not rigorously follow the actual model behavior. It was found that if a more realistic loop is used the results are significantly worse. Thus the results are based on the less rigorous hysteretic loop. The reader is cautioned not to extrapolate the rocking model beyond a weld displacement of about 0.1 inch without further verification.

Using an "a" value of 1.6 and a "b" value of 0.3 reduction factors can be obtained for the rocking and sliding models. The reduction factors, S_{a_r}/S_{a_e} , where S_{a_r} is the spectral acceleration corresponding to the reduced response spectrum and S_{a_e} is the spectral acceleration corresponding to the evaluation response spectrum (see Figure 3) are given by the following equation for both models:

$$\frac{S_{a_r}}{S_{a_e}} = \frac{1}{F_\mu} \quad (2)$$

where:

F_μ = Ductility scale factor obtained from the steps in Table 1 for sliding and Table 2 for rocking

The procedure for obtaining the reduction factors will become more apparent from an understanding of the process for performing the analysis, which is iterative since the reduced response spectrum S_{a_r} is not known a

Figure 11 Response Spectra for Earthquake Time Histories Used in Study Normalized to 0.8g Peak
5-Percent Damped Spectral Acceleration

priori. The procedure is explained as follows for determining the reduction factor at a single high frequency value. The process must be repeated for frequency values greater than 10 Hz to obtain the entire reduced response spectrum. The mass of the component is arbitrary, and any assumed mass will lead to the same results.

As shown in Figure 10 a trial value for the reduced response spectrum value, Sa_r , is assumed initially along with a value for R_u , which is the factor which accounts for the part of the response at the equipment base that comes from the dynamic response. It was found in the study that assuming initially that Sa_r is equal to Sa_e and R_u is equal to 1.0 is a reasonable strategy for selecting starting values.

Note that the total response is composed of dynamic and rigid body parts. The Final values for Sa_r and R_u are not known a priori; thus, an iterative process is required as indicated in Figure 10. The eight steps in Table 1 (sliding model) or Table 2 (rocking model) are followed until convergence occurs. Note that revised values for R_u are obtained after the 8 steps in the tables are completed before the next iteration. This is the reason that an initial value for R_u must be assumed. In general, R_u converges very quickly and final values for Sa_r usually can be obtained in three or four iterations. The steps in Tables 1 through 3 were easily programmed for the study using the computer program MATHCAD, Version 2.5 (8).

Insights into the methodology provide some intuitive understanding of how the methodology works. The equation for F_u from Tables 1 and 2 are repeated below for the special case when the fraction of mass at the base, α , and the coefficient of friction, ϕ , both are equal to zero. These are the values recommended for development of reduced response spectra. This case is equivalent to a one-degree-of-freedom model without sliding resistance.

$$F_u = \frac{Sa(f_f, \beta_f)}{Sa(f_e, \beta_e)} \frac{X_e}{X} \quad (3)$$

where: $X = (f_s/f_f)^2$

$X_e = (f_e/f_f)^2$

f_s = Secant frequency

f_e = Effective frequency

$Sa(f_f, \beta_f)$ = Spectral acceleration evaluated at fixed-base frequency, f_f , and component elastic damping ratio at fixed-base frequency, β_f

$Sa(f_e, \beta_e)$ = Spectral acceleration evaluated at effective frequency, f_e , and effective damping ratio, β_e

The equations for X are different for the sliding and rocking models. As an example the equation for X from Table 1 reduce to the following form for the sliding model:

$$X = \frac{1}{1 + \frac{(2\pi f_f)^2 \delta_u / g}{F_y / Wt}} \quad (4)$$

where:

δ_u = weld ultimate displacement

F_y = yield force in the weld

Wt = weight of the component

g = acceleration due to gravity

Figure 12 shows how Equation 4 for the normalized secant frequency squared X can be directly derived. From the force-displacement diagram in Figure 12 it is seen that the frequency of the model decreases as the displacement of the weld approaches its ultimate value (i.e., elastic stiffness decreases to secant stiffness). This shift in frequency is much greater for components with high initial fixed-base frequencies since the relative elastic displacement of the component stiffness element is small compared to the weld displacement. In contrast, for components with low fixed-base frequencies the stiffness element is so flexible that its displacement is large relative to the weld displacement. This leads to very little shift in frequency at the ultimate capacity.

The model responds with an effective frequency, f_e , with is somewhere in between the fixed-base and secant frequencies. Equation 3 is derived from an equivalent linear elastic one-degree-of-freedom model with an effective frequency that equates the displacement of the mass, δ'_u , at failure to the equivalent spectral displacement times the ductility scale factor:

$$\delta'_u = F_\mu \frac{Sa(f_e, \beta_e)}{(2\pi f_e)^2} \quad (5)$$

The normalized effective frequency squared X_e was obtained from the following equation by performing nonlinear time history analyses. The power term equal to 1.6 was a best fit for the model.

a. Model and Force-Displacement (F/δ) Diagrams

$$1. X = (f_s/f_f)^2$$

$$2. X = K_s/K_f$$

$$3. X = \frac{F_y/K_f}{F_y/K_f + \delta_u}$$

$$4. X = \frac{1}{1 + \frac{K_f \delta_u}{F_y}}$$

$$5. X = \frac{1}{1 + \frac{(2\pi f_f)^2 \delta_u / g}{F_y / Wt}}$$

b. Equations for Obtaining X

Figure 12 Derivation For Normalized Secant Frequency Squared, X , for One-Degree-of-Freedom Sliding Model Without Friction Resistance

$$X_e = 1 - (1 - X)^{1.6} \quad (6)$$

A good approximation for the effective damping term for the sliding model with α and ϕ equal to zero for a 3/16-inch weld based on the equations in Table 1 is:

$$\beta_e = \frac{X}{X_e} [X^{1/2} \beta_f + 0.173(1 - X)] \quad (7)$$

The effective damping term was also determined in the study which correlated the model with the results of the non-linear time history analyses.

The weld approaches failure as the component drifts to a displacement equal to the weld ultimate displacement plus the elastic displacement of the cabinet. The frequency decreases and the damping increases with increasing displacement. The spectral acceleration capacity, Sa_u , is given simply by the following equation:

$$Sa_u = \text{yield capacity} * F_\mu \quad (8)$$

As seen from equations 3 and 8 the ultimate capacity always increases due to the term X_e/X , which is independent of the earthquake response spectrum. The quantity X_e/X increases with increasing fixed-base frequency. This is due directly to the shift in frequency. The capacity may increase or decrease due to the ratio of the spectral ordinates, depending on whether the fixed-base or effective value is the largest. The spectral ratio is dependent on the earthquake record. However, the product of these two effects always produces an F_μ value which is equal to or greater than unity.

Note that the capacity is independent of spectral accelerations which are below the secant frequency. As a first approximation it is the spectral accelerations between the fixed-base and secant frequency which determine the capacity. Thus, any spectral content below the secant frequency has no effect on the response. This result indicates that response to earthquakes which have both high- and low-frequency energy does not depend on the lower frequency part of the response spectrum below the secant frequency.

Equation 3 can be used to indicate the basis for determining the asymptotic reduced pga which is recommended as a conservative lower bound to the reduced response spectrum based on the sliding model, in order to avoid having to use the rocking model (see Figure 4). As the fixed-base frequency, f_f , increases to very high values both the secant frequency, f_s , and effective frequency, f_e , approach constant values while the normalized ratios X and X_e approach zero.

From Equation 6 it can be seen that the ratio X_e/X approaches 1.6. Thus, combining Equations 2 and 3 with the limiting value for X_e/X the asymptotic reduced spectral acceleration, pga_r , becomes:

$$pga_r = \frac{Sa(f_e, \beta_e)}{1.6} \quad (9)$$

A conservative value for $Sa(f_e, \beta_e)$ is just the peak spectral acceleration at the damping ratio β_e . A conservative value for the damping ratio at the asymptote was found to be 10 percent which is consistent with Equation 7.

RESULTS

Example results using the simplified models are presented in this section to provide insight into the characteristics and magnitude of the reduction factors which can be expected using the simplified methodologies. The purpose here is to demonstrate general trends. Individual analyses should be conducted for a site-specific evaluation response spectrum which contains high-frequency energy content typical of EUS earthquakes as recommended above.

Figures 13 and 14 show the influence of weld size on the reduction factors for the sliding and rocking models, respectively. Note the reduced response spectra shown in this section are based on the evaluation spectrum directly for the design case and do not include the scale factor of 2 for developing in-structure response spectra or the limitation on the reduction to simplify the analysis as recommended above.

Greater reductions occur for larger weld sizes and for the rocking model. It is recommended that the reduction factors be based on a 3/16-inch weld size, unless it can be verified that the minimum weld used in a plant is larger. However, if larger minimum welds can be substantiated then the larger minimum weld size should be used as the basis for the reduction factors. Figure 13 indicates the magnitude of reduction which could be expected for this type of UHS for equipment mounted at the ground. Lower reductions would be obtained when developing in-structure response spectra.

Figures 15 and 16 show the effects of the fraction of mass at the base, α , and the coefficient of friction, ϕ . In Figure 15, which is for the sliding model, it is clear that the coefficient of friction has only a minor effect on the resulting reduction factors. Of more significance is the observation that higher values of mass at the base produce lower reductions. This suggests that using a value of α equal to 0 is non-conservative when developing reduction factors as recommended above for the sliding model. However, since the standard design process places the entire mass at the center of gravity times a spectral acceleration to obtain the design weld force, the recommendations follow exactly this design process. The lower reduction factors for α equal to 0.2 in Figure 15 assume that the designer would take advantage of the reduced mass in sizing the weld. Thus, he gets the benefit of smaller design forces, but must use a higher input spectrum (i.e., less reduction) which is consistent.

If a value of α equal to 0 is used to obtain reduced design spectra and it turns out after the design is completed and the weld sized that it is more realistic for the mass to be placed partially at the base, the overall

Figure 13 Example Reduced Response Spectra for Rock Uniform Hazard Evaluation Spectrum Based on Sliding Model and Simplified Methodology for Different Weld Sizes

Figure 14 Example Reduced Response Spectra for Rock Uniform Hazard Evaluation Spectrum Based on Rocking Model and Simplified Methodology for Different Weld Sizes

Figure 15 Example Reduced Response Spectra for Rock Uniform Hazard Evaluation Spectrum Using Sliding Model and Simplified Methodology for Combinations of Mass Ratio, α , and Coefficient Friction, ϕ

Figure 16 Example Reduced Response Spectra for Rock Uniform Hazard Evaluation Spectrum Using Rocking Model and Simplified Methodology for Different Mass Ratios, α

factor of safety will only increase. This is because the minimum factor of safety was used to develop the reduction factors in the first place. Also, it is intuitive that if a portion of the mass is in reality at the base this is conservative, since the amplified spectral accelerations are applied to a smaller mass up above.

The curves for the rocking model in Figure 16 follow the opposite trend. The higher the amount of mass at the base the greater the reduction. Thus, using a value of α equal to zero is always conservative, no matter whether the designer splits the mass or not. To be consistent with the recommendations for the sliding model a value of α equal to 0 is also recommended for the rocking model. This is not a significant limitation since the sliding model usually always controls.

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Appendix B

RECOMMENDED PROCEDURES TO ADDRESS
HIGH-FREQUENCY GROUND MOTIONS IN SEISMIC MARGIN ASSESSMENT
FOR SEVERE ACCIDENT POLICY RESOLUTION

by

John W. Reed
Robert P. Kennedy
Robert P. Kassawara

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Section 1

INTRODUCTION

The purpose of this appendix is to provide recommendations for addressing high-frequency seismic motions in a Seismic Margin Assessment (SMA) for Severe Accident Policy resolution. A research program to address high-frequency seismic motions was performed by the Electric Power Research Institute (EPRI) to provide the basis for the recommendations (1). An analytical investigation was conducted, and a series of tests are planned by EPRI in the near future.

The recommendations for addressing high-frequency motions are made in the context that the SMA procedures (2) are acceptable to both industry and the NRC for seismic capacity reviews of nuclear power plants and that only minor changes are needed for the effects of high frequencies. It is recommended that the SMA screening guidelines for structures and equipment be keyed to response spectrum parameters, not to the peak ground acceleration (PGA). Also, it is recommended that the input ground response spectrum to be used in a SMA be modified in the high-frequency region to reflect the lack of damageability from motions in this range. Recommendations for electrical functionality failure modes (e.g., relay chatter) for relays and other electrical components, which may be sensitive to motions with frequencies greater than about 16 hz, are also provided.

It is important to recognize that the recommendations made here for structures and components reflect the ability of the supporting structural elements to resist seismic motions in a ductile fashion. Although some elements (e.g., welds) are brittle relative to low-frequency damaging seismic motions, they have some ductile capacity to resist the small displacements associated with high-frequency motions. Separate recommendations are also given for electrical functionality failure modes, which are acceleration sensitive and do not have ductile capability.

Section 2 of this appendix gives the background on the high-frequency seismic motion issue. In Section 3 recommendations are given for modifying the SMA procedures to include the effects of high-frequency seismic loads. Finally, Section 4 lists the references used in this appendix.

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Section 2

BACKGROUND ON HIGH-FREQUENCY SEISMIC MOTIONS

The results of recent seismic hazard analyses conducted for the eastern U.S. (EUS) indicate that uniform hazard spectra (UHS) have relatively large high-frequency spectral acceleration content above approximately 10 Hz. This is in contrast to typical plant design or seismic margins-type input, such as the NRC Regulatory Guide (R.G.) 1.60 or NUREG/CR-0098 response spectra. However, these same UHS have significantly lower spectral acceleration content at the low-frequency end of the response spectrum (i.e., between about 1 and 10 Hz), which indicates to the seismic engineering community that these motions are less damaging than traditional design or evaluation-type input. Figure B-1 shows the relationship between the median NUREG/CR-0098 response spectrum anchored to 0.3 g peak ground acceleration (pga) and an example UHS, with a 10,000 year return period (at the 0.85 fractile), from a hazard analysis using the EPRI methodology for a EUS nuclear power plant site.

In the analysis and design of nuclear power plant structures it is widely felt that to be damaging, earthquakes must be rich in spectral content in the 1 to 4 Hz frequency range. Because most nuclear power plant structures and equipment have fundamental frequencies significantly above 1 Hz, strong amplified motions below 1 Hz caused by soft soil conditions, although potentially damaging to flexible structures such as high-rise buildings, do not affect typical nuclear power plant sites. Thus for stiff nuclear power plant structures at typical soil or rock sites it is necessary for energy to be present in the 1 to 4 Hz frequency range for earthquakes to be potentially damaging. It is believed that the damaging frequency range for equipment probably is slightly higher, but less than 10 Hz. A possible exception is anchorage of equipment at high ground motion levels, but for this case spectral displacement, not acceleration, is more important in defining damage potential. As shown in Figure B-1 the NUREG/CR-0098 spectral shape has damage-potential characteristics but the example UHS does not.

It has also been observed that damage correlates well with elastic spectral acceleration in the 1 to 4 Hz range. Thus, it is likely that it is an "effective" frequency in this range as opposed to the fundamental frequency which is important

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Figure B-1. Comparison of example uniform hazard response spectrum with NUREG/CR-0098 response spectrum (5 percent damped).

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to significant damage. These results support the conclusion that it is the low-frequency content of earthquake records which causes damage.

In a recent detailed study of the Diablo Canyon Turbine Building, conducted for the Diablo Canyon seismic probabilistic risk assessment (PRA), 25 earthquake records were input to a nonlinear building model where the model material properties were varied in a probabilistically consistent manner (3). Several hundred deterministic analyses were conducted in this study, which spanned all realistic combinations of structural and ground motion parameters. It was found that for a ground motion to be damaging it must have broad frequency content. Specifically, high spectral accelerations were required at the elastic frequencies for those dynamic modes corresponding to the relatively weak building shear walls (i.e., approximately 8 Hz) to initiate substantial inelastic drift. In addition, high spectral acceleration content also was required in the approximately 2 Hz range in order to drive the shear walls to drift levels associated with the onset of severe distress (i.e., significant damage).

One example of a high-frequency earthquake which occurred near a nuclear facility was the Leroy earthquake (magnitude M_L 5.0) which took place January 31, 1986 about 11 miles from the Perry Nuclear Power Plant (4). At the time of the event Cleveland Electric Illuminating Company was within days of core load and receipt of their 5 percent power license for Perry. The staff observed no indications of damage to systems in operation at the time, which was confirmed by follow-up inspections. However, after the earthquake the analysis of the motion records indicated that the Operating Basis Earthquake (OBE) ground response spectrum had been exceeded at frequencies above 10 Hz, and the Safe Shutdown Earthquake (SSE) response spectrum had been exceeded above 15 Hz. The maximum acceleration of the ground motion at the foundation level was 0.18 g with a corresponding peak 5 percent damped spectral acceleration of about 0.8 g; however, the peak ground velocity was only 0.9 inches per second, and the maximum ground displacement was only 0.06 inches. The latter two values indicate low energy content in the 1 to 4 Hz frequency range; thus, it is not surprising that no damage occurred. In addition, the cumulative absolute velocity (CAV) value for this event was only 0.08 g-sec which is much lower than the OBE exceedance criterion limit of 0.30 g-sec (4).

Experience with high-frequency motions from other environments indicate that they are much less damaging than their low-frequency counterparts. In the development of a criterion for determining the exceedance of the OBE, earthquake experience

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concerning the damaging characteristics of high-frequency motions was collected and documented (4). Based on a review of ground motions caused by conventional high explosive blasts, and their effect on structures, it was concluded that no damage will occur to engineered structures and equipment for short-duration ground motions with spectral accelerations below the envelope of the OBE response spectrum and a threshold cracking spectrum (established from blast data) as shown in Figure B-2. However, it has not been established whether these conclusions can be extrapolated to longer duration large-magnitude high-frequency events. The conclusions that damage is not caused by high-frequency motions is also supported by fragility data for ductile equipment, performance of equipment under industrial environment vibrations and code requirements for high-frequency loading.

Based on past experience, both from earthquakes and other physical phenomena which produce high-frequency motions, it is believed that high-frequency input, which is a predominant part of UHS, as shown in Figure B-1, will not be damaging to ductile structures and equipment. However, there is still a potential concern that these motions may be an issue for acceleration sensitive equipment and relays, contactors, motor starters and switches. For this category of components the effects of high-frequency motions should be addressed only if it is determined from a systems perspective that functional failure (e.g., relay chatter) during strong motion is detrimental or equally that the safety function affected can not be confidently recovered after an earthquake.

In order to evaluate the capacity of relays, high-frequency input at the floor level is required. The difficulty in analyzing acceleration-sensitive components for high-frequency input is in the development of realistic floor response spectra. The number of finite elements and mass nodes used in traditional dynamic analysis of nuclear power plant structures is generally not large enough to properly model the effects of high-frequency input. On one hand, using the current models, the high-frequency motions will be filtered out giving the incorrect impression that high-frequency response at the various floor levels does not exist. On the other hand, if models are constructed which properly pass the high-frequency motions through the structure, the analysis cost would be prohibitive. Based on past experience high-frequency motions do propagate through structures. The earthquake near the Perry plant demonstrated this to be true. However, there is evidence to suggest that high-frequency motions are not significantly amplified.

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Figure B-2. Comparison of R.G. 1.60 ground response spectrum to the threshold cracking spectrum developed from ground motion due to blast (3).

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It is believed that high-frequency ground motions which have dynamic characteristics consistent with the EUS UHS in the range of about 10,000 year return period will be much less damaging than low-frequency earthquakes with the same peak spectral acceleration. These latter ground motions traditionally have been the basis for seismic analysis and design (e.g., Olympia, Taft and El Centro No. 5).

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Section 3

RECOMMENDED PROCEDURES

Conservative recommendations for incorporating the effects of high-frequency seismic motions in SMA are provided in this section. These recommendations reflect the current data and engineering understanding of high-frequency ground motions that might occur at nuclear power plant sites in the central and eastern United States.

The procedures described in References 2 and 5 for performing a seismic margin assessment do not require major changes for the consideration of high-frequency seismic motions, and these procedures are incorporated here by reference. The following modifications are recommended to the SMA procedures:

- The seismic capacity screening guidelines, which are keyed in References 2 and 5 to peak free-field ground acceleration values (i.e., 0.3 g and 0.5 g), are referenced to response spectral parameters: i.e., spectral acceleration (S_a) and spectral velocity (S_v).
- Input ground response spectra used in a SMA should be modified at high frequencies to reflect the lack of damageability to safety-related nuclear power plant components.
- For relays that are sensitive to input motions above 16 hz special considerations are recommended.

A discussion of the three recommended modifications and their supporting bases are given in the following subsections.

RESPONSE SPECTRUM PARAMETER LIMITS FOR SEISMIC CAPACITY SCREENING GUIDELINES

The seismic capacity screening guidelines used in SMA are keyed to peak free-field ground accelerations (2 and 5). Three categories are provided: when the PGA corresponds to less than 0.3 g, 0.3 g to 0.5 g and greater than 0.5 g. For each of the three categories, requirements are given for different structural and equipment classes. For example if the PGA is less than 0.3 g, concrete containments do not need to be evaluated, but masonry walls do. Detailed screening tables are given in Reference 2 which extend the tables originally developed in Reference 5.

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Consistent with the level of conservatism provided in the screening guidelines in References 2 and 5, Figure B-3 shows the corresponding spectral acceleration and spectral velocity limits which should be used with the SMA screening guidelines for earthquakes with high-frequency motions. The lower curve which consists of a S_a limit of 0.8 g and a S_v limit of 20 in/sec corresponds to the 0.3 g PGA value. Similarly, the top curve which has a 1.2 g S_a limit and a S_v limit of 30 in/sec corresponds to the 0.5 g PGA value.

When making a comparison using the screening guideline limits, the amplified portion of the input ground response spectrum in the high-frequency region should be compared to the S_a limits. Note that the check in the high-frequency region only is made with the spectral acceleration limits, not with the PGA values. Since damage is ultimately related to strain, or equivalently displacement, the PGA check is not required.

The basis for the response spectrum limits shown in Figure B-3 has its roots in the original development of the SMA screening guidelines, where the first two authors of this appendix were principal contributors (5). At that time the Seismic Qualification Utility Group (SQUG) had formed in response to the NRC issue designated as: "Seismic Qualification of Equipment in Operating Plants," which is also referred to as Unresolved Safety Issue (USI) A-46.

As SQUG began evaluating the earthquake experience data this information also was available to the authors (referred to as the Expert Panel) of Reference 5. Ultimately, the Senior Seismic Review and Advisory Panel (SSRAP) endorsed the Reference Spectrum given in Reference 6 (which is controlled by S_a equal to 1.2 g) and the Bounding Spectrum (which is controlled by S_a equal to 0.8 g). The Bounding Spectrum, which is a factor of 1.5 below the Reference Spectrum, is currently being used by SQUG in the evaluation of safety-related equipment in existing nuclear power plants. Note that the 1.5 factor reflects conservatism in using the experience data base and for the possibility that floor response spectra in nuclear power plants might be amplified more than motions in the data base buildings (6).

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Figure 8-3. Spectral parameter limits for SMA seismic capacity screening guidelines.

The equipment covered by the SMA screening guidelines essentially corresponds to the SQUG equipment. Structures are included in the SMA review which are not included in the A-46 program. However, nuclear power plant structures are rugged, and the guidance given in the SMA screening tables conservatively reflect their capacities. Moreover, a walkdown will be conducted in both SQUG and SMA reviews and there are important caveats and exclusions that must be followed in order to use the screening guidelines, both of which ensure conservative assessments.

The 1.2 g S_a limit used in the SMA screening guidelines is used to designate capacities for equipment in the 0.8 g S_a to 1.2 g S_a category and for the greater than 1.2 g S_a category. Although the SQUG guidelines do not include multiple categories, the SMA guidelines for the two ranges are consistent with the earthquake experience data base, which corresponds to the higher Reference Spectrum. Thus, the upper curve in Figure B-3 reflects the philosophy of the NRC Expert Panel and is appropriate for delineating the two higher SMA screening categories.

At the lower frequency end of the spectral capacity curves shown in Figure B-3 the spectral acceleration limits change to spectral velocity limits at about 2.5 hz,

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which again correspond directly with the Bounding and Reference Spectra. The lower frequency limits of the spectra in Figure B-3 are about 1.5 hz, which is believed to be at about the practical frequency limit of nuclear power plant structures and equipment on soil sites.

The experience data base, upon which the SQUG program derives nuclear power plant equipment capacities, is tied to the earthquake characteristics at the data base sites. Thus, the Bounding (and Reference) Spectrum roll off at frequencies above 7.5 hz as shown by the spectral curve in Figure B-4. However, at the upper frequency end of the limit spectra shown in Figure B-3 the spectral acceleration values at frequencies higher than 7.5 hz are projected flat at the S_a limits of 0.8 g and 1.2 g corresponding to the two curves. The limitation on the Reference and Bounding Spectra is not the equipment capacities per se, but rather the frequency characteristics of the data base earthquakes, which are western North American events that may be less rich in high-frequency content.

Extending the spectral acceleration limits at constant levels for frequencies beyond 7.5 hz is justified based on an understanding of the potential failure characteristics of structures and equipment at nuclear power plants. For critical components (i.e., safety-related components that have structural properties in the range that require seismic evaluation) the fundamental frequencies are generally less than about 10 hz with a few classes of equipment with fundamental frequencies up around 15 hz. Some classes of equipment have higher fundamental frequencies, but their elastic strength is also relatively high (e.g., check valves and well-anchored heat exchangers). Thus for most components the high-frequency earthquake motions exceed their fundamental frequencies.

As discussed in Reference 1 most components in nuclear power plants are very strong. If components have high fundamental frequencies (i.e., relatively stiff compared to their mass) they also have high strength, since stiffness and strength are correlated. In addition to being strong most high-frequency components in nuclear plants also have ample capacity to resist the small displacements associated with high-frequency motions. Examples include the casing on a vertical pump motor and the exterior shell on a small heat exchanger. These types of components are also ductile, and small displacements associated with high-frequency motions can be safely accommodated if the yield capacity is exceeded.

In Reference 1 it is concluded that the potentially limiting case for high frequency components is welded equipment anchorage where the welds are sized

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Figure B-4. Extension of SQUG bounding spectrum into high-frequency region.

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exactly to met the seismic demand. Note that anchorage is not included in the SMA screening tables and must be considered separately in addition to the screening table decisions for the equipment per se. Thus, the extension of the spectral limits into the high-frequency region is for functionality and structural issues covered in the screening tables, not for equipment anchorage.

In general, the capacity of a component is either controlled by an elastic displacement limit (e.g., the gap between a fan and its housing) or a strain limit where the component goes into the inelastic range (e.g., bending of a steel support). At the top of Figure B-4 an idealized capacity curve is shown above the recommended capacity spectrum, which corresponds to the $0.8 g S_a$ value. It is assumed here that the fundamental resonance frequency is to the left of the left end of the true capacity curve. This capacity spectrum increases as the frequency of input increases. Figure B-5 shows example real fragility curves for a pressure control valve from Reference 7. Note that the capacity in Figure B-5 increases with frequency in a manner similar to the idealized curve at the top in Figure B-4.

The results of the high-frequency analytical study reported in Reference 1 demonstrate that the capacity beyond yield also increases at higher frequencies. In general, if components are designed to yield at a given earthquake level the ratio of the earthquake input at failure to the earthquake input at yield increases as the component frequency is increased. This is observed in detailed nonlinear time history analyses and also is explained by pseudo-linear elastic models which fit the time history results. The principal reason for this increased capacity is due to the shift in frequency from the elastic case to the equivalent frequency at failure. A greater shift occurs as the component frequency is increased, which leads to a greater margin from yield to failure.

It is expected that the capacity beyond 7.5 hz should increase as indicated in Figure B-4. For either an elastic or inelastic type of failure model the S_a capacity increases with frequency, and the assumption that the capacity is constant (i.e., flat as shown in Figure B-3) for frequencies above 7.5 hz is conservative.

There are some limitations to these arguments. The principal issue is when the fundamental frequency of the component under consideration occurs at a frequency higher than the upper bound frequency at which the capacity has been established from experience data. For this case there may be a dip in the capacity versus

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Figure B-5. 2 percent damped pressure control valve capacity spectra (Z).

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frequency curve in the vicinity of the fundamental frequency, and the shape of the capacity curve would be different than shown in Figure B-5. However, it is unlikely (anchorage aside) that high-frequency components exist that either do not have high yield capacities, or which do not have high displacement capability between yield and failure which can accommodate the small displacements associated with high-frequency earthquakes. Experience from other environments supports the conservative recommendations. Based on the work in References 1 to 4, strong evidence is found that high-frequency seismic motions will not be damaging to nuclear power plant components. Again these results emphasize that it is the seismic motions in the low frequency range (i.e., approximately 1 to 4 hz) that potentially can cause damage to nuclear power plant components.

INPUT RESPONSE SPECTRUM REDUCTION AT HIGH-FREQUENCIES

Reference 1 provides a procedure and technical basis for reducing ground-level response spectra at high frequencies for the purposes of analyzing components which have not been screened out and for equipment anchorage. For acceleration sensitive components such as relays the unmodified ground response spectra must be used in the evaluation. Consideration of electrical functionality failures is discussed the last section of this appendix.

In resolving severe accident issues a probabilistically-based required level earthquake (RLE) is recommended to be used as discussed in Appendix E. The RLE for a EUS site generally will be rich in high-frequency content. Because of the increased capacity for ductile components as the component fundamental frequency increases the RLE can be reduced at high frequencies. This follows in order to be risk consistent with the demand-to-capacity ratios for low-frequency components, where the ultimate capacities are essentially at the yield level. It is recommended that the RLE ground response spectrum be modified to a review level ground motion (RLGM) response spectrum following the procedures given in Reference 1 as summarized below.

Note that in screening components the RLE should be compared to the screening spectral limits shown in Figure B-3. This avoids double counting the benefits of ductility that are used in the arguments for extending the 0.8 g and 1.2 g spectral limits into the high-frequency region. The same arguments are also used for decreasing the RLE, as presented in Reference 1. However, once the screening step is completed components which are not screened out should be analyzed using the modified RLGM spectrum.

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Figure B-6 shows an example RLE for a rock site and two RLGM spectra. The higher RLGM response spectrum should be used to generate in-structure response spectra for analysis of equipment up high in a building, and the lower RLGM can be used to analysis equipment which is supported at the ground level. The higher RLGM spectrum is required because of amplification of the ground motion at higher elevations.

It is recommended that spectral reduction factors used to reduce a RLE response spectrum to obtain a RLGM response spectrum for analyzing high-frequency components in a SMA be obtained using the simplified sliding model presented in Reference 1. It is recommended that the fraction of mass at the model base and the coefficient of friction both be set at zero. Figure B-7 shows the model for component sliding from Reference 1. The following guidance is given when applying these recommendations:

1. The reduction should be performed for frequencies above 10 Hz, and the reduced response spectrum should be reconnected to the RLE evaluation spectrum at 8 Hz.
2. The reduced response spectrum should not be reduced below a response spectrum value equal to the peak evaluation spectral acceleration at 10 percent damping divided by 1.6 unless additional cases are considered as discussed in Reference 1.
3. The RLGM to be used to generate in-structure response spectra should be based on a safety factor, F_{SM} , of 2.0. This is equivalent to using a F_{SM} of 1.0, but based on a response spectrum equal to twice the RLE ground response spectrum. The resulting reduction factors are applied to the original ground response spectrum.
4. For equipment mounted at grade the evaluation ground response spectrum with a F_{SM} value of 1.0 (i.e., using the RLE without scaling) can be used directly to determine the reduced response spectrum.

A minimum weld size of 3/16-inch weld should be used to develop the reduction factors. However, for plants where minimum weld sizes larger than 3/16-inch welds are used, the response spectrum reduction factors should be based on the actual minimum weld size in order to obtain benefit for the larger displacement capacity. Otherwise, the reduction factors should be based on a 3/16-inch weld.

The analyst is cautioned, when performing a SMA evaluation using reduced response spectra as input, not to include a ductility factor in the SMA analysis for ductile components unless justified, since a ductility factor is already implied in the development of the reduced response spectrum. Any additional ductility

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Figure B-6 Example Reduced Response Spectra for Rock UHS Anchored to 0.3 g PGA.

Figure B-7. Model for Component Sliding

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factor assumed in a SMA must be an incremental factor which is justified in addition to the factor already embedded in the reduced RLGM ground response spectrum.

Figure B-6 shows example reduced ground response spectra for a rock site UHS anchored to 0.3 g pga based on the sliding model following the above recommendations. The two spectra are based on F_{SM} values of 1.0 and 2.0 and would be used for an SMA evaluation for equipment at the ground and for developing in-structure response spectra, respectively.

The shape of the reduced response spectra in Figure B-6 are very similar to the type of ground response spectra used in the past to design and evaluate nuclear power plants (e.g., R.G. 1.60 and NUREG/CR-0098 response spectral shapes). Thus, the UHS ground response spectra currently being obtained for Eastern U.S. sites have similar characteristics to traditional design spectra when the UHS are modified to have consistent safety margins.

An intuitive justification for the recommended reductions is provided by examining displacement capacities of equipment anchorages, which are the critical structural links. Figures B-8 through B-12 show force-deflection curves for single-direction loading for fillet welds, wedge-type expansion anchors and cast-in-place concrete inserts. As can be seen from these figures welds have the minimum displacement capacity.

Figure B-8. Normalized Weld Curves (9)

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Figure B-9. Example force-displacement curve for 3/4 in. wedge type anchor bolt in tension, $f'_c=3700$ psi (concrete spalling failure mode) (9).

Figure B-10. Example force-displacement curve for 3/4 in. wedge type anchor bolt in shear, $f'_c=3700$ psi (shear failure through threads) (9).

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Figure B-11. Example force-displacement curve for 1 in. Richmond insert in tension, $f'_c=2850$ psi (10).

Figure B-12. Example force-displacement curve for 1 in. Richmond insert in shear, $f'_c=2850$ psi (10).

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In Fig. B-8 the fillet weld data in the transverse direction (i.e., θ equal to 90°) has a minimum displacement capacity. However, compared to the longitudinal direction weld (i.e., θ equal to 0°), which has a high ductility limit but lower strength capacity, the transverse direction weld strength is approximately 40 percent stronger. This extra strength in the transverse direction offsets its lack of ductility. The recommended reduction factors are based on the smallest practical weld size, which is conservative.

Note that typical high-frequency motions are small. For example, a 1 g acceleration at 10 hz is only about a 0.1 inch displacement. Ductility is relative, depending on the frequency of the input motion. For low-frequency damaging motions the displacements are large and the inelastic response of welds as shown in Figure B-8 does not provide ductile capacity. In contrast, at higher frequencies where the displacements are small, the ductility characteristics of weld material contribute significantly to the total strength and must be considered in determining component capacity. Note that in both the low and high-frequency cases the ultimate capacity is assumed to occur at the same displacement.

ELECTRICAL FUNCTIONALITY FAILURE MODES

For relays and other electrical components which are acceleration sensitive, the procedures given in Reference 2 for addressing functionality concerns are adequate when the component frequency is less than about 16 hz because cabinet amplification factors can be confidently estimated. For these cases no new requirements are needed and the SMA procedures are adequate (1). Care should be exercised in developing realistic floor response spectra up to 16 hertz which reflect the true high-frequency content of the ground motion (i.e., without the reduction procedure recommended above for ductile and displacement sensitive components).

For relays and other electrical components which are acceleration sensitive, and are sensitive to frequencies in excess of 16 hz, alternate procedures are recommended. Strategies other than direct evaluation should be attempted before trying to qualify the relays per se, for the seismic floor motion. Recently a list of relays that are sensitive to high-frequency low-level seismic motions has been developed (11). These relays should be avoided if at all possible. Other high capacity relays also will be identified in the near future by SQUG and EPRI which should be used. The recent relay functionality guidelines developed by EPRI can be used to determine whether chatter is acceptable (11). The following

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approaches are recommended:

- If relay chatter occurs causing an unacceptable change of state, determine whether the plant operators can recover the required plant operating condition in a relatively short period of time after the earthquake.
- If necessary, modify the electrical circuitry to bypass the dependance on the critical relay in question.
- Replace the relay in question with a rugged model which can be easily qualified (e.g., critical frequency less than 16 hz).

If these options are not feasible then a direct capacity check of the candidate relay will be required. For this case realistic floor response spectra based on the unmodified ground response spectrum which span the frequencies of interest will have to be estimated. Next, a cabinet-level capacity spectrum should be developed which includes the critical relay mounted in the cabinet. This step is necessary since at high frequencies (i.e., greater than about 16 hz) motion amplification factors from the floor to the mounting point of the relay device are highly variable and a single value that can be used in SMA can not be confidently determined. It is likely that a shake-table test will have to be performed to develop the required cabinet-level capacity spectrum. Using this capacity spectrum, the calculated floor response spectrum can be compared in the standard way to evaluate the relay.

SUMMARY

In summary, the capacity screening guidelines have been defined in terms of spectral acceleration and velocity limits (see Fig. B-3). In performing a seismic evaluation for ductile components the input ground response spectrum should be modified as demonstrated in Figure B-6 in order to reduce the portion of the response spectrum which is not potentially damaging to ductile components. For evaluation of relay chatter and other functionality failure modes the unaltered response spectrum should be used. Care should be exercised in developing floor response spectra for relay chatter evaluation in order to properly transmit the high-frequency motion to the floor level.

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Section 4

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