Attachment 1

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
	)	
TEXAS UTILITIES ELECTRIC COMPANY, et al.	)	Docket Nos. 50-445 and 50-446
	)	
(Comanche Peak Steam Electric Station, Units 1 and 2)	)	(Application for Operating Licenses)

AFFIDAVIT OF J. C. FINNERAN, R. C. IOTTI AND R. D. WHEATON REGARDING SAFETY FACTORS

We, Robert C. Iotti, John C. Finneran and Randall D. Wheaton, being first duly sworn hereby depose and state as follows:

(Finneran). I am the Pipe Support Engineer for the Pipe Support Engineering Group at Comanche Peak Steam Electric Station ("CPSES"). In this position, I oversee the design work of all pipe support design organizations for Comanche Peak. I have previously provided testimony in this proceeding. A statement of my professional and educational qualifications was received into evidence as Applicants' Exhibit 142B.

(Iotti). I am employed by Ebasco Services, Inc. as Chief Engineer of Applied Physics. In this position I am responsible for directing analytical and sometimes design work in diverse technical areas, including analysis of the response of structures, systems, and components to various normal and abnormal conditions. I have been engaged by TUECO to coordinate and over-

see the technical activities performed to respond to the Board's Memorandum and Order of December 23, 1983. A statement of my educational and professional qualifications is attached to Applicants' letter of May 16, 1984, to the Licensing Board.

(Wheaton). I am employed by the IMPELL Corporation as the Staff Consultant for the Company's Advanced Engineering Division. In this position I am responsible for the technical direction of all seismic related work and research and development activities in the area of earthquake engineering. I have been asked by TUECO to evaluate the margins of safety which are inherent in the design of the Comanche Peak Steam Electric Station's structures, systems and components and in particular piping and piping support systems and components. A statement of my educational and professional qualifications is attached hereto as Attachment 1.

I. INTRODUCTION AND OVERVIEW

The purpose of this affidavit is to respond to the concerns raised by CASE in their Proposed Findings of Fact and Conclusions of Law, principally in Section I, that industry practice (followed at CPSES) of not expressly factoring small, potential loads (which are individually not significant) into design calculations is not supported by adequate design margin (factors of safety). (See, e.g., CASE's Proposed Findings at I-6, 7).

Safety factors inherent in design stem from three distinct categories. CASE, in its Findings of Fact at Section I, only focuses on a portion of one of those categories, i.e., the factor

of safety associated with margins inherent in the ASME or AISC Code requirements. (see, e.g., CASE's Proposed Findings at I-10, 13, 14). CASE maintains that these margins are on the order of 1 4 and "less than the factors of safety for the AISC code used to design warehouses." Id. at I-15. While we believe that this statement is clearly erroneous, in any event, the other two categories of safety factors are just as, if not more important. If the one category just mentioned (including code margins raised by CASE) is termed "capacity" safety factors, then the other two categories may be called "design input definition" and "method of analyses" caused safety factors. It is the cumulative effect of safety factors in all three categories that supports industry practice of not expressly factoring small, potential load contributors (e.g., gap effects and self excitation of supports) into design calculations, i.e., such effects are more than compensated for by the cumulative margins of safety that are built in the design by all three categories.

In that seismic loading is the design determining force for virtually all piping supports, the principal issue here, this affidavit will focus on the safety margin which stems from consideration of seismic design. Loadings from sources other than a seismic event are generally well known, and in many instances the impacts of such loads are tested, e.g., hydrostatic tests, hot functional tests, and operational tests. See e.g., Chapter XIV of the FSAR for a list of tests that have been and will be conducted. While the focus will be on seismic margin, it

is clear that many of the margins discussed apply equally well to static and other dynamic loads, and are discussed in following sections.

While we will quantify the safety factors associated with many of the seismic design considerations, many other considerations can not be quantified, though we know they exist. Accordingly, the final factor of safety presented (on the order of 40, see Table 2) does not represent all margins associated with seismic design. However, it is not necessarily important to know whether all margins are present in a given configuration in the exact amount stated, or that even all are present. What is important is that most (if not all) are there, and that hence the safety margin for seismic loads is large. Obviously, it matters not whether the ultimate factor is 40, 20 or 10. What matters is that the factor is large and would more than compensate for not expressly considering in design each load, no matter how minor.

In addition, it is important to state that structural systems in nuclear power plants are, in general, highly redundant. In effect, this means that there are many alternate ways to carry a given load. If one portion of the structure begins to be overloaded, the excess load is redistributed to other members.

In the particular case of piping systems, this means that if one span becomes overstressed and starts to yield, adjacent spans will immediately begin to pick up the excess load. Similarly, if one pipe support yields, or even fails, the load will be transferred to neighboring supports.

Since the design of piping and supports is based on the localized maximum stress occurring anywhere in the component, there is generally a considerable margin in the rest of the components to readily accommodate the redistribution of loads. In this analysis, no attempt was made to quantify the reserve margin associated with these effects of redundancy and load redistribution, although a margin is clearly there. For instance, one study (Reference 36) has shown that, not only is there a large seismic reserve margin in the average piping system, but that it is virtually impossible to actually fail a pipe through seismic excitation. This study compared the energy that is available in seismic motions with the energy required to form a plastic collapse mechanism in a segment of pipe. It concluded that the most severe ground motion that can be reasonably imagined would not result in failure, or even a significant impairment of nuclear power plant piping.

This is borne out by observations conducted at several plants subjected to severe earthquakes. It is to be noted that these plants were not built with the stringent QA requirements applied to nuclear plants. A summary of these observations is provided in Attachment 1.

II. Seismic Design Margin

The performance of structures and components during past earthquakes indicates that the average facility has a seismic capacity well in excess of its design value. This is true even in those cases, such as petro-chemical plants, where only minimal

attention was originally paid to seismic issues. In the case of nuclear power plants, seismic reserve margins would be even greater than for the average facility. (Reference 1 through 4). In short, current nuclear industry practice leads to both a significant overestimation of seismic forces and an underestimation of seismic capacity. The result is a seismic reserve margin, or added safety factor, that far exceeds original design targets.

It should be noted that the seismic reserve margin specified herein is not the total factor of safety in the design; rather, it is the amount by which the actual safety margins exceed the original design objectives. There is generally an additional margin which is provided by the fact that the designer seldom designs right up to its original design objectives (i.e., design limits as specified by Code). In designing systems to withstand seismic excitations, the design process can be broken down into the following three basic steps (which correspond to the previously stated three categories in which safety margins are built in the design):

1. Seismic Input Definition - specification of the seismic ground motions which could occur at the site as a result of the maximum credible earthquake.

2. Seismic Response Analysis - determination of the maximum stresses, accelerations, displacements, etc., which could be produced in the facility if the specified seismic ground motions actually occurred.

3. Seismic Capacity Evaluation - comparison of the maximum seismic response levels with code-specified minimum capacities.

If the maximum seismic demand is less than the minimum seismic capacity, the design is acceptable. If not, appropriate modifications are made and steps 2 and 3 are repeated until a satisfactory design is achieved.

In theory, any structure could be designed quite rigorously using the above approach. In practice, however, there are many sources of uncertainty in each step of the process and many approximations which must be introduced. The following is a brief discussion of the major areas of uncertainty and the current nuclear industry practice for addressing these in a conservative manner. Also included is an assessment of the conservatism (margin of safety) introduced in the design by the conservative nuclear industry practice. This is included to provide a better understanding of why it is considered appropriate to not expressly factor "secondary" effects (load contributors) into calculations when performing the design.

A. Seismic Input Definition

When the ground moves during an earthquake, it tends to carry the structure along with it. Structural inertia resists the motion, however, producing deformations both within the structure and in the ground; the more severe the ground motions, the more severe the inertial response. The first step in any seismic design, therefore, is to determine how severe future ground motions could be at a particular site.

For a complex facility such as a nuclear power plant, the "severity" of ground motion is dependent upon many different factors. Some of the most important factors are how quickly the ground accelerates, what are the maximum velocities and displacements achieved, and at what frequencies the ground motion changes direction. Each of these and other factors place unique demands on a facility and affects its performance both during and after the earthquake.

The actual value of each ground motion parameter is in turn dependent upon the detailed characteristics of the earthquake, e.g., its magnitude, depth below the surface, and location relative to the site. Different classes of earthquakes will tend to produce characteristically different types of ground motion. In general, therefore, each factor affecting the "severity" of ground motion might be controlled by a different earthquake.

Current nuclear industry practice is to define a hypothetical event which is a combination of all the worst individual earthquakes that credibly could affect the site, and which simultaneously produces the most severe ground motion in all respects. This hypothetical "maximum credible earthquake" then becomes the seismic design basis for all subsequent work on the facility. No single earthquake, no matter what its individual characteristics, produces a ground motion with the all-encompassing severity of the hypothetical composite earthquake. Designing a facility to resist this event, therefore, builds in a reserve margin against any real earthquake which might actually occur.

The three basic conservatisms which are introduced into the design at the stage of seismic input definition are associated with (1) conservatively evaluating the seismic hazard, (2) developing composite ground motions encompassing all possible earthquakes, and (3) the process of developing synthetic time histories.

1. Seismic Hazard Evaluation

Regulatory requirements for nuclear power plants specify that the facility must be designed to resist the ground motions associated with the maximum credible earthquake that could affect the site. While the term "maximum credible" is ill-defined, in practice, it is taken to be the largest earthquake which could occur in roughly a 1,000 to 10,000 year time period (Reference 5).

The peak ground acceleration which might result from the maximum credible earthquake is taken at the 50 percent non-exceedence level. The spectral accelerations associated with the peak ground acceleration are, in turn, taken at the 84 percent non-exceedence level (Reference 6). The actual design objective, therefore, is a facility which can safely resist the maximum ground motions which might occur roughly once every 12,500 to 125,000 years.

Time intervals of such a magnitude are approaching a geologic time scale and are probably physically unrealistic for engineering purposes. Nonetheless, current nuclear industry practice

results in a seismic ground motion that is "at least a factor of 2.4" times the design objective (Reference 4) (underline included in reference).

Overestimating seismic ground motions is the same as over-predicting the seismic forces on every structure and component in the plant. The first quantifiable seismic reserve margin, therefore, is a factor of 2.4 which is carried through to every aspect of piping system design.

2. Composite Ground Motions

The process described above determines the most severe ground motions that could be associated with any one future earthquake. As discussed above however, current nuclear industry practice is actually to define a composite earthquake which is far more severe than any real event. The seismic reserve margins that result from using this hypothetical earthquake can be readily determined from comparative analyses.

In one study (Reference 7) a typical nuclear power plant-type structure was first analyzed using the recorded ground motions obtained from each of 18 different real earthquakes. The same structure was then reanalyzed using a hypothetical motion which represented a composite of the 18 events. The process used to derive the composite ground motion was identical to that used on the Comanche Peak project.

The results show that the composite ground motion over-predicts the peak seismic response that would be expected in a real earthquake by a factor ranging from about 1.5 to 1.7. Simi-

lar results were obtained in the study described in Reference 8, which examined many different types of structures and which used a different set of real earthquakes and composite motions. The general conclusion is that the process of designing to resist a hypothetical, composite earthquake, instead of individual real earthquakes, builds in a seismic reserve margin that is expected to be at least a factor of 1.5. (This margin would apply to all subsystems in the plant including piping systems and components.)

3. Synthetic Time-History

The ground motion characteristics associated with the hypothetical earthquake are actually defined in terms of a seismic response spectrum rather than a specific time-history of motion. (A response spectrum is a plot showing what would be the maximum response of a range of very simple structures to a given motion.) In order to use the response spectrum for design purposes, it is first necessary to generate a synthetic motion that would, in fact, produce the indicated levels of response.

The response spectrum of the synthetic motion is always somewhat more severe than required for design. In the case of Comanche Peak, FSAR Figure 3.7B-2 (Reference 14) shows that the synthetic ground motion leads to seismic response levels that are about 20-30 percent higher than those expected from the hypothetical composite earthquake. The factor of 1.2 is a reserve margin that also applies to all aspects of piping system design.

There are several additional margins associated with seismic input motions that are very difficult to quantify using current technology. For example, a single variable (peak ground acceleration) is used to define the severity of the design response spectrum at all frequencies. It is known that this single variable approach conservatively biases the response spectrum in the frequency range of interest for nuclear power plant design. In addition, there is generally a conservative bias built into the estimate of the peak ground accelerations actually expected in any given earthquake. For the purposes of this study, the combined effect of these and other similar factors will simply be noted, but no attempt will be made to quantify the resulting margin.

B. Seismic Response Analysis

Given the most severe ground motion, as described above, the next step in the seismic design process is a mathematical analysis to determine what impact this motion will have on the facility. This is accomplished by constructing a mathematical model of the facility and then "shaking" it with the prescribed motion. Any response quantity of interest can be determined using various analytical techniques.

The level of analytical sophistication used in the nuclear industry, and the degree of accuracy achieved, is considerably greater than that attempted elsewhere. Even under the best of

circumstances, however, a seismic response analysis involves many simplifying assumptions. For example, a rigorous analysis would require that the mathematical model include a detailed representation of each and every item in the entire facility, as well as a large portion of the surrounding site. Such a model could not be contained in any existing computer, and the time and cost to analyze it would be prohibitive, even if it could be assembled.

Similar constraints apply to all other aspects of the seismic response analysis. In view of these practical limitations, such analyses are simplified in four major respects. The first of these is to reduce the complexity of the models and analyze the facility in increasing levels of detail, one portion at a time. In a typical case, such as Comanche Peak, the first level of modeling would be a simple representation of the main structural elements of the plant: the floors, walls, basement, etc. The second level is an individual substructure: for example, one particular piping system or an electrical cabinet. The third level of modeling is an individual component such as a valve mounted on a pipe or an instrument mounted in the cabinet.

Each model is then analyzed in sequence, with the response of one model becoming the input to the next. When the structural model is analyzed, for example, it yields the motion at each floor level in the plant. If the piping system is floor mounted,

this motion can then be applied to the piping model and used to analyze it. Similarly, the response of the pipe will determine what motion the valve must be designed to resist.

The use of simplified models and a sequential analyses provides a practical solution to the seismic response problem for complicated facilities. In the case of piping systems, however, it tends to introduce a conservative bias into the analysis. This occurs because the process of "uncoupling" the piping model from the structural model will always lead to an overestimation of the severity of motion that is actually transmitted to the pipe. As in the case of the original seismic ground motion, the use of a more severe input motion for the piping analysis builds a seismic reserve margin into the design.

The second major simplification in seismic response evaluations is the assumption that all models behave in a linear, elastic matter. The term "linear" in this context means that the deformation in a structure will increase in exact proportion to the applied load (i.e., if the load is doubled, the deformation will also double). The term "elastic" means that when the load is removed, the structure returns exactly to its original, undeformed state.

The use of linear elastic models, and corresponding analytical techniques, biases the seismic response evaluation in a very pronounced fashion. Real structures dissipate energy with each cycle of motion. This occurs through a variety of different

mechanisms, e.g., internal damping, sliding friction, microcracking, and small permanent deformations. Linear elastic models do not properly account for this dissipation of energy. Instead, the energy supplied by the ground motion accumulates in the model and leads to a considerable overestimation of seismic response.

In addition, real structures do not transfer motion perfectly from one point in the structure to another. There are always small gaps, a certain amount of slipping and sliding, discontinuities, etc., that tend to disrupt motions in the structure and limit any build-up of response.

Linear elastic models cannot reproduce this type of behavior. A seismic ground motion applied to the base of a first-level structural model will be amplified and perfectly transferred to each point in the structure. The motion at the floor level is perfectly transferred to the piping model, further amplified, and then transferred to the valve model. At no point do the models account for the disruptions of motion that occur in real structures.

Since this perfect linear elastic behavior overpredicts the response of structures, using this approach is the same as building a seismic reserve margin into the structure, piping, etc. In direct reference to the concerns raised by CASE, it may be stated that for ductile structures neglecting the effects of gaps is a generally conservative procedure. (We are addressing those

effects under a separate affidavit.) It should be noted that this margin is in addition to the "uncoupling" margin described above. Had the piping system been included in the model of the structure, it would also have disrupted the motion at the floor level through a process termed "dynamic feedback." This is true even in the case of linear elastic models. The nonlinear, inelastic behavior of real structures would result in a further disruption and, hence, reduction in motion (i.e., more margin).

The third major simplification in response evaluations involves the mathematical tools that are used. Given a model and a description of the design ground motion, it is possible to obtain an almost continuous time-history of structural response. A separate response time-history can be obtained for each variable of engineering interest, and for each location in the structure. The time-histories can then be scanned to determine the maximum values for design purposes.

The time-history approach provides a rigorous solution to the seismic analysis problem, but is also quite costly. In a typical application, such as Comanche Peak, it will only be used for the most important first and second level models or where there is some significant benefit to help offset the cost. For the majority of the thousands of second and third level models, however, there are more simplified techniques to choose from. However, the simplified analytical methods lead to a significant overestimation of seismic response.

For the Comanche Peak piping systems, two simplified approaches have been used for almost all of the analyses. The first of these, termed the response spectrum method, has been applied to all large diameter piping (greater than two inches) and roughly 70 percent of the small diameter piping. The second has been used for the remainder of the pipes smaller than two inches in diameter.

The response spectrum method is based upon the observation that, when a pipe is shaken, it responds by vibrating in a number of distinct, repeating patterns of deformation. The more complex the piping system, the more individual patterns that can be detected, each with its own natural frequency of vibration. The total response of the system is the sum of all of the individual vibration patterns, which are called "modes."

Given the design ground motion, it is a relatively easy matter to predict the maximum response of each separate mode; this can be done quite accurately. Each mode reaches its maximum value at a different time, however, which is not given by the response spectrum method. The difficulty, then, is in combining all of the individual modes to determine the maximum response of the complete piping system.

To solve this problem, engineers have devised simple modal combination rules which account for the uncertainty by assuming that all, or at least certain identifiable modes, reach their peak values at the same time. This approach deliberately biases

the results toward upper bound values and builds a reserve margin into the piping design. The exact amount of the margin is never known, but the reserve margin that can be expected for the average piping system has been determined by numerous studies.

The simplified calculations that have been used for the Comanche Peak small diameter piping are also based upon an upper bound approach. In this case, it is noted that any piping system can be analyzed one section at a time using very simple methods that always overstate the true response. Sometimes the reserve margin that is built into small diameter piping is enormous, but the cost penalty for the overdesign is generally less than it would cost for a more rigorous analysis.

The last major simplification that is used in seismic response evaluations deals with the fact that there are a very large number of parameters that influence the detailed nature of the results. It is not possible to consider all of the combinations of parameters that could actually occur in something as complex as a nuclear plant. The only practical solution to this problem is to use the parameter values that lead to the most conservative results. If it is unknown which value will be most conservative, then several different values are used and the design is based on the one that gives the highest response.

Since it is not credible that each parameter will simultaneously assume its worst effect value, this approach builds a considerable reserve margin into the design. In most cases, the extent of the margin cannot be determined in a generic manner. A few areas can be quantified, however, and these are stated below.

1. Site-Structure Interaction Analysis (SSI)

The first stage in a seismic response evaluation is an analysis of the primary structural system, including interaction effects between the foundation materials at the site and the structure itself. If the structure were resting on the surface of a perfectly rigid foundation, the computed response at the base of the structure would be identical to the seismic input motion. If the foundation material is less than perfectly rigid, however, or if the structure is embedded, site-structure interaction effects will tend to reduce seismic response (References 9, 10).

An evaluation of the motions actually recorded during real earthquakes (Reference 11) shows that the combined effect of foundation flexibility and structure embedment resulted in a reduction in seismic response that ranged from 39 to 44 percent. For a depth of embedment similar to that encountered at Comanche Peak, the embedment effect alone reduces seismic response approximately 30 to 40 percent with respect to a surface structure. (References 12, 13). However, both the analytical and observational results cited here apply to structures embedded in foundation materials considerably softer than those found at the Comanche Peak site. For the rock-like foundation at Comanche Peak, foundation flexibility effects would be relatively minor,

and the effects of embedment would be somewhat lessened. Instead of the 30 to 40 percent reduction obtained in References 12 and 13, a 20 to 30 percent reduction would be more reasonable.

The Comanche Peak SSI analysis, however, has been performed using a simplified approach which has resulted not in a decrease in response due to site-structure interaction, but a substantial increase. A comparison of Figures 3.7B-2 and 3.7B-44 in the FSAR, for example, shows an increase due to site-structure interaction that is at least a factor of 2.0 throughout the frequency range of interest. If the average reduction that should have been obtained is roughly 25 percent, this increase represents a seismic margin of 2.67 for all subsystems and components located near the base of the structure.

Higher up in the structure, however, the response is governed more by structural stiffness and less by SSI effects, and hence the reserve margin is considerably less than 2.67. For a base mat spectral acceleration of 0.5g. the corresponding response at the top of the containment building should be in the range of 1.3 to 1.5g. The computed response at Comanche Peak is 1.5g. The seismic reserve margin at the top of the containment is thus somewhere between 1.0 and 1.15. Conservatively taking the value of 1.0 for locations near the top of the structure (where incidentally the only piping is the containment spray piping), and the value of 2.67 at the base mat level, the average reserve margin for the structure as a whole would be about 1.8.

Conservatively we can assume a factor of 1.5. In the preceding, only the horizontal response has been discussed. The reserve margin for vertical response is even higher. Since this margin occurs in the first step of the seismic response analysis, it applies to all subsequent stages of the design, including design of piping systems.

2. Enveloping of SSI Results

One of the reasons that the Comanche Peak SSI results are so conservative is that they are not derived from a single analysis but are, in fact, the envelope response of six separate SSI studies for the containment building and three for the other buildings.

Two of the major sources of uncertainty in any SSI analysis are the engineering properties of the site foundation materials and the extent to which cracking affects the properties of concrete and, hence, the structure. Given these uncertainties, current engineering practice calls for the use of those properties which give the highest response. In neither of these cases, however, is it possible to tell beforehand which properties to select.

At Comanche Peak this problem has been solved for the containment by performing six independent analyses, each using a different consideration of properties for the concrete and the site foundation material. Together, the six evaluations were combined to produce a bounding set of results. As with the hypo-

thetical composite earthquake, however, no one set of real properties could ever produce results as severe as the envelope response of all six separate analyses. An evaluation of FSAR Figures 3.7B-2 and 3.7B-44 (Reference 14) shows that the enveloping procedure has raised the seismic response spectra across the entire range of frequencies. The resulting seismic reserve margin can be expected to be nearly the same as that accruing from using a composite earthquake to define the seismic input motion for the site (see A2 above). Thus it could be about 1.5.

Because of the procedure used to generate the envelope spectra, this margin would not, however, apply to the design of the primary structures, but would apply to piping systems and all other subsystems. Since, however, the same enveloping procedure has not been used for all buildings the margin may vary. Although there is clearly margin present, rather than speculate regarding the margin, no credit will be taken. It is noted that this margin is distinctly different from that discussed in II.B.2 above for the simplified SSI analytical approach. That approach leads to an overestimation of response in particular frequency bands. The use of the enveloping procedure extends the overestimation to a wider range of frequencies.

3. Damping

In a seismic analysis, (or for that matter, other analyses of dynamic events) damping controls the rate of energy dissipation in the mathematical model. It is one of the primary vari-

ables affecting peak response. To be conservative, current engineering practice calls for the use of lower bound values. This leads to an overestimation of the seismic response and the introduction of reserve margins at all levels of design.

The damping values generally used in the nuclear industry are those specified by Regulatory Guide 1.61 (Reference 15), which are reproduced in Table 1. Also shown in Table 1 are comparable values recommended by Newmark (Reference 16). The Newmark values are based upon a review of experimental data for all types of structures at different stress levels. The values quoted represent the average damping that can be expected at the stress levels used for design purposes. Reference 42, p. 129 recommends the use of the upper range of Reference 16 damping values in the design or evaluation for stresses at or near yield. Reference 43, p. 38 recommends that the damping values given in Reference 16 replace the Regulatory Guide 1.61 Table 1 damping values. This recommendation is echoed by a task force commissioned by the NRC (Reference 17) that also recommends that Regulatory Guide 1.61 be revised to incorporate the more realistic damping values suggested by Newmark. It should be stated, in all fairness, that these would be damping values applicable when the designer chooses to make full use of the allowable stress values for design bases seismic events, which would then be near yield. If the designer chooses to design well below yield, then the

lower damping values of Newmark would be appropriate. However, in this instance, the designer would already have provided a safety margin.

TABLE 1
DAMPING VALUES
(Percent of Critical Damping)

Structure or Component	Percent of Critical Damping	
	NRC Guide 1.61 (SSE Values)	Newmark (16) (At or just below yield)
Equipment and large diameter piping (more than 12 in.)	3	3
Small diameter piping (less than 12 in.)	2	3
Welded steel structures	4	7
Bolted steel structures	7	15
Prestressed concrete structures	5	10
Reinforced concrete structures	7	10

The subject of damping values for nuclear power plant piping systems has also been the subject of an extensive study conducted by the Pressure Vessel Research Committee (PVRC). After a detailed review of all available data in this area, the PVRC recommendations (Reference 18) are as follows:

a) For all piping modes in the frequency range of 0-10 Hz, the response analysis should be performed using a damping value of five percent.

b) In the mid-frequency range, damping should be linearly reduced from 5 percent at 10 Hz to two percent at 20 Hz.

c) For frequencies above 20 Hz, the damping should remain consistent at two percent.

The seismic reserve margins that are introduced through the use of lower bound damping values can be determined from a direct evaluation of the design response spectra used for Comanche Peak (Reference 14). For the primary structural model, the use of seven percent damping as opposed to the more realistic value of 10 percent (for instance Reference 43 reports damping values in excess of 10 percent, and Reference 44 reports damping coefficients for the fundamental mode of the Tokai 2 reactor building in Japan, which was only excited to about 0.01g, in excess of 20 percent) would build a reserve margin of about 1.19. Since the response of the structure becomes the input to the piping analysis, this reserve margin also applies to the piping system. Similarly, the piping analysis has been performed using damping values that average roughly 1.5 percent for OBE and 2.5 percent for SSE, considering both large and small bore piping. If the more realistic value is conservatively taken as four percent, the corresponding reserve margin is a factor of 1.16. The combined margin attributable to the use of conservative damping values is thus about 1.38. This value must still be considered a conservative estimate because it is based on the influence that damping has on a broad-banded response spectrum. For the more sharply peaked spectra that are obtained in structural and piping response analyses, the effect is more pronounced.

4. Uncoupled Analysis

As discussed above, seismic response evaluations are carried out through a sequential analysis of separate, "uncoupled" models. This approach, while a practical necessity, cannot account for the seismic interaction that occurs when models are properly interconnected. As a result, a phenomenon termed dynamic feedback is completely neglected.

The importance of dynamic feedback to seismic response predictions has been investigated fairly extensively. Ref. 19 provides an exact set of results in the case of very simple (one degree of freedom) primary and secondary systems. While the results are highly dependent on the relative mass and stiffness of the two separate models, the trend is clear: neglecting dynamic feedback leads to a systematic overestimation of seismic response levels. The overestimation can be as much as 43% for closely tuned systems.

Extrapolating the data (Ref. 19) to the case of real structures and piping systems is a difficult task. Standard design practices would ordinarily ensure that the fundamental piping modes are not tuned to the fundamental mode of the structure. The 43 percent conservative bias that occurs in the case of perfect tuning would thus not apply to the peak value of the typical floor response spectrum. On the other hand, if the seismic input to the piping system is controlled by one of the higher structural modes, dynamic feedback would also reduce the response

of those modes. The amount of the reduction would be the same as shown in Ref. 19. The degree of conservatism resulting from an uncoupled seismic analysis is ultimately a function of how closely tuned the piping system is to the structural mode that dominates the input to the system. For the typical piping system at Comanche Peak, a nominal seismic reserve margin of 10 percent is a reasonable value to use. Several detailed piping analysis studies (Ref. 20) support values at least this great.

5. Envelope Support Excitation Approach

Piping systems contain a large number of supports that are attached to the structure at many different locations. Each structural support point responds somewhat differently during an earthquake. In particular, supports at higher elevations, or closer to the perimeter of the structure will see a stronger motion than supports located at lower elevations, or near the center of the structure.

A time-history approach to piping analysis can account for the different motions that occur at each support location. When using simplified analysis methods, however, this becomes a difficult, if not impossible task. The current industry practice, and the one used at Comanche Peak, is to apply the same seismic input motion at all pipe support locations. In order to ensure that this approach is conservative, the motion that is actually applied is the envelope of the most severe structural responses that occurred at each of the separate attachment points.

Several studies have been undertaken to evaluate the envelope approach in direct comparison with the more rigorous time-history analysis method (Ref. 8,21). The studies cover a wide variety of piping systems, support configurations, input motions, and detailed analytical procedures. In each case, however, the use of the simplified approach has lead to a systematic overestimation of piping system response. The margins of conservatism that were reported in the studies ranged from about 1.4 to 2.0.

Unfortunately, each of the studies also combine in the margin at least one other potential source of conservatism, e.g., the effects of dynamic feedback, modal combination rules, or the prior broadening of the input response spectra. It is thus difficult to isolate the conservatism that can be attributed solely to the use of an envelope excitation approach. For the purposes of this evaluation, it will be assumed that a nominal seismic reserve margin of at least 10 percent can be associated with this simplified analytical technique.

6. Broadened Floor Response Spectra

The response spectra that are used to define the input motions for a piping analysis are usually termed "floor" response spectra. They represent the response for the primary structural model at the major locations where the piping system is attached. In addition to the enveloping procedure described previously, the floor spectra are also broadened prior to use. This broadening is carried out to account for any uncertainties that may be

present in the mode shapes or frequencies of the primary structural model. At Comanche Peak, a ± 10 percent broadening criteria was applied to all floor response spectra. As a result, the envelope spectra used for the piping analysis were also broadened ± 10 percent.

The impact of the broadening is identical to that resulting from the composite earthquake ground motion and the use of envelope results from the site-structure interaction study. The broadened floor spectra represent a more severe response than would be obtained from any model of the primary structure, no matter what its frequency. The use of these spectra lead to a correspondingly more severe piping response.

The use of broadened floor response spectra in general leads to approximately a 20 percent overestimation of piping response. (Ref. 8 & 17). The actual level of conservatism at Comanche Peak may not be quite this large because the extent of broadening was somewhat less than that considered in the referenced studies. For this evaluation, a nominal seismic reserve margin of 1.1 will be attributed to spectral broadening effects.

7. Orthogonal Input Motions

The ground motions associated with earthquakes are multi-directional in nature. In seismic design, this is accounted for by performing the response analysis with three orthogonal input motions: two horizontal and one vertical, all perpendicular to each other. Current practice, as was applied in the case of

Comanche Peak, is for both of the horizontal motions to have the same peak acceleration and response spectrum. The acceleration and spectrum are those associated with the hypothetical composite earthquake.

Measurements of real seismic ground motions show that the orthogonal components are rarely the same, either in terms of peak accelerations or frequency content. Ref. 23 shows a comparison between the peak acceleration value of the larger component and the average acceleration for both components, for all California and Nevada earthquakes recorded between 1954 and 1970. The ratio (R) of largest peak to average peak is shown to increase with distance from the earthquake source. At very short distances, such as 10 km, the ratio is 1.13. This is equivalent to the smaller of the two components being only 77% of the larger.

Given the data from real earthquakes, the current practice of applying three orthogonal input motions all of which are based on the maximum credible ground motion at the site, probably results in a conservative bias of at least 15% to 25%. For this evaluation, the corresponding seismic reserve margin will be taken as a nominal factor of 1.1

8. <u>Modal Combination Rules</u>

When using the response spectrum analysis approach, the primary source of uncertainty lies in the combination of individual modal responses to obtain the total response of the system.

NRC requirements concerning modal combinations (Ref. 24) fall into two categories. If the modal frequencies are well separated, the maximum response of interest can be obtained by taking the square root of the sum of the squares (SRSS) of the individual modes contributing to the response. This is an unbiased approach and yields an accurate prediction of the maximum response of the system.

If the modes are closely spaced, i.e., frequencies differing by 10 percent or less, the NRC requires that possible correlations between the modes, and potential "beating" effects must be accounted for. They allow three alternative approaches that may be used in the case of closely spaced modes. Each, however, shares the requirement that all of the modes that are closely spaced must be combined by absolute summation.

There is no theoretical basis for any of the three alternative methods proposed by the NRC. In fact, theoretical research directly contradicts the absolute summation approach and suggests that it is unnecessarily conservative (Ref. 25, 26). When the NRC rules were originally developed, however, the underlying problems were poorly understood and there was no practical alternative to the use of an upper bound approach.

The degree of conservative bias built into the NRC rules has been evaluated in Reference 27. Based upon a comparison with rigorous time history analysis, it was concluded that the abso-

lute summation of closely spaced modes resulted in an overestimation of seismic response levels that was approximately 35 percent on the average.

There is one area, however, in which the NRC modal combination rules are not conservative. High frequency modes (greater than about 20 Hz) respond in unison and should be combined in an absolute manner. Present NRC criteria allow such modes to be combined using the SRSS approach unless they are also closely spaced. This will lead to an underestimation of the influence of high frequency modes on total response.

While high frequency modes do not often provide a major contribution to the peak stresses in piping systems, the study described in Reference 27 did not adequately address this issue. For the purposes of this evaluation, therefore, it is felt that a value of 1.2 is a more reliable indicator of the seismic reserve margin at Comanche Peak which results from the conservative modal combination approach.

9. Inelastic Deamplification

The term inelastic deamplification will be used herein to refer to the reduction in seismic response that occurs as a result of nonlinear, inelastic behavior in structures. In general, there are two types of nonlinearity that must be addressed. The first is material nonlinearity; that is, the increased energy dissipation that occurs due to material yielding and inelastic stress cycles. The second is geometric nonlinearity, which

occurs due to the presence of small gaps, sliding surfaces, etc. The linear elastic models used for nearly all seismic response evaluations cannot account for either type of nonlinearity.

Of the two types, only material nonlinearity will be addressed here, as margins of conservatism associated with this effect have been quantified by various researchers (References 28 through 32). It will be assumed that conservatism associated with geometric nonlinearity has been at least partially accounted for by considering the more realistic damping values that are expected in real structures (see above).

All seismic input motions which reach the piping systems must first be filtered through the structure. Since designs are roughly balanced, any motion severe enough to challenge the piping design would produce nonlinear behavior in the structure first. Inelastic deamplification would then reduce the motion that is actually transmitted to the piping systems. Even for lower levels of seismic excitation, small, localized nonlinearities in the structure would result in a deamplification of the floor response spectra. Analytical studies reported in Reference 8 quantify an average reduction of 20 percent when the results of an explicit nonlinear analysis are compared to those of a linear analysis of a typical nuclear power plant structure.

The fundamental frequencies of nuclear power plant piping systems are generally in the range of about 8 to 15 Hz. For this frequency band, the reduction in floor response spectra that

accompanies nonlinear behavior of the structure is proportional to the quantity $(2M-1)^{1/2}$ where M is called the ductility ratio and is equal to the maximum nonlinear displacement divided by the displacement occurring at the effective yield point of the structure.

As discussed in Reference 28, a very conservative estimate of the minimum allowable ductility ratio for nuclear power plant structures is 2 to 3. These values have actually been recommended for use in the original design of nuclear facilities. The corresponding reduction in floor response spectra would range from about 40 to 55 percent. The reserve margins due to neglecting nonlinear behavior would be 1.7 to 2.2.

Based upon the above discussion, a value of 1.25 is believed to be a minimum estimate of the seismic reserve margin in piping systems due to neglecting inelastic deamplification effects in the primary structure model. The actual value is probably much greater. Since, however, we have taken credit for an increased primary structure damping to 10%, we have chosen not to include the margin.

C. Seismic Design Capacity Evaluation

Once the seismic response analysis has been completed, the results must be evaluated to determine whether they are within acceptable limits. For nuclear power plant piping systems, the limits for each type of component are defined by the ASME Boiler and Pressure Vessel Code.

The most widely used sections of the Code deal with the minimum yield stresses of the various materials used in piping systems. The yield stress defines the level at which a material begins to accumulate permanent, inelastic deformations. The minimum capacity of a piping component is specified at some percentage of the yield stress. In order for a design to be acceptable, therefore, the computed maximum stresses due to seismic and non-seismic loads must be less than the specified allowable values.

The Code recognizes that piping systems can actually withstand stress levels and deformations well in excess of yield without endangering the pipe or impairing its functionality. One section of the Code (Appendix F) does, in fact, provide a more realistic method of determining the minimum capacity of various components. In general, however, Appendix F is difficult and costly to apply and is rarely used. As a result, a large portion of the real capacity of piping systems is unclaimed and is thus available as a seismic reserve margin.

In addition to the plastic reserve strength, it is recognized that the materials used in nuclear power plants are fabricated and installed to higher standards than is ordinarily the case. Extensive testing documents the higher quality levels achieved. Thus, nuclear grade materials have greater minimum capacities

than those specified in the Code. This high quality also provides a seismic reserve margin which can be quantified, as will be shown below.

1. Material Overstrength

The average strength of a material considerably exceeds the value used for design purposes. This occurs as a result of testing requirements which specify that if any specimen falls below a certain minimum value, the entire lot must be rejected. Manufacturers thus provide an extra margin in their products to avoid severe economic consequences.

Based on the test results for 60,609 specimens (Reference 8), it was found that the actual yield strength of steel materials is 18% higher than the design specified value. The ultimate tensile strength was observed to be 10% higher than that given in the code. For piping and piping support systems, it is generally the yield strength that is most significant for design purposes. The seismic reserve margin associated with the use of code-specified material yield stress is thus 1.18.

It is appropriate here that we address one of the specific concerns of CASE, which deals with the A500 steel reduction in yield strength to that of the basic material A-36 due to annealing welding. (See CASE Findings at I-13). This concern has been separately addressed by Applicants. However, here it is important to note that Applicants' records indicate that the minimum yield strength of their A-500 steel prior to welding is not the

42 ksi listed by the Code, but 56 ksi. Thus, after welding it can be expected that the material properties will be higher than those listed by Code by about 30%.[1]

?. Static Reserve Strength (Code Margin)

Piping and pipe support systems design is based upon allowable stresses which are less than the actual capacity of the various components. There is a reserve margin already built into the ASME Code. For safety-related piping systems, the allowable stresses will range from 0.67 times the material yield stress to 0.7 times the material ultimate stress. A detailed evaluation of the corresponding static reserve margins has been carried out under the direction of the Nuclear Regulatory Commission and is reported in reference 2, which is the same reference quoted by CASE in section I-9, 10 of their Findings. The results range from a minimum value of 1.43 up to a value of 10.41. Thus, there is a minimum margin due to static reserve strength of 1.43 (which is present for all other types of loading besides seismic).

3. Dynamic Reserve Strength

In addition to the static reserve strength, it is also well known that structures and components can withstand much greater loads if the loads are dynamic in nature, i.e., cycled on and off the structures (References 29 to 31). Since the ASME is based entirely upon static strength of components, there is a seismic reserve margin associated with neglecting the dynamic aspects of

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1 Applicants' Response to Partial Initial Decision Regarding A500 Steel (April 11, 1984) at 20.

the loading. Reference 33 describes one major study to quantify the margin in the particular case of nuclear power plant piping systems. The results show that a dynamic load must be, on the average, 1.52 times a static load in order to produce the same stress and strain levels that are currently allowed by the ASME Code for service load D conditions (i.e., loads which include the design basis earthquake). Applicants have considered the SSE as an emergency load (level C), and that this in itself introduces an additional margin which will be discussed in 4 below. Restated, the reserve margin that is currently built into the Code for static loading conditions will still be present for dynamic load 1.52 times as great. The dynamic reserve margin for piping and support systems may be therefore taken as a value of about 1.5.

4. Oversized Members

Seismic design is a trial and error process. First, a preliminary layout is prepared, models generated, and seismic response analysis performed. The preliminary design is then evaluated to ensure that it has adequate capacity. If it does not, the design is strengthened, reanalyzed, and the capacity is reevaluated. This is repeated until the design meets all applicable acceptance criteria. Those portions of the design that are overstrength are not downgraded unless the over-design is clearly

excessive. In order to speed up the process, overstrength components are widely used to ensure that adequate capacity can be demonstrated.

In the resulting design, therefore, every structural member, including piping, will have a strength at least as great as that required to resist all seismic and non-seismic loads. No member will fall below minimum levels and most will be significantly overdesigned. There is thus a built in reserve margin in every piping system, as well as every other aspect of the facility. While this margin could be quantified by reviewing the Comanche Peak qualification reports, this would be an enormous task, moreover it would vary from pipe to pipe and support to support, sometimes being very little and sometimes very large. In addition, Applicants have in some instances set up very conservative design commitments with respect to what they could have allowed. For instance, the SSE is normally considered an emergency load rather than a faulted load. Further, in most instances piping supports are designed to normal and upset allowables for emergency loads. Thus, the allowable stresses are correspondingly lower than one would have allowed.

While, it is recognized that "oversized" members do create a substantial margin, no credit is taken here for it.

III. Static and Dynamic Loads

It should be noted that some of the safety factors described herein are also applicable to loads other than those caused by seismic events, i.e., other dynamic loads and static loads. Specifically, in addition to margins inherent in computation of dynamic loads, the following safety factors set forth in Table 2 also apply to dynamic load design: B.3 (damping), B.4 (uncoupled analysis), B.7 (orthogonal input motions), B.8 (modal combination rules), C.1 (material overstrength), C.2 (static reserve load) and C.3 (dynamic reserve load). The conservatively calculated factor of safety regarding dynamic loads would be the product of these margins, which is 5.0 (See Table 2). Similarly, reserve margin exists for static loads. These minimum margins from Table 2 are C.1 (material overstrength) and C.2 (static reserve strength). The factor of safety regarding static loads is at least the product of these margins which is 1.68.

IV. Conclusion

This completes the quantitative description of margins which exist in the design of Comanche Peak. It is not necessarily important to dwell on whether all are present in that exact amount, or that even all are present. What is important is that most (if not all) are there, and that hence the safety margin for seismic loads is large and would more than compensate for not expressly factoring some small, potential loads into the design.

Accordingly, there is reasonable assurance that the industry practice of not considering certain small, potential contributing loads (i.e., contribution of support masses) or considering generic support stiffness rather than actual, doesn't affect plant safety. While it is recognized that such practices can result in estimated loads which may not be as conservative as those which would be computed by the most conservative practice, nevertheless they are conservative with significant built-in margins of safety which are inherent in the approach taken to define the input loads and then to analyze their effects.

TABLE 2

CUMULATIVE SEISMIC RESERVE MARGINS

The following is an evaluation of the cumulative effect of the individual seismic reserve margins which have been identified. Since each of the margins occurs "in series", the total or cumulative margin is obtained from the product rather than the sum of the individual margins.

Item	Reserve Margin	
A. Seismic Input Definition		
1. Seismic Hazard Evaluation	2.4	
2. Composite Ground Motions	1.5	
3. Synthetic Time-History	1.2	
Cumulative Margin in Seismic Input Definition		5.1
B. Seismic Response Analysis		
1. Site-Structure Interaction Analysis	1.5	
2. Enveloping of SSI Results	None assumed	
3. Damping	1.38	
4. Uncoupled Analysis	1.1	
5. Envelope Support Excitation Approach	1.1	
6. Broadened Floor Response Spectra	1.1	
7. Orthogonal Input Motions	1.1	
8. Modal Combination Rules	1.2	
9. Inelastic Deamplification	None assumed	
Cumulative Margin in Seismic Response Analysis		3.63
C. Seismic Capacity Evaluation		
1. Material Overstrength	1.18	
2. Static Reserve Strength (Code Margin)	1.43 - 10.41	
3. Dynamic Reserve Strength	1.5	
4. Oversized Members	None assumed	
5. Redundancy	None assumed	
Cumulative Margin in Seismic Capacity Evaluation		2.53
D. Total		
Cumulative Reserve Margin in Nuclear Power Plant Piping and Support Systems for Seismic		46-47

Robert C. Iotti

Sworn to before me this 20th day of May, 1984.

M Pamela Blake

Notary Public

John C. Finneran Jr.

John C. Finneran

Sworn to before me this 20th day of May, 1984.

M Pamela Blake

Notary Public

My Commission Expires January 31, 1985

_______ ____________________
R. D. Wheaton *

Sworn to before me this _____ day of May, 1984.

____
Notary Public

* The original of Mr. Wheaton's notarized signature will be furnished under separate cover.

RANDALL D. WHEATON

EDUCATION	B.S., Civil Engineering, University of California, Berkeley, 1968
	M.S., Structural Engineering and Structural Mechanics, University of California, Berkeley, 1969
	M.E., Structural Engineering and Structural Mechanics, University of California, Berkeley, 1972

PROFESSIONAL EXPERIENCE

Mr. Wheaton has over 15 years experience in the design and analysis of civil engineering projects. For the past 10 years he has specialized in the field of earthquake engineering. Mr. Wheaton is presently the Staff Consultant for Impell Corporation's Advanced Engineering Division. In this role he provides technical direction for virtually all of the company's structural analysis and seismic-related work. Mr. Wheaton also directs the company's research and development activities in the areas of earthquake engineering.

Mr. Wheaton's professional experience encompasses all aspects of seismic design and analysis; seismic hazard evaluations, site ground motion and site stability studies, site-structure interaction and structural response analysis, subsystem response and component qualification, in-situ and shake table testing, and probabilistic seismic safety evaluations. Representative projects include: nuclear, fossil and geothermal power plants, dams, pipelines, floating and buried structures, and various types of industrial facilities.

Mr. Wheaton has also helped to develop standardized design criteria, analytical methods and procedures for evaluating and combining extreme loadings on structures (ASCTZ, ASTM, ANSI standards). He has also provided testimony on seismic issues in several public forums, including hearing by the Nuclear Regulatory Commission, Advisory Commission on Reactor Safeguards, and the California Energy Commission.

Some of the recent projects which Mr. Wheaton has directed include the following:

- o Generic inelastic analysis for the quantification of seismic margins for the V.C. Summer Nuclear Station
  Client: South Carolina Electric Gas Company

- o Development of seismic design criteria for non-nuclear power generation and transmission facilities
  Client: Pacific Gas and Electric Company

- o Probabilistic evaluation of seismic safety for selected dams in Southern California
  Client: Los Angeles County Flood Control District

- o Evaluation of liquefaction potential at the Thorpe's Head Chemical Separation Facility.
  Client: British National Fuels Limited

- o Development of more reliable methods for the seismic analysis of Alaska Natural Gas Transmission System
  Client: Fluor Engineers and Constructors

- o Confirmatory site-structure interaction analysis for the GESSAR II Standardized Nuclear Island
  Client: General Electric Company

- o Seismic Reevaluation of the Millstone power project, including criteria development
  Client: Northeast Utilities Service Company

- o Generic site-structure interaction analysis for the Advanced Boiling Water Reactor (ABWR) project
  Client: Toshiba Corporation

- o Complete seismic hazard evaluation and generic seismic analysis for the San Joaquin Nuclear Project
  Client: Los Angeles Department of Water and Power

PROFESSIONAL AFFILIATIONS

Seismological Society of America

Earthquake Engineering Research Institute

American Society of Civil Engineers

- Seismic Analysis Committee
- Committee on Uncertainties and Conservatism in Seismic Analysis
- Committee on Seismic Input to Subsystems
- Committee on the Seismic Analysis of Buried Pipelines
- Committee on Probabilistic Risk Assessment

SELECTED PUBLICATIONS

"Probabilistic Evaluation of the SSE Design Spectrum for a Nuclear Power Plant Site: A Case Study", 5th International Conference on Structural Mechanics in Reactor Technology, Berlin, August 1979

"Uniform Probability Response Spectra for a Site Near the San Andreas Fault", 7th World Conference on Earthquake Engineering, Istanbul, September 1980

"Criteria for Protection Against the Consequence of Pipe Rupture", 6th International Conference on Structural Mechanics in Reactor Technology, Paris, August 1981

"Evolving Technology in Earthquake Engineering", seminars presented in Japan, Korea, and Taiwan, 1982

"Earthquake Engineering: Theory and Application", seminar presented to the United Kingdom Atomic Energy Authority, January 1983

"Damage Potential of Small Earthquakes: A Generic Case Study", ASCE Speciality Conference, Raleigh, North Carolina, June 1984

"Integrated Dynamic Testing and Analysis", ASCE Speciality Conference, Raleigh, North Carolina, June 1984

"A beam Analogy for the Seismic Analysis of Buried Lifelines", submitted to 8th International Conference on Structural Mechanics in Reactor Technology, Brussels, 1985

ATTACHMENT 2

EARTHQUAKE EXPERIENCE ON OTHER STRUCTURES

Non-nuclear piping systems in steam plants and oil refinery plants have been subjected to earthquake events which have high seismic acceleration levels. The damage to the piping systems has been small. Several examples of these events are discussed in References 37 through 41. Some of these cases are discussed below.

El Centro Steam Plant

The El Centro Steam Plant experienced a significant seismic disturbance during the October 15, 1979, Imperial Valley earthquake. Since the facility is considered similar by the NRC, in both design and types of equipment, to older operating nuclear power plants, an NRC team inspected the plant. Only minor damage to the plant's structural and mechanical systems was found by the NRC team, even though the plant was subjected to an estimated 0.5 g peak horizontal ground acceleration (0.66 g vertical). The plant was designed for a static lateral load equivalent to 20 percent of the combined dead and live load. The NRC contracted Lawrence Livermore Laboratory to analyze Unit 4 of the steam plant and evaluate the equipment response. The frequency response characteristics, acceleration levels, damping, and stress levels were studied. The results of the study are reported in Reference 38. They found that the equipment acceler-

ation responses varied from 0.5 to 1.8 g in the N-S direction, 0.4 to 1.2 g in the E-W direction, and 0.4 to 1.55 g in the vertical direction. The following statement is made in Reference 38, page xvi, concerning the plant equipment reserve strength:

> The forces experienced by the plant equipment were on the order of 2 to 9 times greater than the 0.2 g specified design load. This would seem to imply a reserve seismic equipment capacity of about 200 percent. However, it is difficult to verify this value without knowing the actual lateral load the equipment could withstand. One can only conclude that nuclear power plant equipment similar to that in Unit 4 and anchored as well, should perform equally well during a similar earthquake.

The seismic response characteristics of a selected piping system at this plant were studied by Westinghouse. The results of the seismic analysis performed are reported in Reference 39. In the site review of the piping systems, it was apparent that pipe motion in excess of two inches occurred. An eight inch nominal diameter turbine gland steam line was chose for analysis. This piping system was verified to move 2 3/4 inches south, 5/8 inches east and 1/4 inch vertically upward during the earthquake. The piping line was evaluated using ± 15 percent broader spectra generated from a time-history analysis of the building. The time-history was consistent to the seismic site input seen during the Imperial Valley earthquake. It was determined from the seismic analysis, using the ANSI B31.1 Code, that the piping line would be severely overstressed. Even using the ground input without amplification in the building, stresses are almost twice the allowable stress (2.4 S_h) are predicted. Even though the analysis predicted significant overstress, the pipe showed no

indication of failure. Also, a typical support capacity for this line was calculated, and the predicted seismic loads associated with five of the ten rigid supports were over allowable, and none of the supports showed signs of failure

Huachipato Steel Plant, Chile (1960)

The plant seismic behavior during an earthquake of magnitude 7.5 is discussed in Reference 40. It is concluded from a review of the seismic behavior and plant condition after the earthquake that:

1. The maximum magnitude of the earthquake that damaged the Huachipato Steel Plant on May 21, 1960, was approximately 7.5.
2. The moving fault may or may not have been opposite the Huachipato plant but indications are it was, or was very close.
3. The Huachipato design was a static type. However, it did not adequately compensate for buckling possibilities (which occurred) and, of course, it did not consider dynamic response phenomena.
4. Huachipato did not have its piping and equipment properly anchored for seismic effects.
5. In spite of items (1) through (4) above, the Huachipato plant was damaged only 0.4 percent of its cost and it was operating normally in 6 days, during which period strong aftershocks occurred.

ATTACHMENT 3

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