SEISMIC REEVALUATION PROGRAM

RETURN TO SERVICE PLAN

SAN ONOFRE UNIT 1

December 23, 1983

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1.0 INTRODUCTION

In June of 1982, SCE committed to structurally upgrade San Onofre Unit 1 to withstand a 0.67g ground motion. Since that time significant backfit work has been completed at the plant. However, in view of SCE's belief that the seismic hazard is not a significant concern for the site and in view of the time and resources required to complete the total scope of the seismic reevaluation program, SCE has developed a detailed plan which would permit return to service following completion of a limited scope of modifications. The basic premise of this plan is that all structures, systems and components whose failure could cause an accident and/or whose function is required to obtain and maintain a hot standby condition will be available following a 0.67g earthquake. As such, the plant can return to power without undue risk to the health and safety of the public even considering the possibility of a major earthquake at the plant site.

This report is divided into six sections including this introduction. Section 2.0 provides a summary of the current plant status with respect to implementation of the seismic reevaluation program. Section 3.0 provides a general overview of the return to service plan. Sections 4.0 and 5.0 provide detailed discussions of the two main aspects of the plan. Finally, Section 6.0 provides an overall conclusion. As outlined in Section 3.0, the two main aspects of the return to service plan are: first, a demonstration that based on the current status of the plant and current knowledge, the seismic hazard should no longer be a significant concern for the plant as a whole; and second, an evaluation prior to return to service of all structures, systems and components required to obtain and maintain a hot standby condition to ensure their availability following a 0.67g earthquake.

2.0 CURRENT PLANT STATUS

Since the plant was designed and constructed in the early 1960's, a significant effort to analyze and upgrade SONGS 1 to a 0.67g design basis earthquake level has been undertaken. This effort is completed to the point that SONGS 1 is no longer the same plant that it was in the early 1960's or even that it was in early 1982.

Table 2-1 lists the analyses and modifications completed prior to 1982. Of note on this list are that the reactor coolant loop and containment are capable of withstanding a 0.67g earthquake and that a new sphere enclosure building, a new diesel generator building and two new diesel generators with all of the necessary auxiliary systems were designed and installed to 0.67g.

Table 2-2 lists the analyses and modifications which have been initiated since 1982. Of note on this list are that all structures required to get to a safe shutdown have been modified to 0.67g, the masonry walls have been tested and shown capable of withstanding 0.67g, a new auxiliary feedwater tank has been constructed, approximately 80% of the electrical raceway supports identified in the seismic reevaluation program have been installed and approximately 50% of the pipe supports identified in the program have been installed.

All of the analyses and modifications identified on Tables 2-1 and 2-2 have been in accordance with criteria previously discussed with and reviewed by the NRC. These include the Balance of Plant Structures criteria and the Balance of Plant Mechanical Equipment and Piping criteria forwarded by letters dated February 23, 1981 and May 23, 1983. Based on review of these tables, SONGS 1 has substantial capability to withstand a large earthquake such as 0.67g.

TABLE 2-1 MODIFICATIONS PRIOR TO 1982

- Reactor Coolant Loop Equipment Supports Installed
- Containment Okay As Is
- New Sphere Enclosure Building
- New Diesel Generator Building
- New Diesel Generators and Auxiliary Systems
- New Auxiliary Feedwater Discharge Piping
- Service Water Reservoir Okay As Is
- Electrical Equipment Anchorages Modified
- Control Building and Seawall Okay As Is

TABLE 2-2 MODIFICATIONS SINCE 1982

- Turbine Building Structural Modifications Installed (South Turbine Extension Not Completed)
- Modifications To Masonry Wall Connections in the Ventilation Building, Reactor Auxiliary Building and Fuel Building Installed. Additional Modifications to Masonry Walls Installed in the Turbine Building
- Masonry Wall Test Program Successfully Completed
- Strengthening Brace Added to the Fuel Building at the East Wall of the New Fuel Room
- Strengthing Beam Added to the Intake Structure Pump Well Walls
- In-Situ Soil Conditions Mapped and Defined
- Approximately 1,800 of 4700 Piping Supports Modified or Installed
- Approximately 400 of 600 Cable Tray Modifications Installed or Modified
- Approximately 1300 of 1500 Modifications to Conduit Supports Installed or Modified
- Approximately 300 of 600 Cable Tray Tiedown Locations Modified
- A New Seismically Qualified Control Room Ceiling Installed
- A New Auxiliary Feedwater Tank Constructed
- Containment Spray Rings Modified

3.0 RETURN TO SERVICE PLAN

Although the analyses and modifications completed to date, as described in Section 2.0, provide substantial confidence in the capability of SONGS 1 to withstand a 0.67g earthquake, the implementation of this return to service plan provides further confidence that SONGS 1 will be capable of obtaining and maintaining a hot standby condition following a 0.67g earthquake. This plan consists of two aspects: first, a demonstration that based on the current status of the plant and current knowledge, the seismic hazard should no longer be a significant concern for the plant as a whole; and second, an evaluation prior to return to service of all structures, systems and components, required to obtain and maintain a hot standby condition to ensure their availability following a 0.67g earthquake.

The first part of the plan includes an examination of the conservatism of the ground motion used in the seismic reevaluation program, a review of the performance of industrial facilities in past earthquakes, and an examination of the seismic risk at SONGS 1 based on a probabilistic risk assessment review. This information is provided in Section 4.0 of this report.

The second part of the program includes an identification of those systems required to obtain and maintain a hot standby condition, a definition of the acceptability criteria to be applied to demonstrate the availability of these safe shutdown systems following an earthquake, and the evaluation of the safe shutdown systems. This information is provided in Section 5.0 of this report.

4.0 SEISMIC HAZARD

The discussion of the seismic hazard at SONGS 1 is separated into three areas: (1) ground motion, (2) earthquake experience, and (3) seismic risk. These discussions are provided in the following subsections.

4.1 Ground Motion

4.1.1 Instrumental PGA's for the SONGS Site

The most comprehensive study of ground motion for the SONGS site was done by the TERA Corporation, and the report is presented formally in the written testimony of L. H. Wight (Ref. 1) for the ASLB Hearings for SONGS Units 2 and 3. That work was reviewed by other consultants to SCE, and was thoroughly litigated at the ASLB Hearings. The results have been published by the Seismological Society of America (Ref. 2).

Figure 4-1 is Figure 1-1 of the TERA report, and it gives the essential results as they apply to the SONGS site. The maximum local magnitude was rounded up to M7, and the offshore zone of deformation (OZD) was characterized as being 8 km from the site at its point of closest approach. For those conditions, reference to Figure 4-1 shows that the best-estimate value for the Instrumental Peak Ground Acceleration (IPGA) for the SONGS site is about one-third g.

In normal design practice it is customary to choose the best-estimate, or median, value of IPGA, and to rely upon the safety margins to care for the lower-probability events. In the design of critical facilities, however, it has become the practice in the United States to choose the design IPGA as one standard deviation (84th percentile, 1-sigma) above the median. That value is tabulated in Table 4-1, based upon the TERA report, as about one-half g. By contrast, the reanalysis IPGA of two-thirds g is shown in Table 4-1 to be about two standard deviations above the median.

The return periods for these two IPGA's are shown in Figure 4-2, which is taken from a SONGS site seismic hazard study which is included as Appendix A to this report. In that figure, the results are shown for the cases where the data have been truncated at $\pm$ one, two, and three standard deviations for the case of M7 and a closest faulting distance of 8 km. Those results show that the return period for the one-sigma value, one-half g, is about 10^5 to 10^6 years; while that for the reanalysis value, two-thirds g, is about 10^6 to 10^7 years.

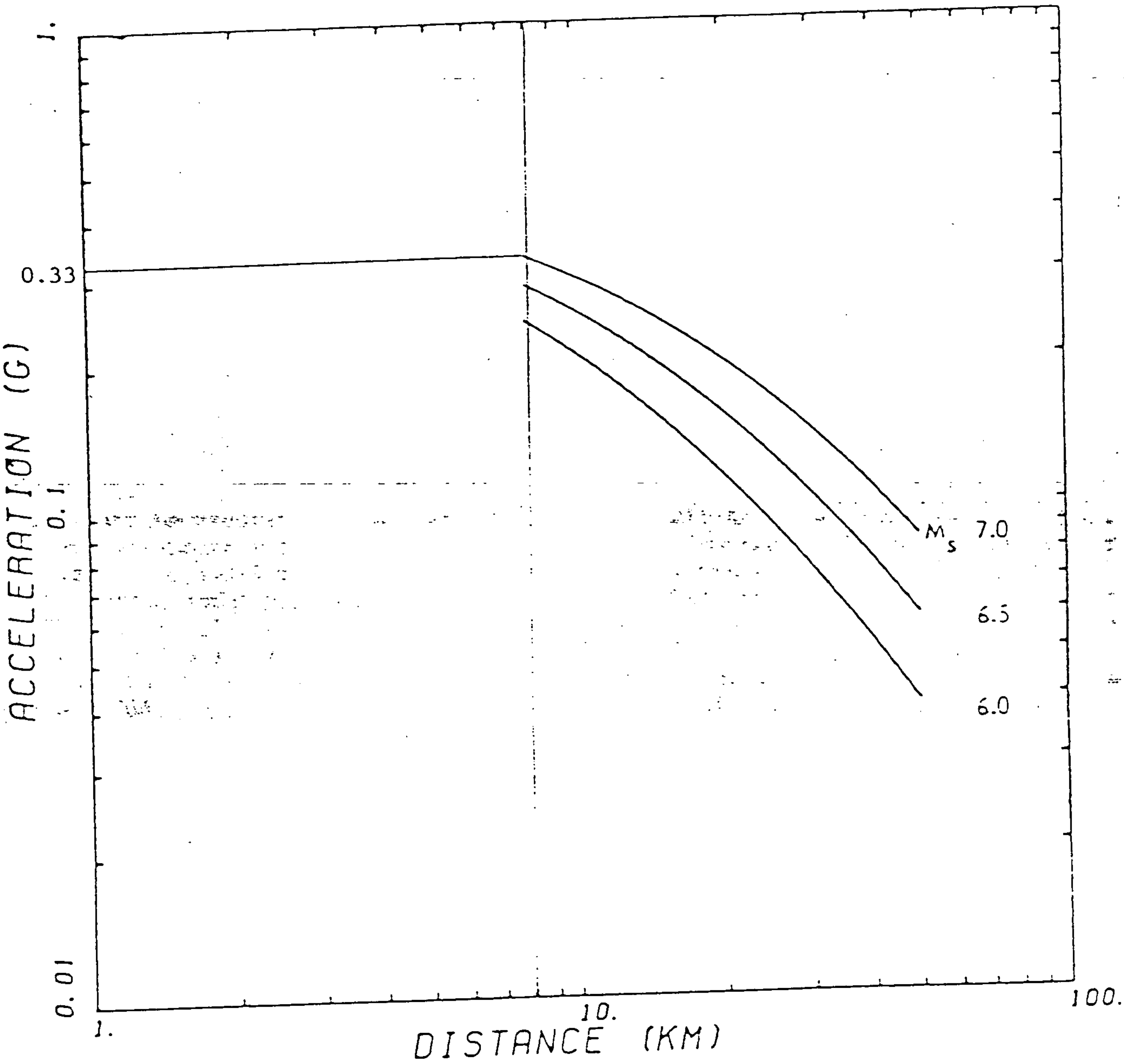


FIGURE 4-1

PREDICTED MEDIAN PEAK ACCELERATIONS
FOR MAGNITUDES 6.0, 6.5 and 7.0

TABLE 4-1

MAGNITUDE, M_S	HORIZONTAL PEAK GROUND ACCELERATION AT 8 KM (G)		
	MEDIAN	MEDIAN + 1σ	MEDIAN + 2σ
6.0	0.24	0.38	0.52
6.5	0.29	0.46	0.63
7.0	0.33	0.52	0.71

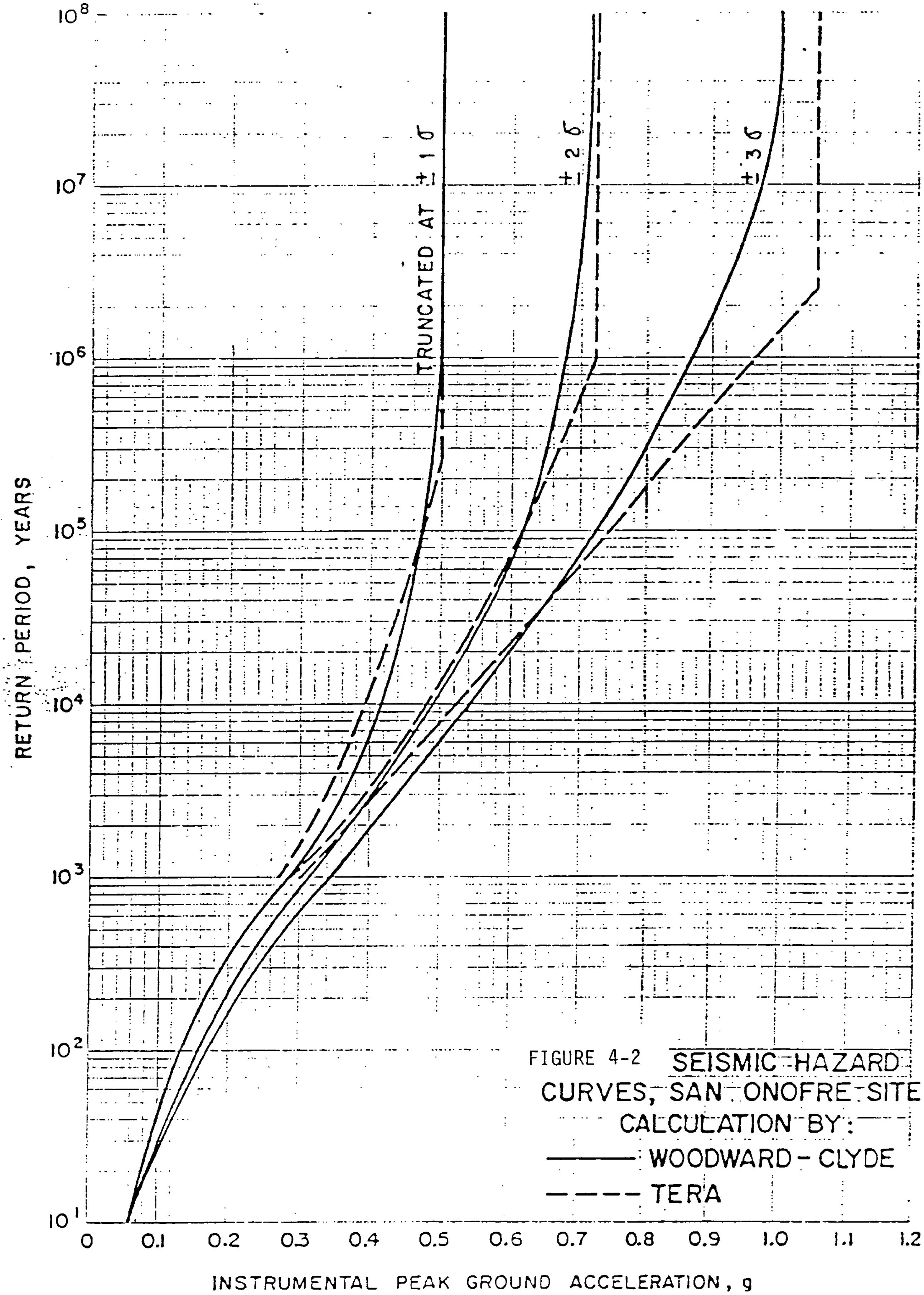


FIGURE 4-2 SEISMIC HAZARD CURVES, SAN ONOFRE SITE

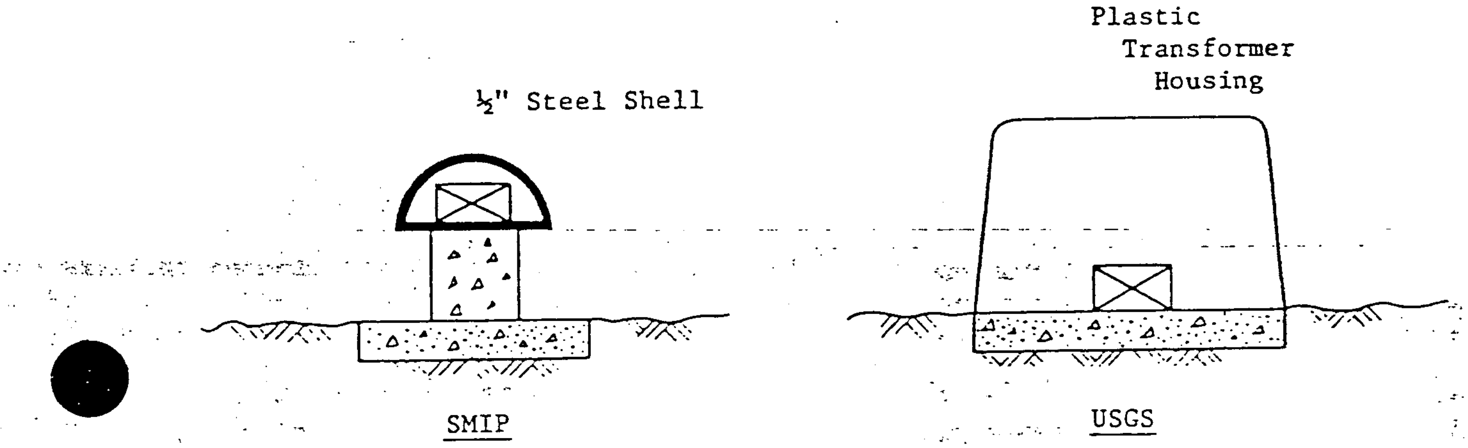
Thus if the SONGS site were to be evaluated today, the design IPGA would be 0.5g, not the present reanalysis value of 0.67g. This higher value was prescribed in the early 1970's, when the implications of the Pacoima Dam record from the 1971 San Fernando event were not yet clearly understood. At that time a spirit of conservatism prevailed, and that led to the very conservative choice of 0.67g, which turns out to be about a two-sigma value. The Pacoima record has since been studied by Boore (Ref. 3), who demonstrated that the ridge upon which the instrument was founded had amplified the ground motions. Subsequent data from large earthquakes, which were well instrumented close in, have supported that point of view, and have supplied the data for the TERA ground motion studies.

4.1.2 Validity of Instrumental PGA's

Recent work (Ref. 4) has shown that small slabs, such as those used to found many seismometers, appreciably distort the free-field motions which are exciting them. That work is based upon earlier theoretical (Ref. 5, 6, 7, 8, and 9) and experimental (Ref. 10, 11, and 12) efforts, and suggests methods for correcting existing records based upon a simple theoretical approach (Ref. 13). That work and some of the evidence it compiled will now be reviewed.

Graphic evidence of the distortion phenomenon has been provided by McJunkin (Ref. 14). In 1972 California initiated a Strong-Motion Instrumentation Program (SMIP). The standard SMIP installation was a 3-ft. square concrete pad, about 5 in. thick, supporting a 16-in. cube of concrete upon which the seismometer was placed. To protect the seismometer, an 0.4-in. thick iron dome was placed over the concrete cube. The dome was larger than the cube, so that the installation resembled a stubby inverted pendulum. In 1978, a seismometer installation similar to many USGS installations (4 to 6 ft. in plan, several inches thick; Ref. 7) was placed about 20 ft. away from a SMIP installation. Both installations had the same model of seismometer by the same manufacturer. The two are sketched in Figure 4-3. Subsequently, an earthquake of M4.9 occurred nearby. Both instruments wrote unambiguous high-quality records, for which the IPGA's are given on Figure 4-3. Compared to the IPGA's of the USGS installation, the IPGA's of the SMIP installation are 1.6 to 2.0 times the horizontal component, and are 1.25 times the vertical component.

The Imperial Valley event of October 15, 1979 furnished much valuable data, some of which apparently supports the position that the method of founding a seismograph influences the resulting records. Figure 4-4 shows the locations of the instruments, and several other features which will now be discussed.



Magn = 4.9
R = 8 mi.

Component	Peak Accel. g SMIP	Peak Accel. g USGS	Ratio SMIP/USGS
H - T	0.22	0.11	2.00
H - L	0.16	0.10	1.60
V	0.05	0.04	1.25

FIGURE 4-3

Instrumental PGAs From Two Adjacent Installations From Same Earthquake

The Instruments Were Both SMA-1s.

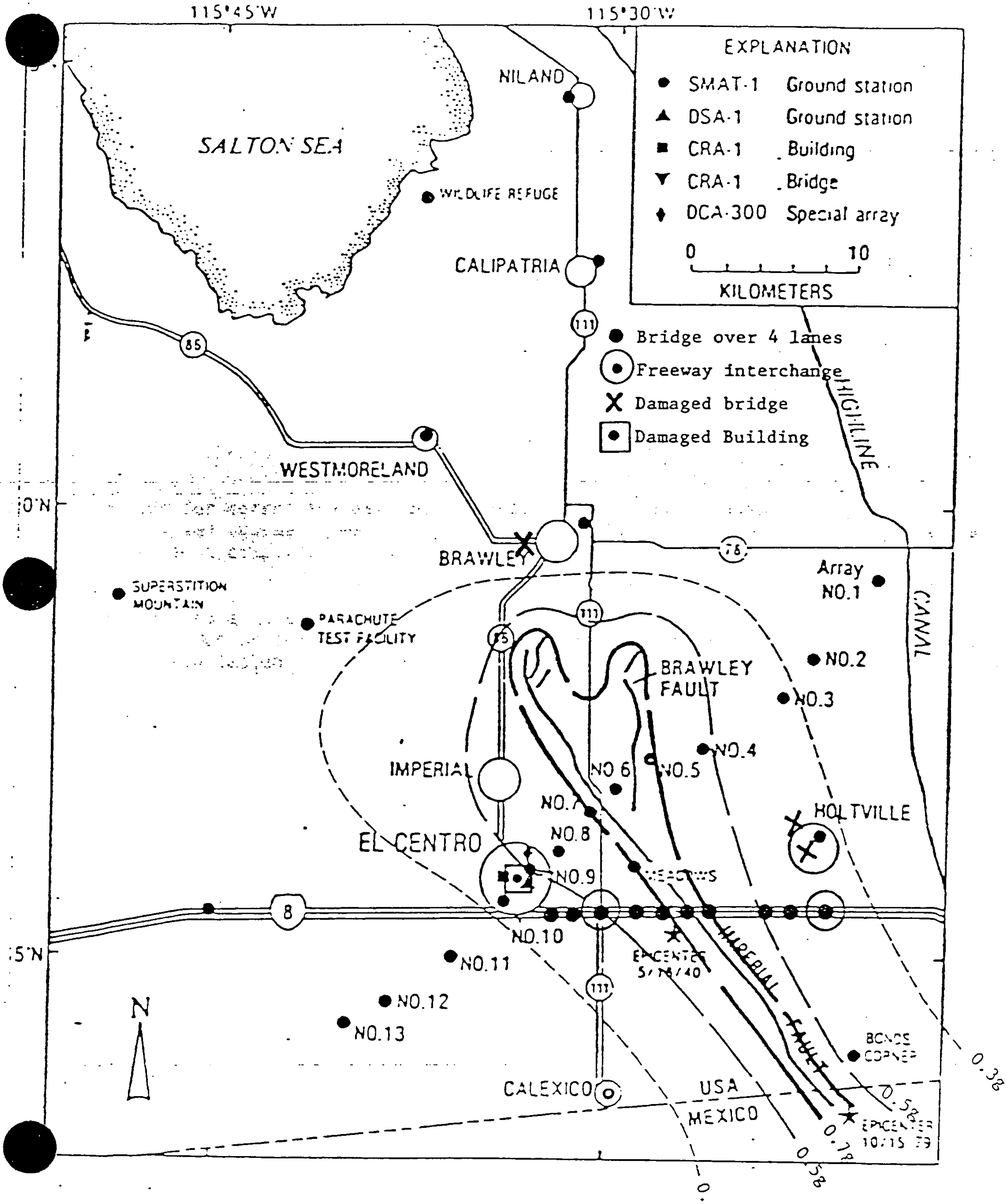


FIGURE 4-4

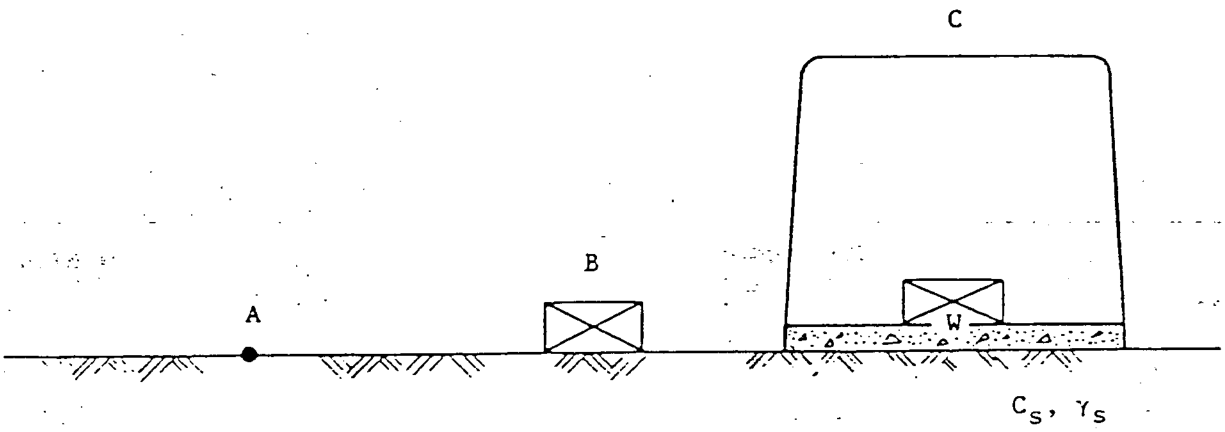
Strong-Motion Stations in the Imperial Valley, California, October 1979

Array No. 6 (lower middle of the figure) wrote 1.75g vertically during the main shock. A short distance away, and nearer to the fault scarp, was a house which was not damaged (Ref. 15), but which should have been if it truly had been hit with almost two g's. A few kilometers to the south lies Interstate 8. One of its overpasses, Meloland (just to the upper right of the star which shows the 1940 epicenter), is as close or closer to the fault than is 6; but its instrument only wrote about one-third g. Array No. 6 also fairly consistently wrote records greater than its neighbors, 5 and 7, even though some of the aftershocks were physically nearer to 7.

Taken as a whole, the data set from the main shock are adequately internally consistent to allow contouring the area for IPGA for the main shock. This has been done, and the results are shown on Figure 4-4. An IPGA or two do not agree with the contouring shown: the Meloland record is a noteworthy case. There were and are a number of overcrossings and two interchanges on I-8, as highlighted on Figure 4-4. According to the contouring, several of these were in areas of very strong IPGA's, probably quite a bit higher than their designs. None suffered distress or loss of function.

Observations and data of this sort force the asking and answering of the question illustrated in Figure 4-5: "When the ground shakes, does point A in the free field move the same as instrument B at the gound surface as instrument W inside installation C?" It seems clear at this point that the answer to the question is "no," and that has been recognized at least since the late 1930's (Ref. 10) and later (Ref. 5, 6, 7, 8, 9, 11, and 12); and study of the situation continues to the present day (Ref. 16).

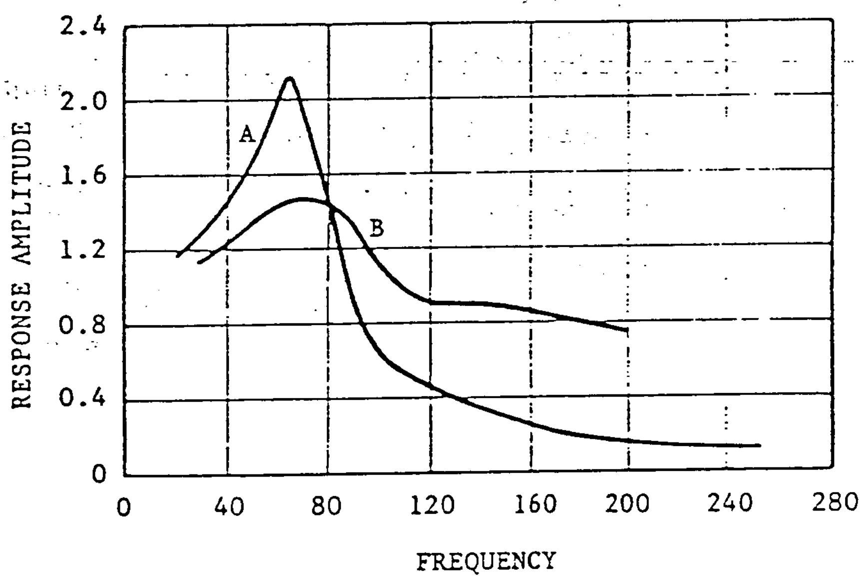
Figures 4-6, 4-7, and 4-8 show the results of experiments in which two vertical seismographs were rigidly connected axially, and one was wired as a driver while the other was left in its normal receiver mode (Ref. 10). They were first suspended in air on a rope and calibrated by scanning through a range of frequencies. They were then placed on the ground and scanned through the same range of frequencies. In both cases the amplitude was measured, probably in the form of velocity. For each frequency, the ratio of on-ground to in-air amplitude was computed, and is plotted in Figures 4-6 through 4-8. When that ratio is unity, the on-ground motion is not distorted. When the ratio is other than unity, the founding condition is distorting the input motion. As the data show, the distortions are appreciable, identifying strong dependencies upon founding area, installation weight, and soil conditions. Data from similar and different experiments by other investigators using equipment and techniques from several countries lead to the same conclusions.



When Ground shakes, does

A move same as B move same as C?

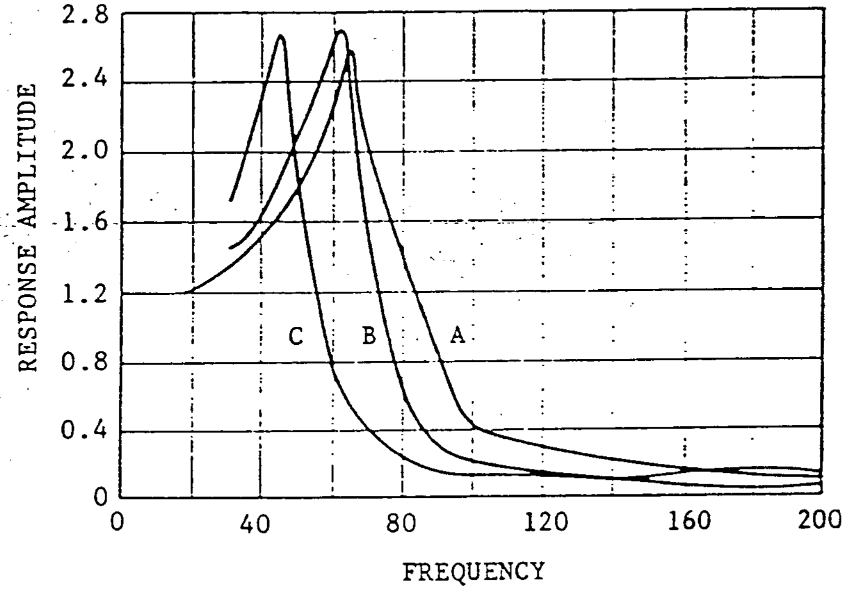
FIGURE 4-5 Statement of the Problem



Seismometer
Base Area

A - 14 sq. in.
B - 80 sq. in.

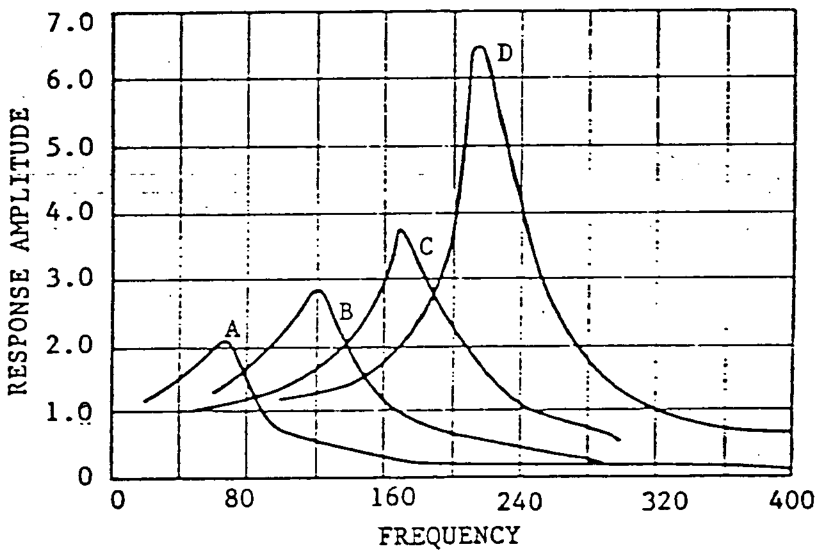
FIGURE 4-6 Seismometer Response For Two Base Areas



Seismometer Weight

A - 11 lbs.
B - 16 lbs.
C - 27 lbs.

FIGURE 4-7 Response For Three Seismometer Weights



A - Wet spongy mud.
B - Dry sandy clay loam (plowed).
C - Moist clay and gravel.
D - Very dry clay and gravel.

FIGURE 4-8 Seismometer Response For Several Types of Ground and Surface Conditions.

The data just discussed were obtained from small, light seismometers, and therefore yielded responses at frequencies higher than the range of interest in earthquake engineering. The same results can easily be calculated for other installations by utilizing the theory of the elastic half-space in its simplified form (Ref. 4 and 13). That calculation requires knowledge of the geometry of the installation and the nature of its contact with the ground; and also certain properties of the supporting ground, probably only to a depth of one foundation radius or less. Three properties are needed, but only one really controls the result: the soil stiffness, usually expressed in the form of shear-wave velocity. It would be a very simple thing to correct the existing significant strong-motion, close-in records by letting the installation measure its own shear-wave velocity by plucking the slab, then back-calculating the shear-wave velocity, and then correcting the measured earthquake accelerograms for that installation. But that is not done at the present time.

A range of shear-wave velocities has been assumed to calculate the steady-state transmissibility curves of Figure 4-9. Those curves assume a small slab, 3 ft. in diameter and 6 in. thick, to lie at the ground surface. The vibrations are coming up from the ground below. The ordinate is the ratio between the response of the slab and the free-field motion which is exciting the slab. This steady-state calculation is probably not a bad approximation of a transient, such as an earthquake, because the spatial damping of a little slab of this sort is so high that it takes only a cycle or so of a given frequency to achieve steady-state amplitude for that frequency. The results show that a small slab, of the type used by USGS in many of their installations, does indeed distort the free-field motions to a degree which should not be ignored, although it presently is. The importance of these results to IPGA is at the top end: if any pulse of the record suffers from any amplification so as to make it the highest-amplitude pulse on the record, then that amplified pulse will be the IPGA of a response spectrum. Thus the only effect this phenomenon can have on a response spectrum is to raise it, certainly in the PGA range and perhaps at other frequencies as well.

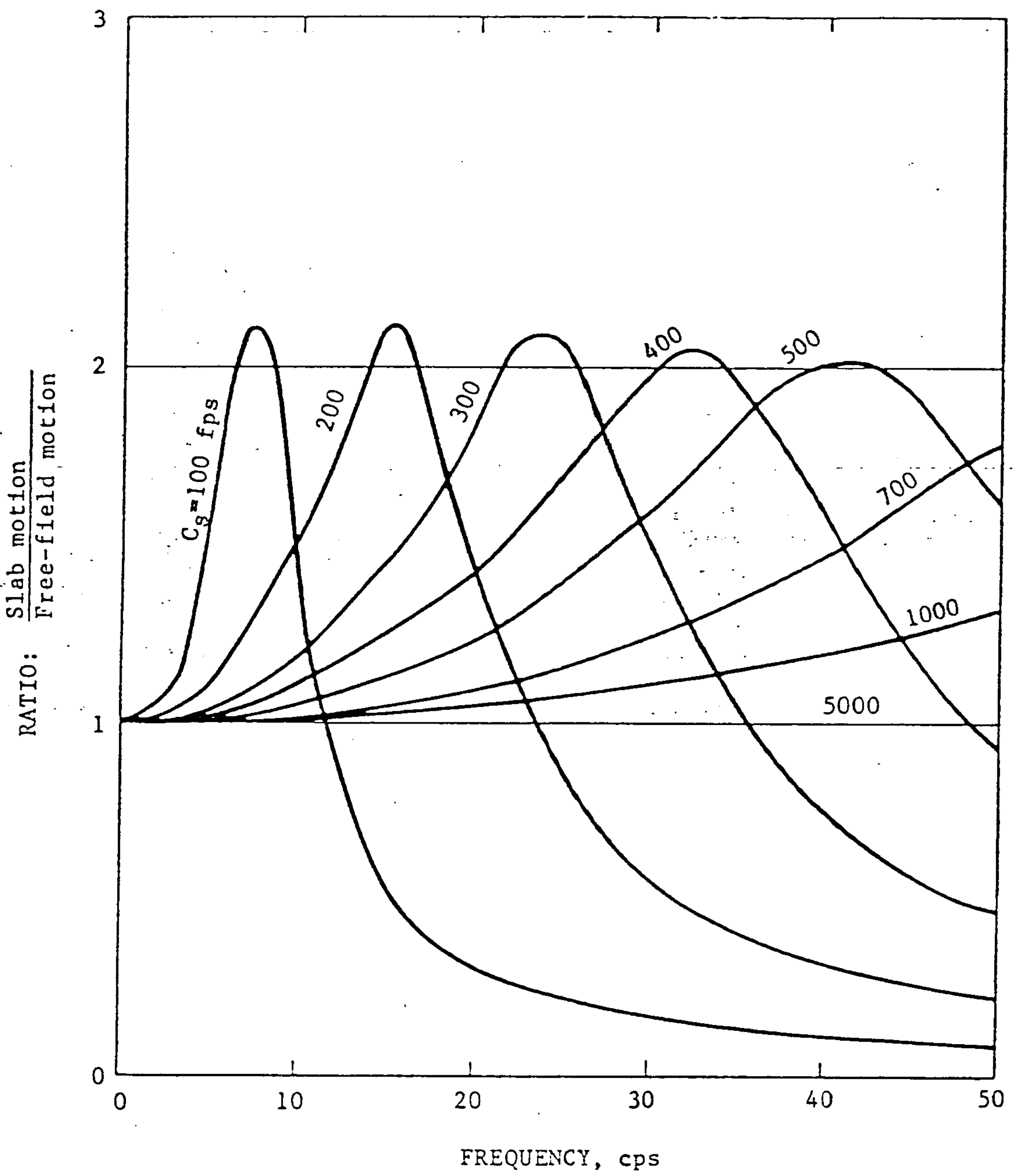


FIGURE 4-9 Horizontal Transmissibility of Standard Installation, as a Function of Shear-Wave Velocity.

4.2 Earthquake Experience

4.2.1 Comparison of Instrumental to Design Spectra

Based on certain physical principles, such as wavelength comparisons and reflections from the free surface and from hard structural surfaces, it is expected that soil-structure interaction effects do exist and are probably appreciable. It is expected that these effects will depend upon the depth of embedment of the structure, the plan area of the structure, and perhaps upon the mass of the structure.

The San Fernando earthquake of February 9, 1971 furnished several examples of the effects of embedment. There were in downtown Los Angeles a number of building pairs, in that two buildings were close together, both had instruments at their respective ground floors, but one of the pair had a basement while the other was founded at ground level. The results for peak acceleration are given in Table 4-2, showing that the effect of the presence of a basement is to reduce the motions, compared to a building without a basement, and by nontrivial amounts. Similar results for structure pairs for which the response spectra could be calculated are given in Figure 4-10. The basement reductions are about the same as those in Table 4-2 for short periods, but there seems to be a consistent increase in the reduction of the motions due to the presence of a basement in the range of periods from about 0.05 sec. to 0.1 sec. for some of the pairs but to 0.2 sec. for most of the pairs. This embedment effect persists until about 0.5 sec., past which most of the basement structures and ground-founded structures seem to respond about the same.

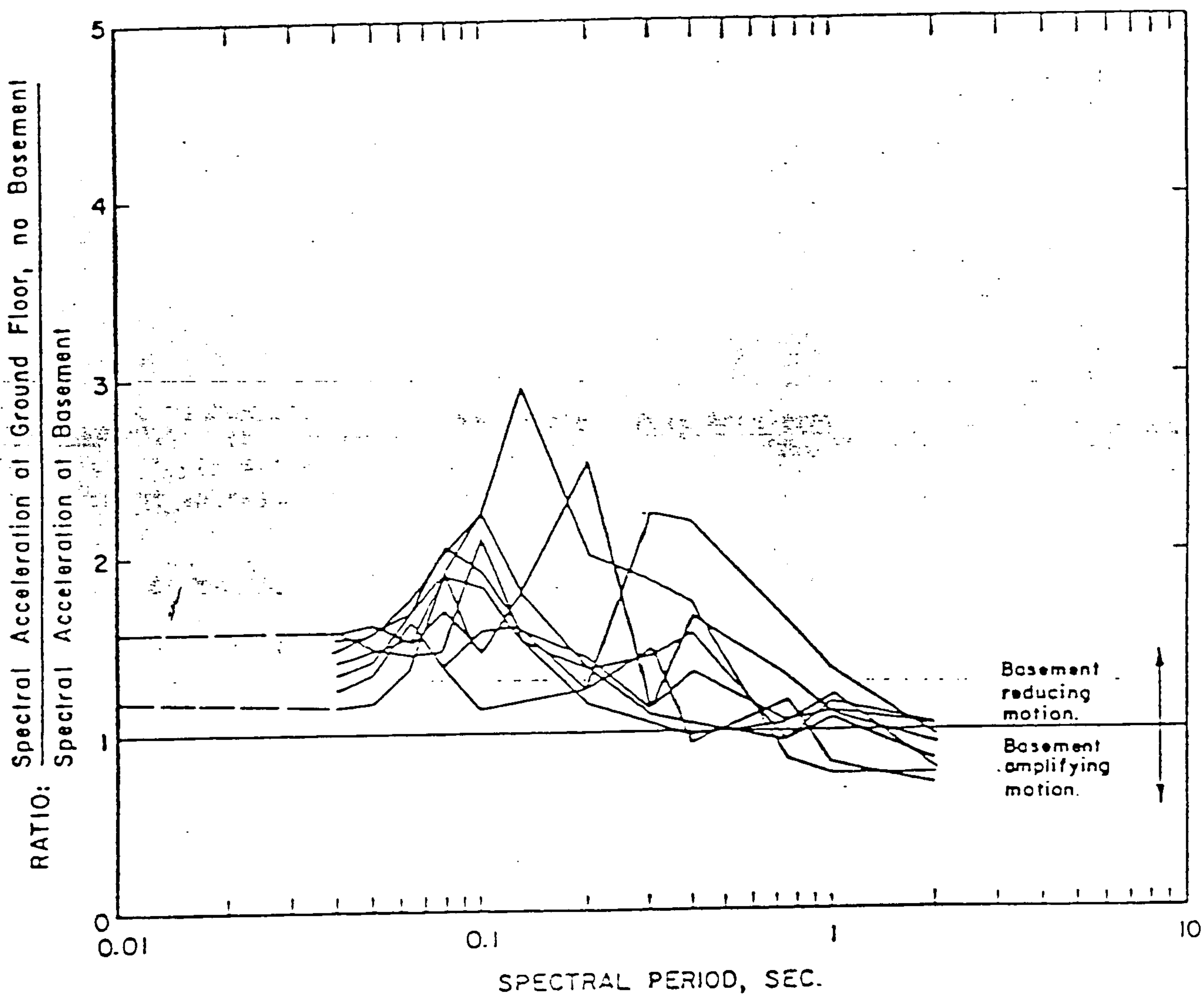
There are a few experiences in which the total soil-structure interaction effects can be observed. In these cases, there was a recording on the ground floor of a structure and a nearby free-field recording. The results for peak motions are given in Table 4-3, showing that the effects of soil-structure interaction are appreciable, with most values being in the range of 1.5 to 2, for peak motions. The results for response-spectra values are given in Figure 4-11. Those data also show that the free-field IPGA's are in general about 1.5 to 2 times the motions in the structure at short periods, with an increase in the reduction of motions in the structure in the period range from about 0.05 sec. to about 0.2 sec. For the case of the reactor (Humboldt), the free-field IPGA's are as much as four times the structure responses.

TABLE 4-2 COMPARISONS OF ZPA's OF BUILDINGS WITH AND WITHOUT BASEMENTS

in 1971 San Fernando Event

Location*	Instr+	Separation, m	Distance, km	Ratio**
14724 Ventura	G	914	15	1.2
15250 Ventura	B			
1260 Orchid	G	450	19	1.9
7080 Hollywood	B			
6430 Sunset	G	100	20	1.6
6464 Sunset	B			
6200 Wilshire	G	500	24	1.6
5900 Wilshire	B			

* In Los Angeles
\+ G = Ground Floor, B = Basement
** Ratio: $\dfrac{\text{Peak Acceleration at Ground Floor, no basement}}{\text{Peak Acceleration at Basement}}$



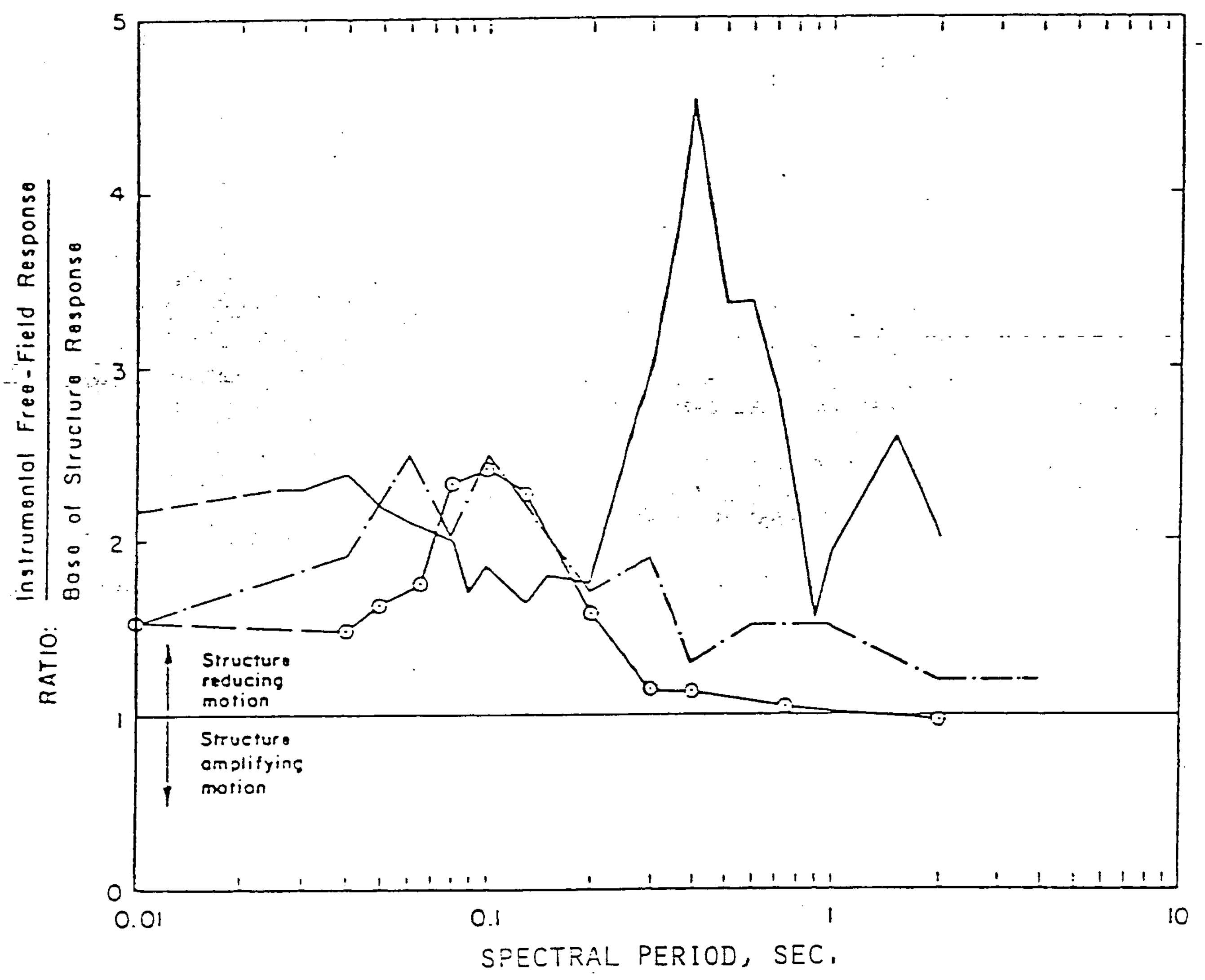
Los Angeles, 09 Feb '71 San Fernando Earthquake
after Campbell (1980).

FIGURE 4-10	Examples of How Structures Without Basements Have Higher Spectral Accelerations than Structures with Basements

TABLE 4.3 COMPARISON OF IN-STRUCTURE ZPA's TO FREE-FIELD PGA's

Earthquake	Date	ML	Dist, km	Location	Ratio*
Kern County	21JL52	7.2	107	Hollywood Storage	1.3
San Fernando	09FB71	6.4	35	Hollywood Storage	1.6
			39	616 Normandie	1.5
			39	3470 Wilshire	1.4
			39	3411 Wilshire	1.4
			39	3550 Wilshire	1.2
Lillis Ranch	03SP75	4.9	18	Pleasant Valley Pump Plant	1.6
Ferndale	07JN75	5.3	25	Humbolt Bay Power Plant	2.2
Guerrero	14MR79	7.6	*	Steel Mill	1.8
Coalinga	02MY83	6.7	*	Pleasant Valley Pumping Plant	1.6

$$* \text{ Ratio: } \frac{\text{Instrumental Free-Field Response}}{\text{Base of Structure Response}}$$



Damping = 5%

Hollywood Storage Bldg: Instrument at - 3m.

Humboldt Power Plant: Instrument at - 25m.

Pleasant Valley Pump Plant: Instrument at - 5m.

FIGURE 4-11	Examples of How Structures Reduce Ground Motion

The available data clearly show a consistent trend of soil-structure interaction in which the structure experiences smaller peak and smaller response motions than the free-field IPGA's. The effect is probably not solely due to the potential errors in the measurements of free-field motions, as discussed in Section 4.1.2 above, because the basement/no-basement pairs show an appreciable effect due to embedment alone.

4.2.2 Component Behavior During Earthquakes

It is expected that structures and components will have reserve capacity when subjected to earthquake motions, and those reserves can be estimated, when compared to actual ground motions. From the foregoing Section 4.2.1, it appears that soil-structure interaction effects can contribute a reserve of about 1.5 to 2. Structural and component designers usually incorporate safety factors of about two to four in their designs, and sometimes higher. Thus the range of excess capacity for structures and components is expected to be in the range of about three to eight or so.

That expectation is borne out by the available evidence. Figure 4-12 shows, as a function of the buildings' natural periods, the design base shear coefficient (ZPA), the peak ground-floor response measured during the February 9, 1971 event, and the peak roof response measured during the same event. The important points are the three sets with the diagonal tick marks, because those are for reinforced concrete structures which sustained some damage. For those three structures, the reserves (ratio of felt to the design acceleration) ranged from about three to five. For those structures which did not sustain structural damage, the ratio of felt to the design accelerations were less than three, as would be predicted by the considerations mentioned above.

The performance of the ENALUF steam power plant through the December 23, 1972 Managua earthquake is an excellent example of the reserve capacity of not-so-modern power facilities subjected to close-in strong ground motions, and is an indication that modern, and particularly nuclear-type, power facilities may have considerable reserve capacity. Figure 4-13 shows the location of the plant, on the shore of Lake Managua, and directly adjacent to one of the two principal faults which ruptured during that event. According to the TERA results (Ref. 1), the IPGA that close to the fault should have been about 0.6 to 0.8g. Although adjacent structures suffered catastrophic damage, the plant came through amazingly well (Ref. 17, 18, and 19): the structure was slightly damaged; equipment which was not secured moved about as would be expected; and the turbines were damaged due principally to the failure of a dc backup electrical supply which was put out of operation by the earthquake. The plant was back in partial operation in a few days, and was completely restored to operation in about three weeks.

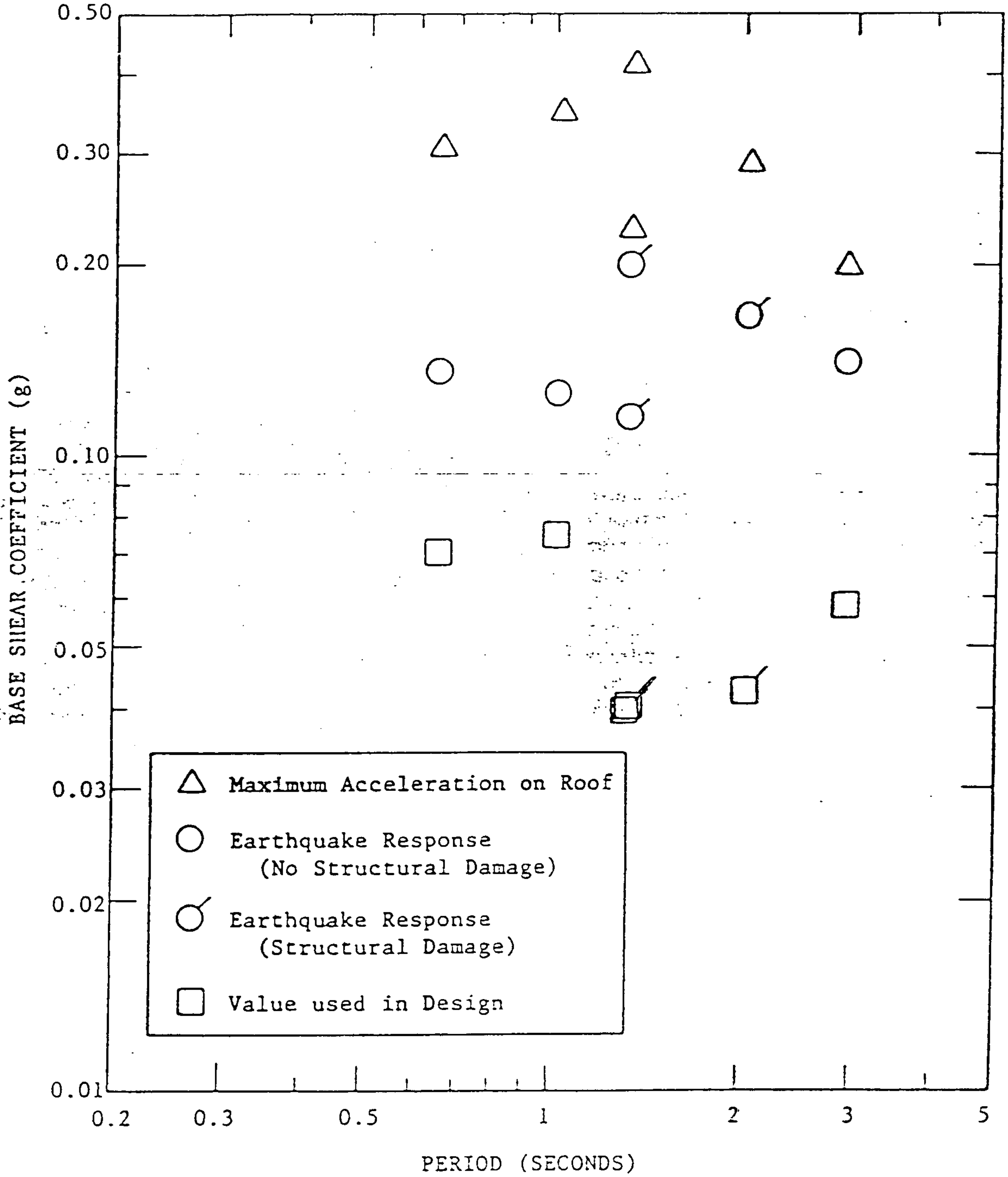


FIGURE 4-12

Capacity of Reinforced Concrete Structures Demonstrated During the San Fernando Earthquake

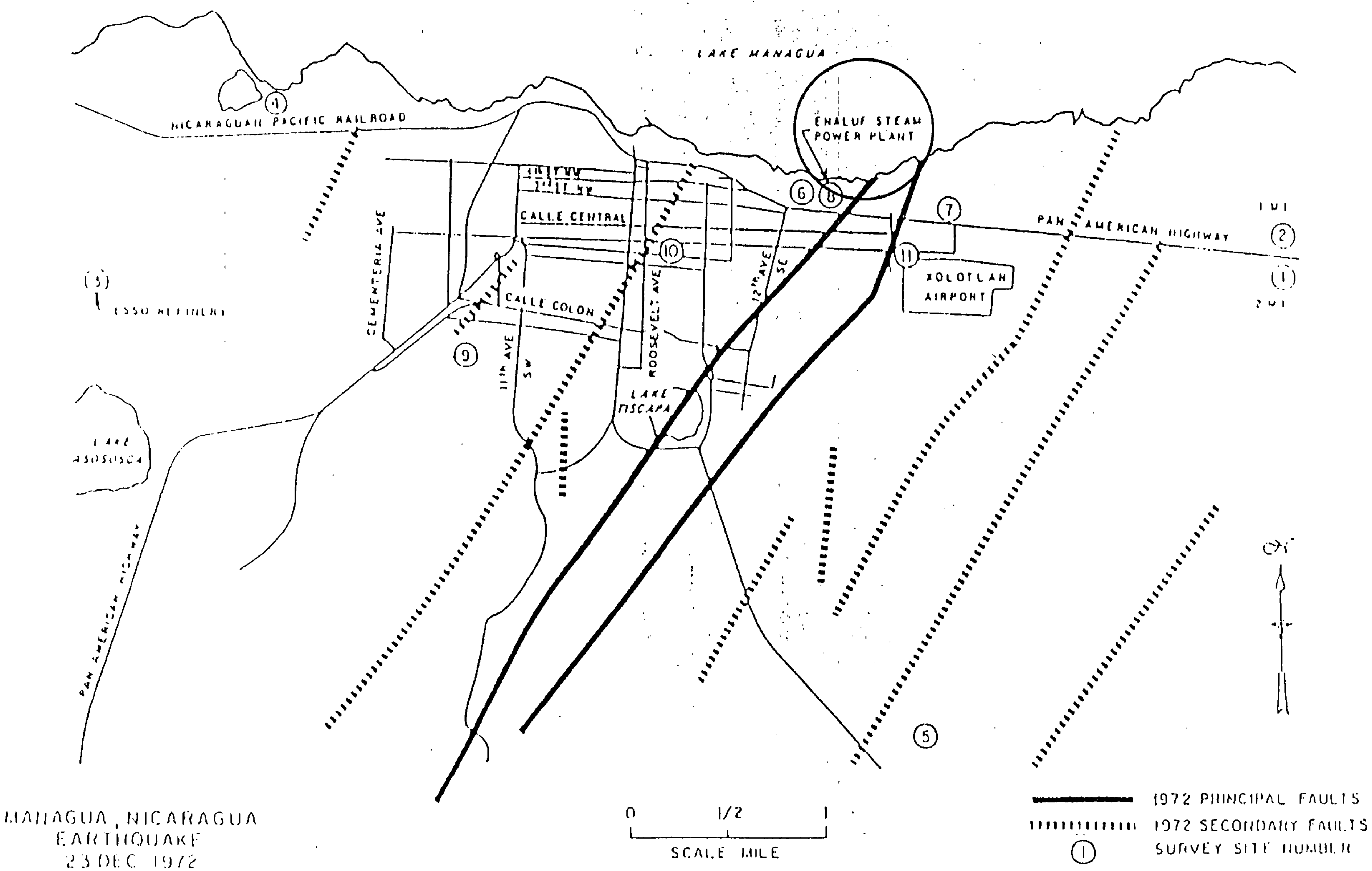


FIGURE 4-13

MAP OF MANAGUA, NICARAGUA SHOWING THE LOCATIONS OF FACILITIES AND BUILDINGS

Another significant experience is that of the El Centro Steam Plant in the Imperial Valley 1979 event. That plant probably experienced IPGA's of 0.5g or so. That experience was studied by the NRC staff, and a comprehensive report has been issued (Ref. 20). Of significance are some of the principal conclusions of that report, as follows:

- "Noteworthy is that the two operating units safely shut down after having experienced a severe seismic environment which generally exceeds that used in the design of nuclear power plants...."

- "Most importantly there were no known malfunctions of electrical control and instrumentation equipment."

- "Except for buckling of a few members in the boiler support frame, significant structural damage was not observed." The report concluded that this condition was "dissimilar to nuclear applications."

There was no damage to high-pressure or high-temperature piping. The report noted that: "the piping systems are hung in a more flexible manner than would be required by current NRC criteria;" and that "in most cases, the piping is supported in a similar manner to older operating nuclear power plants." Thus this experience seems to demonstrate that a conventional plant, probably designed for a ZPA of 0.1 to 0.2g, successfully withstood a much higher IPGA, probably on the order of 0.5g.

A significant case occurred during the May 2, 1983 Coalinga event. There were three electrical facilities close-in to the epicenter. The pertinent information for all three is summarized in Table 4-4 (Ref. 21). The significant case is the Coalinga Substation 2, located very close-in to the epicenter, in an area which must have experienced very high IPGA's. The TERA (Ref. 1) median (0.45g) and 84th percentile (0.67g) are noted in the table, along with the examining engineer's comments. Of interest are the failed transformer anchor bolts. There were four 0.50-in. diameter bolts provided. If the bolts had been designed to the SONGS reanalysis PGA, an area equivalent to four 1.125-in. diameter bolts would have been provided (Ref. 22). Thus, as shown by the calculation on Table 4-4, the resulting SONGS capacity of the anchors would have been more than five times the capacity of the failed bolts. While it is not presently possible to know the loads on the Substation 2 bolts when they failed, it is encouraging to know that a SONGS design at the site would have carried more than five times those loads.

TABLE 4-4

OBSERVED DAMAGES TO ELECTRICAL FACILITIES FROM

COALINGA EARTHQUAKE OF 02 MAY 1983, AND AFTERSHOCKS

All owned by PGE

(From Yanev et al.)

Facility	Epic Dist mile	PGA,g (Tera) Med	84th	Damage Observed
Gates Sub.	14	.12	.19	Minor spilling of oil from large transformers. One broken ceramic bushing on a transformer. Fallen ceiling tiles in control building. No apparent structural damage.
Coalinga Sub. 1	10	.17	.25	Broken ceramic bushing on transformer. Broken bolts on tranformer mounted on steel racks. Sloshed oil from transformer. No apparent structural damage.
Coalinga Sub. 2	1	.45	.67	Partial collapse of unreinforced block structure. Rupture of anchor bolts around transformers and subsequent sliding. Yielding of supports for rack-mounted transformers.

Transformer bolts were four @ 0.50-in. diameter.

SONGS criteria would have been four @ 1.125-in. diameter.

Stress ratio = (1.125/0.50)^2 = 5.1

From these examples it is apparent that modern structures and components have considerable reserve capacity; and, due to the intentional additional conservatisms in the design of nuclear facilities, it should be expected that nuclear facilities would have at least that reserve capacity, and probably more.

4.3 Seismic Risk

The seismic risk at SONGS 1 has been assessed based on the results of probabilistic risk assessments (PRA's) done at other nuclear plants. Two specific evaluations were done. First, published PRA's were reviewed to identify the dominant contributors to seismic core melt frequency based on these studies. These dominant contributors were then compared against the current status of the SONGS 1 plant. Second, an evaluation of the seismic risk at SONGS 1 was performed using data from the PRA study done for Zion by the Seismic Safety Margin Review Program (SSMRP). This evaluation resulted in a conservative numerical value for the seismic core melt frequency at SONGS 1. These two evaluations are discussed in the following subsections.

4.3.1 Lessons Learned From PRA Studies

The purpose of this effort was to compare the SONGS 1 design against the dominant contributors to seismically induced core melt frequency as identified by the seismic risk portions of PRA's prepared for other nuclear plants. The first step in the evaluation was a review of three published PRA's (Zion, Indian Point Units 2 and 3, and Limerick) to attempt to characterize the dominant contributors. The list of dominant contributors to core melt was supplemented based on discussions with PRA experts as well as to include seismically induced failures which are major contributors to offsite consequences. The SONGS 1 design was then evaluated to assess the current plant status for each dominant contributor category. The details of this evaluation are provided in Appendix B.

Based on this review of existing seismic PRA studies, the dominant contributors which were identified are broadly categorized as (1) onsite power, (2) essential water supplies, (3) structures and (4) reactor coolant system. Another significant conclusion from these seismic PRA's is that failure of ductile steel piping is not a dominant contributor to seismic risk. The only types of piping systems that were identified as potential contributors were non-ductile pipe, threaded joints and piping routed between structures.

Each of these main areas identified from the PRA studies are discussed with respect to SONGS 1 in the following paragraphs.

a. Onsite Power

During the 1976-77 outage of the plant, an entirely new safety-related diesel generator onsite power system was added to SONGS 1. This effort included the addition of two new 6000 kw diesel generators complete with their own structures and dedicated support systems.

In response to NRC letters dated January 1, 1980 and July 28, 1980, the seismic reevaluation and upgrade of the support and anchorage of all safety-related electrical equipment (e.g., panels, racks, MCC's, switchgear, inverters, etc.) was initiated. Also included was the anchorage of non-seismic Category 1 ancillary items which could damage the safety-related items identified if the ancillary items were to fail during a seismic event. Where required, modifications have been completed on all items.

One structurally related loss-of-control failure identified was initiated by collapse of the control room ceiling. The SONGS 1 control room ceiling has been replaced with a new 0.67g seismically designed ceiling.

SONGS 1 does not utilize the types of conduit and cable tray supports that were identified as being vulnerable to seismically induced failure at ground accelerations in the range of interest. Moreover, as part of the current seismic reevaluation and upgrade program, a total of approximately 1100 cable tray supports and 7300 conduit supports were evaluated, and the majority of required modifications, over 1700, have been implemented to upgrade the seismic capability of these systems. In addition, testing of raceways similar to those at SONGS 1 demonstrate their capability to safely withstand a 0.67g seismic event.

Therefore, all failure mechanisms associated with the onsite power system at SONGS 1 have been effectively eliminated from being an important contributor to seismic core melt frequency for ground accelerations up to 0.67g.

b. Essential Water Supplies

At SONGS 1 a new seismically qualified auxiliary feedwater tank has been designed and constructed to withstand a 0.67g seismic event. To supplement the RWST, an additional source of borated water for primary side makeup will be designed and constructed to withstand a 0.67g seismic event. This source will consist of a crosstie from the spent fuel pool to the charging pumps. Accordingly, these items are effectively eliminated from being an important contributor to seismic core melt frequency for ground accelerations up to 0.67g.

c. Structures

During the Sphere Enclosure Project, Standby Power Addition Project, Seismic Backfit Project, as well as part of the current seismic reevaluation program, safety-related structures in the plant have been analyzed to assess their adequacy to withstand a 0.67g seismic event. Where modifications were identified, they have been implemented such that all structures required to ensure safe shutdown are capable of withstanding a 0.67g seismic event. Separation of structures was also evaluated. The evaluation demonstrated that impact between structures will not occur at ground accelerations up to 0.67g.

To assess the seismic capacity of existing masonry walls in the plant, extensive nonlinear analyses were performed. Then, a testing program was conducted to verify the results of the nonlinear analyses. The evaluation showed that the existing walls will all withstand a 0.67g seismic event without collapse. In addition, a few masonry walls whose deflections may have affected attached electrical trays and conduit have been modified to limit their deflections during a 0.67g seismic event.

Except for limited portions of the control administration building, ventilation equipment building and isolated footings of the turbine building, all plant structures are founded upon native San Mateo sand that is not susceptible to failure at 0.67g. For the three areas of the plant site where in-situ soils are a concern, evaluations have been performed and (where necessary) modifications have been initiated or completed to ensure that structures in these areas required for safe shutdown will withstand a 0.67g seismic event. The only affected essential systems and equipment required for safe shutdown are the auxiliary feedwater pumps foundation and 480 V switchgear room slab. These areas will be addressed prior to return to service to ensure that consequences of settlement of the slabs will not impair the integrity of the supported equipment.

As a result of the extensive analysis and modifications at SONGS 1, the essential structures have been demonstrated to be adequate or upgraded to effectively eliminate these items from being major contributors to seismic core melt frequency for ground accelerations up to 0.67g.

d. Reactor Coolant System

During the period from 1972 to 1977, the reactor coolant loop piping, reactor vessel, steam generators, pressurizer, reactor internals, CRDM's, and supports were reevaluated and upgraded as necessary to ensure their ability to withstand a 0.67g seismic event. Included in this effort was a concurrent reevaluation of the containment sphere and reactor building to ensure that the supporting structures for the RCS can also withstand a 0.67g seismic event.

As a result of these analyses and modifications, the SONGS 1 reactor coolant system has been upgraded to effectively eliminate this item as a major contributor to seismic core melt frequency for ground accelerations up to 0.67g.

d. Piping

Seismically induced piping system failures have generally not been found to be a dominant contributor to seismic core melt frequency except in special circumstances. These special circumstances are discussed below with respect to the design of SONGS 1:

With the exception of a small amount of cast iron pipe, all SONGS 1 safety-related piping systems are welded ductile steel in nature. Historical experience has shown that such systems have a very high seismic withstand capability. This experience is reflected in seismic PRA fragility data, all of which indicate that the median ground acceleration capacities of welded ductile steel piping systems are sufficiently high that - even considering uncertainties - the probability of failure at ground accelerations of 0.67g and lower is small and not a major contributor to seismic core melt frequency even for piping systems which were designed for an SSE much lower than 0.67g.

SONGS 1 contains a limited amount of buried cast iron pipe associated with the salt water cooling (SWC) system. Cast iron pipe is known to be susceptible to failure under seismic load. To ensure the safe shutdown capability of SONGS 1, an alternate means of heat removal will be provided for systems required for safe shutdown. Thus, this item will be eliminated as a potential contributor to seismic core melt frequency for SONGS 1.

SONGS 1 utilizes no threaded joints in process piping for essential systems. Therefore, this item is eliminated as a potential contributor to seismic core melt frequency for SONGS 1.

SONGS 1 contains some essential piping spanning buildings on separate foundations. The evaluation of piping spans between structures that are required to attain safe shutdown will be specifically addressed prior to return to service. Thus this item will be eliminated as an important contributor to seismically induced core melt frequency for SONGS 1.

Based on this evaluation, the seismic upgrade work completed to date on SONGS 1 has concentrated on the most important contributors to seismic core melt frequency. Virtually every dominant contributor category has been eliminated by work already completed. On the basis of these comparisons, SONGS 1 has a low probability of seismically induced core melt for peak ground accelerations up to and including 0.67g.

4.3.2 SONGS 1 Seismic Risk Evaluation

The purpose of this effort was to perform an assessment of the frequency of seismic event induced core damage for SONGS 1. This evaluation relies heavily on the Seismic Safety Margin Review Program (SSMRP) and data generated by Lawrence Livermore National Laboratory. Key features of SONGS 1 capability to preclude core damage from earthquakes have been modeled in sufficient depth to include known important contributors to seismic risk. The analysis includes substantial conservatism. Where required, estimates of SONGS 1 specific response have been included, particularly estimates of location specific accelerations. Failure probabilities have been estimated from SSMRP directly for piping, inferred from SSMRP fragility data for components, and estimated conservatively for SONGS 1 structures specifically.

This study has been undertaken to evaluate the seismic risk for the unit based upon a combination of data from other studies (SSMRP) and a plant specific fault tree model of risk sensitive features. The frequency of earthquakes is taken to be higher than Zion (the study plant of SSMRP) for SONGS 1, and plant features of SONGS 1 are then evaluated conservatively to determine the effect of these plant features and their design level on composite risk.

The general approach involves performing a seismic risk evaluation utilizing a mixture of inferred and site/plant specific data. A fault tree model similar to that of SSMRP was constructed and quantified. The result is a single value, or point estimate, most closely representing a mean value. The component parts of the analysis are shown below and discussed in detail in Appendix C:

- The hazard curve utilized is a SONGS 1 site specific .5 probability of exceedence curve.

- System features and fault trees are SONGS 1 specific.

- Location specific accelerations are estimated based on SONGS 1 specific data.

- Component fragilities are from SSMRP.

- Structural failure is assumed to occur as a step function for the earthquake band above 0.67g, the design value for structures.

- Piping failure probabilities are estimated from SSMRP directly.

- Random failure rates are from WASH 1400 and NUREG CR 1278.

- Uncertainty is not estimated but is large as in all such studies.

Based on this analysis it is concluded that the earthquake risk at SONGS 1 is approximately equal to the earthquake probability at Zion (2×10^{-5}/year). In the review and system modeling no specific area of concern was identified which SCE has not addressed or is not addressing as part of the return to service program. Much data from SSMRP has been adopted and used directly; in all cases there is strong reason to believe that such use is conservative. Finally, the uncertainty is large, but the results are consistent with SSMRP.

4.4 Summary of Seismic Hazard

The foregoing Sections 4.1, 4.2 and 4.3, have highlighted (1) conservatisms which apply to structures and components in general and at the SONGS site and (2) the seismic risk at the SONGS site. Specifically, in the first area discussed above (1) the original design PGA, 0.5g, is a one-sigma value, with a very long return period, (2) the reanalysis PGA, 0.67g, is a two-sigma value, with an even longer return period, (3) the present practice of founding many of the seismographs tends to bias the recordings, making them and the response spectra calculated from them too high, (4) due to soil-structure interaction effects, there is an additional conservatism because structures respond at short periods less than the free field, and (5) there is a reserve capacity inherent in the design and construction of modern structures and components. Although these conservatisms have not been quantified for the SONGS 1 structures and components, it is clear that they should be accounted for on a generic basis when considering the seismic risk of SONGS 1.

In the second area discussed above, it has been shown that virtually every dominant contributor to seismically induced core melt frequency has been eliminated at SONGS 1 by work already completed and that a conservative assessment of seismically induced core damage at SONGS 1 is comparable to that calculated for other plants and is less than the safety goal. Of particular note is that the primary area of the seismic reevaluation program which has not been completely implemented to date, i.e., piping systems, was not identified in either of the evaluations as a significant contributor to seismic risk.

Based on these considerations, namely, that there are significant conservatisms in the design and that the seismic risk is low, the seismic hazard should no longer be a significant concern for the SONGS 1 plant as a whole.

5.0 SAFE SHUTDOWN

The discussion of safe shutdown is separated into four areas: (1) safe shutdown systems, (2) acceptability criteria, (3) implementation procedure and (4) piping review examples. These discussions are provided in the following subsections.

5.1 Safe Shutdown Systems

As previously indicated, the basic premise of this plan is that all structures and systems whose failure could cause an accident and/or whose function is required to obtain and maintain a hot standby condition will be available following a 0.67g earthquake. Based on a review of the plant, the systems required for safe shutdown are: (1) reactor coolant pressure boundary, (2) main steam and feedwater lines, (3) charging, and (4) auxiliary feedwater. These systems are discussed in the following paragraphs.

5.1.1 Reactor Coolant Pressure Boundary

The reactor coolant pressure boundary (RCPB) includes the primary coolant loop and all connected lines up to the first isolation valve. As noted in Section 2.0, the primary coolant loop and main components have been analyzed and upgraded to 0.67g. Seismic upgrade of the RCPB ensures that an earthquake will not cause a loss of coolant accident.

5.1.2 Main Steam and Feedwater Lines

The main steam lines are included from the steam generators to the turbine stop valves and to the atmospheric dump valves. The feedwater lines are included from the steam generators to the feedwater control valves. All branch lines greater than 2 inches will be included. Seismic upgrade of these lines ensures that an earthquake will not cause a main steam or feedwater line break.

5.1.3 Charging

Charging for makeup to the reactor coolant system will be provided from the charging pumps to the reactor coolant system through the reactor coolant pump seal injection lines (FCV's 1115D, 1115E, 1115F) and the recirculation lines (MOV's 356, 357, 358). The source of makeup water will be a new connection installed from the charging pump suction to the spent fuel pool. Seismic upgrade of the charging ensures a means and source for makeup to the primary coolant loop.

(6) Soil structure interaction effects are conservatively considered.

(7) The time history used in the development of in-structure response spectra conservatively envelopes the design response spectrum.

When these additional conservatisms are considered in combination with the fact that the seismic hazard is not a significant concern as discussed in Section 4.0, review of the safe shutdown systems to the acceptability criteria defined herein provides adequate assurance that these systems will be available following a 0.67g earthquake. Specific discussions of each of the acceptability criteria are provided in the following subsections.

5.2.1 Containment Building Response Spectra

The response spectra used in piping analyses inside the containment building were generated using conservative methodology for development of in-structure response spectra. As such these in-structure response spectra show substantial amplification above ground motion. However, as discussed in Section 4.2.1, actual earthquake recordings indicate that the effects of soil structure interaction are appreciable. Free-field instrumental ground motion are in general about 1.5 to 2 times the motions in large structures (such as the SONGS 1 containment building) at short periods. This fact was specifically demonstrated for the SONGS 1 containment building in an analysis (Ref. 23) done by the Lawrence Livermore National Laboratory (LLNL). As shown in Figure 5-1 the LLNL spectra at the foundation are substantially below the seismic reevaluation in-structure spectra and even the Housner ground motion. Based on the recorded data and the LLNL results, the acceptability criteria will consider a reduction of one half of the in-structure response spectra inside the containment building. As can be seen from Figure 5-1, this still provides substantial margin over the LLNL spectra.

5.2.2 Higher Damping for Piping Analysis

The seismic reevaluation program piping analyses have considered damping values of 2% for small piping and 3% for large piping in accordance with NRC Regulatory Guide 1.61. The Task Group on Damping Values of the PVRC Technical Committee on Piping Systems has recently completed a review of a significant data base of damping tests. The results of the review clearly indicate the justification for increasing the present damping values for seismic design of nuclear power plant piping above those specified in Regulatory Guide 1.61.

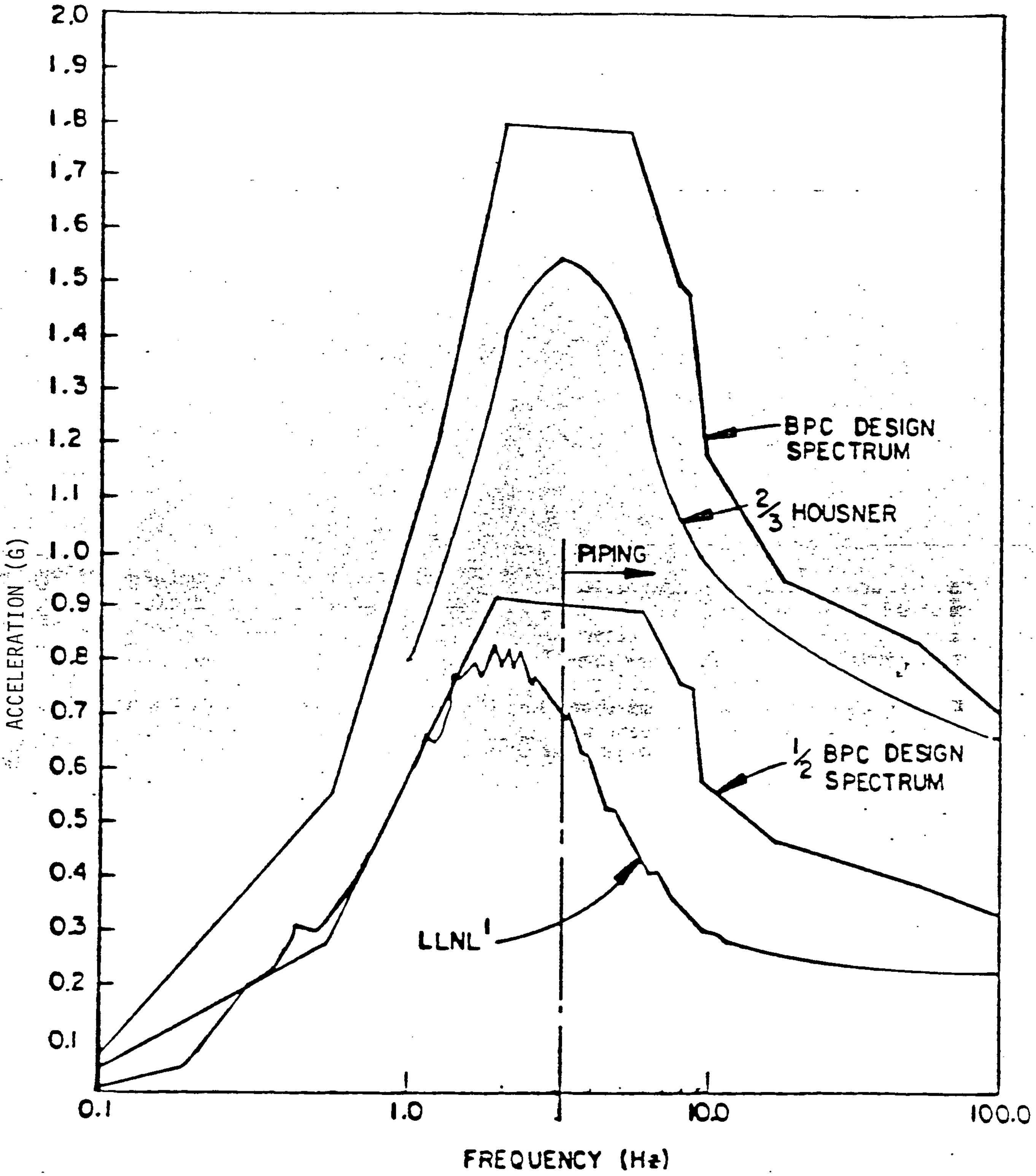


FIGURE 5-1 Containment Foundation Response Spectra (Taken from LLNL Draft Report Date June 18, 1982)

Based upon their evaluations, the current recommendation of the Task Group members is that damping of 5% is acceptable to 10Hz linearly decreasing to 2% at 20 Hz and held constant at 2% to 33Hz. Figure 5-2 illustrates the position in relation to present Regulatory Guide 1.61 values. Recommendtions are for both OBE and SSE and are independent of pipe diameter. Based on these recommendations, the acceptability criteria will consider this increase in the damping values.

5.2.3 Spectra Shifting

In-structure response spectra have been developed in the seismic reevaluation program in accordance with NRC Regulatory Guide 1.122. This includes all of the conservatisms associated with peak broadening. However, as part of the acceptability criteria, an alternative method of broadening of the structural peaks can be based on a probabilistic approach. In the particular case where there is more than one piping frequency located within the frequency range of a widened spectrum peak, the floor spectrum curve may be more realistically applied in accordance with the following criterion.

Based on the fact that the actual natural frequency of the structure can assume only one single value within the frequency range defined by $f_j \pm \delta f_j$, but not a range of values, only one of these piping modes can respond with the magnitude of the peak spectral value. Therefore, seismic analysis of piping systems using the broadened floor design response spectra may be accomplished by the following alternative:

1. Determine the natural frequencies (fe) of the piping system to be qualified.

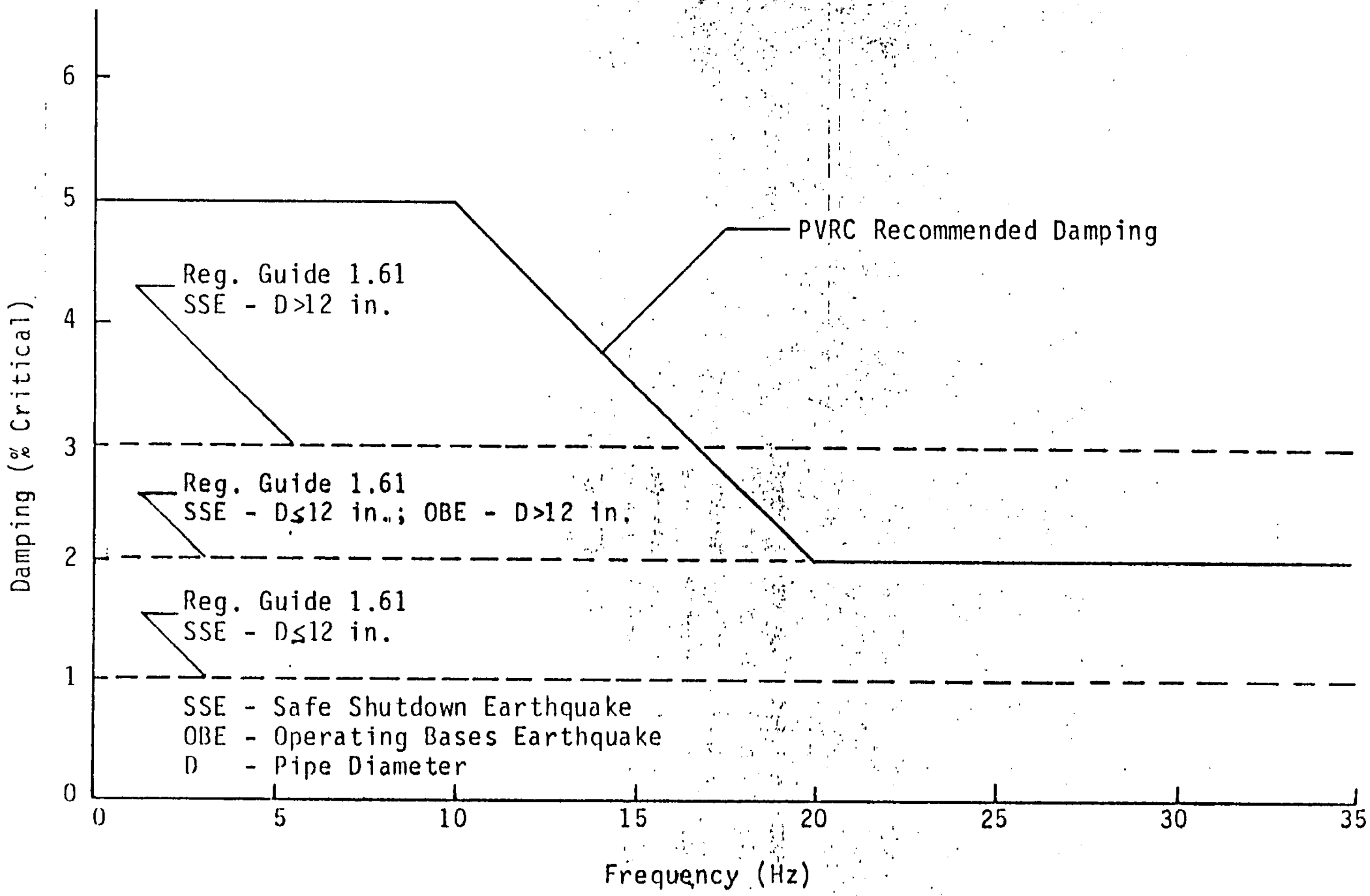
2. Consider all piping natural frequencies in the interval

   $$f_j - .15\ f_j \leq (f_e)_\eta \leq f_j + .15\ f_j$$

   where f_j is the frequency of maximum acceleration in the unbroadened spectra, and η = 1 to K.

3. The piping system shall then be evaluated by sequentially performing K + 3 analyses using the unbroadened floor design response spectrum and also the unbroadened spectrum modified by shifting the frequencies associated with the spectral values by +.15 f_j - .15 f_j and

FIGURE 5-2 : Pipe Damping Values



$$\frac{(fe)_{\eta} - f_j}{f_j}, \text{ where } \eta = 1 \text{ to } K.$$

4. The results of these separate analyses shall then be enveloped to obtain the final resultant desired (pipe stress, support loads, accelerations, etc.)

If no piping system natural frequencies exist in the interval associated with the maximum acceleration peak, then the interval associated with the next highest peak shall be used in the above procedure.

This approach has been accepted for inclusion in Standard Review Plan 3.9.2.

5.2.4 Functionality Criteria for Piping and Pipe Supports

The acceptance criteria for the seismic reevaluation program piping analyses were the Balance of Plant Mechanical Equipment and Piping (BOPMEP) Criteria. These criteria were based on the ASME Code and the NRC Guidelines. Substantial evidence now exists that many of the provisions in these criteria are overly conservative when applied to short-term dynamic load cases, such as the DBE. Diverse nuclear industry groups, such as the ASME Code committees, Electric Power Research Institute (EPRI), and groups in foreign countries (e.g., Japan and West Germany), have been investigating this issue for the past few years. The final resolution of the issues and changes to the ASME Code are still several years away, however, the trend in results is clear: namely, piping systems subjected to short-term dynamic loads can maintain integrity and functionality at stress levels well above ASME Code allowables without a decrease in safety margins. Therefore, the acceptability criteria will include the consideration of a functionality criteria for piping, pipe supports and equipment.

a. Functionality Criteria for Piping

The objective of the functionality criteria is to define a stress level below which the piping system is assured of performing its safe shutdown function following a DBE. For consideration of the 2/3 g level earthquake defined as the DBE for SONGS 1, the criteria require that pipe stresses satisfy the following equations:

$$\sigma_{pr} + \sigma_{dw} + \sigma_{DBE} \leq 2\sigma_y \quad \text{(carbon steel)}$$

$$\sigma_{pr} + \sigma_{dw} + \sigma_{DBE} \leq 2.2\sigma_y \quad \text{(stainless steel)}$$

where

σ_{pr} = stress due to internal design pressure

σ_{dw} = stress due to gravity loads

σ_{DBE} = stress due to inertia effects of the DBE

σ_y = material yield strength at temperature (as defined in Appendices to ASME Code)

All stress terms include ASME Code-defined stress intensification factors and are equivalent to stresses calculated in conventional nuclear piping stress analysis. The satisfaction of these equations at all locations on the piping systems is sufficient to demonstrate both pressure boundary integrity and ability to pass rated flow through the pipe.

The concept of using less restrictive piping stress allowables to justify continued operation has been applied before. Commonwealth Edison used a 2.0 σ_y stress limit in a similar situation during their IE Bulletin 79-14 program. The BWR Mark I containment program used a factor of safety of two against failure to justify continued operation of twenty-two plants over a seven year period. In each case, the design condition was a low probability event and the criteria was justified through plant-specific analyses. The proposed application meets both of these conditions: the 2/3 g earthquake at SONGS-1 has a large return period and the criteria is justified through analyses on SONGS-1 piping as described in Appendix D.

Finally, it is worth noting that all studies investigating short-term dynamic loading of piping reach the same conclusion: even though stresses in the piping far exceed ASME Code-stress limits, no failures (either in integrity or functionality) are observable under extreme seismic loading. Such studies are based on analytical results (e.g., EPRI study for dynamic stress limits), experimental results (e.g., German scale model tests of piping), and empirical evidence (e.g., actual recordings from El Centro steam plant).

The piping stress criteria provides for a higher stress allowable than currently defined in the ASME Code for primary stresses. For carbon steel, the limit on elastically calculated stresses is twice the material yield stress; for stainless steel the allowable is 10% higher. Stress allowables greater than the yield stress are acceptable because the DBE is essentially a deformation- controlled event for piping systems. That is, load redistribution throughout the system will occur during the event so that failure of the piping, if it were to occur, could occur only through excessive deformation. Stresses due to thermal expansion and seismic anchor motions (SAM) are excluded from consideration based on the following:

- Both thermal and SAM are treated as secondary stresses in the ASME Code. These stresses are due to constraint provided by piping supports against free motion of the piping and are, hence, limited in the amount of deformation they can induce in the pipe. In conventional analysis, these effects are not included in primary stress equations but are contributors to fatigue; a failure mode not of concern in this evaluation of the DBE.

- The extreme case of SAM stresses induced by differential motion of adjacent buildings has been treated as a special case. In fact, all safe shutdown piping systems have already been checked for SAM stresses and been shown acceptable in the final configuration.

- Sufficient margin has been shown in the nonlinear analyses to accommodate thermal strains.

- Pipe support configurations with fewer restraints are likely to have lower stresses than fully restrained systems for thermal and SAM load cases.

The higher allowable for stainless steel piping is appropriate since yield properties for stainless steel are normally at least 10% greater than those listed in the ASME Code and the margin between ultimate and yield strengths is far greater for stainless steel than for carbon steel.

The nonlinear analysis program is discussed in detail in Appendix D. The program involved selection of two representative piping systems which were highly stressed in the current configuration. One system was a 2 1/2 inch diameter carbon steel line in the auxiliary coolant system. The other was an 8-inch diameter stainless steel line leading to the recirculation heat exchanger. Linear elastic analyses were performed to verify the piping models, determine the critical direction of input motion, and define the scale factor on the seismic input to bring stresses to the functionality criteria limits. A nonlinear analysis using the ANSYS program was then performed to verify that pipe strains remained within acceptable limits.

The results show a decrease in moments throughout the system when nonlinear results are compared to the linear elastic results. Support loads are also typically decreased. Maximum plastic strains remained below 2%, well within accepted limits for collapse and loss of functionality. Moments on elbows were also at their maximum, only 7% above the ASME Code defined collapse moment. It is worth noting that the ASME Code collapse moment assumes a ductility of two, which is extremely conservative when compared with experimental data on elbows. Therefore, the analyses confirm that for representative piping systems at SONGS 1, linear elastic stresses of up to $2.2\sigma_y$ correspond to very limited deformations and no impairment of functionality.

b. Functionality Criteria for Pipe Supports

Pipe support functionality throughout the DBE is not required to assure the ability of the piping system to safely shut down the plant. That is, some supports may be inactive for a portion of the earthquake response or may catastrophically fail in limited numbers, and the piping would still meet its functional requirements. The criteria for pipe supports involves a multi-step process to verify that two failure modes are avoided:

1) Large, unsupported spans of piping will not lead to plastic collapse of the span.

2) Load redistribution to supports will not cause successive failure of adjacent supports (zipping).

The methodology for evaluating pipe supports will consider the use of an energy balance approach. The energy available in the earthquake is compared to the amount of strain energy involved in large plastic deformations of piping and supports. Energy available in the earthquake is derived from the input floor spectra. Strain energy is derived from standard elasto-plastic theory. The energy balance approach was used in Reference 24 to demonstrate large margins to failure of conventional piping design when subjected to typical earthquakes.

Pipe strain energy is estimated using static loading and static deflection patterns. A span of piping is expected to respond predominantly in a single mode, and the static strain energy is a reasonable approximation for the dynamic case. The dynamic deflection patterns may differ from the static ones for more complex configurations. However, multi-modal response is expected to provide additional margins against collapse since simultaneous formation of required multiple hinges in a span should require more input energy due to non-uniform distribution of loads and moments.

The pipe span strain energy is the energy absorbed from an unloaded state to the formation of a plastic collapse mechanism. The span is initially assumed to have fixed-fixed boundary conditions. Elastic strain energy is calculated in two steps. First, the energy absorbed upon full formation of plastic hinges at the support points is calculated. Second, the hinges are assumed to be elastic-perfectly plastic, such that the plastic moment is constant. The energy absorbed until full formation of the midspan hinge is then calculated, based upon this assumption. Plastic strain energy is calculated by approximating the rotation of the hinges at the support points, using an elastic equation. This underpredicts the actual rotation, including nonlinear effects, and therefore conservatively underpredicts the maximum strain energy the beam can absorb. Figure 5-3 illustrates the beam collapse and strain energy equations used.

The strain energy equations can be expressed in terms of piping cross-section parameters and span length. The total strain energy in the beam is obtained by:

$$U = \text{Strain Energy} = (A)\,\frac{IL}{D_o^2}$$

where,

I = Pipe section moment of inertia
L = Pipe span length
D_o = Pipe section outer diameter
A = Variable depending on load state and material properties. For a typical fixed-fixed carbon steel piping span subjected to a uniformly distributed load, $A = 17.63$ at first yield at supports; $A = 177.6$ at full yield of midspan.

5.1.4 Auxiliary Feedwater

The auxiliary feedwater system will include piping from a new seismically qualified auxiliary feedwater tank through the two auxiliary feedwater pumps to the three main feedwater lines. The steam line to the turbine driven auxiliary feedwater pump will be included. Seismic upgrade of the auxiliary feedwater system ensures a means for cooling the primary system through the steam generators.

5.2 Acceptability Criteria

As indicated in Section 2.0, extensive modifications have been made to the plant. These modifications have been implemented in accordance with the seismic reevaluation criteria previously discussed with the NRC staff. For the remaining modifications addressed in this plan, mainly in the area of piping systems, several alternative criteria have been developed which will be applied to these systems. These criteria include (1) reduction of in-structure response spectra in the containment building, (2) use of higher damping for piping analyses, (3) use of spectra shifting as an alternative to peak broadening, (4) use of functionality stress limits for piping and pipe supports, (5) use of inelastic criteria for the evaluation of structural members and (6) use of test data and walkdowns to qualify small bore piping.

Although implementation of these criteria will reduce some of the conservatism in the current piping analyses, substantial conservatisms still remain in the piping systems. Some of these conservatisms are:

(1) Strain rate effects, which increase the yield strength in dynamic load cases, are neglected.

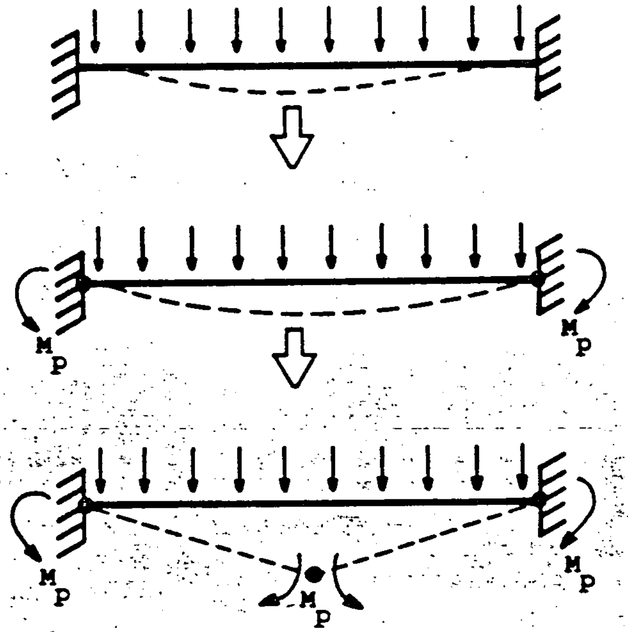
(2) Stress intensification and flexibility factors are taken directly from the ASME Code. These factors tend to be extremely conservative in that they predict yielding at lower load values and do not account for redistribution of loads to other components after yielding.

(3) Material strengths are from the ASME Code and are typically lower bound values.

(4) Component wall thicknesses are typically greater than the nominal dimensions specified.

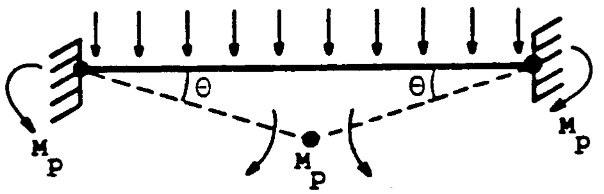
(5) Internal pressure, which in the criteria decreases the margin to failure, can increase the resistance of components such as elbows to collapse.

(1) Elastic Strain Energy



$$U_e = \int_0^L \frac{M^2(x)\,dx}{2EI}$$

(2) Plastic Strain Energy



$$U_p = 2M_p\Theta$$

$$\Theta = \int_0^{L/2} \frac{M(x)\,dx}{EI}$$

$$U_p = 2M_p \int_0^{L/2} \frac{M(x)\,dx}{EI}$$

FIGURE 5-3 Calculation of Strain Energy for Piping Span

The beam models will respond at their fundamental frequency since in-phase motion will be applied; therefore, it is desirable to express the earthquake input energy as a function of frequency of system response. A convenient measure of earthquake energy at a particular frequency is the Fourier amplitude spectrum. The Fourier amplitude spectrum $F(\omega)$ of an earthquake at time t_1 is given by (Reference: Clough & Penzien, Introduction to Structural Dynamics).

$$|F(\omega)| = \left[\left[\int_0^{t_1} \ddot{v}_g(\tau)\cos\omega\tau\, d\tau\right]^2 + \left[\int_0^{t_1} \ddot{v}_g(\tau)\sin\omega\tau\, d\tau\right]^2 \right]^{1/2}$$

The maximum total energy of an earthquake (combined kinetic and strain energy), E(t), can be expressed in terms of the Fourier amplitude spectrum:

$$E(t) = \frac{|F(\omega)|^2 m}{2}$$

where,

$F(\omega)$ = Fourier amplitude spectrum
m = Total mass of the piping system

The Fourier amplitude spectrum of an earthquake is usually evaluated at the end of the time history. It is the measure of the average amplitude of the Fourier components at each frequency chosen. The maximum energy state may occur some time before the end of the earthquake when average amplitudes at certain frequencies peak. Using peak amplitudes is very conservative, and the average energy values calculated at the end of the earthquake is a more rational measure of its total energy.

The earthquake energy imparted to a pipe span is given in terms of the system's mass. Thus, the energy can be expressed in terms of the pipe span parameters:

$$E(t) = \frac{|F(\omega)|^2 m}{2} = \frac{|F(\omega)|^2 WL}{2g} = \frac{|F(\omega)|^2 WL}{772.8}$$

where,

$F(\omega)$ = Fourier amplitude spectrum, in/sec
W = Span weight, lb/in
L = Span length, in
g = Gravity, lb-sec^2/in

The Fourier amplitude spectra are given as a function of frequency; hence, it is desirable to express the earthquake energy as a function of frequency to determine the distribution of energy for all frequencies. The elastic frequency of a fixed-ended span can also be expressed in terms of the span length and pipe cross-sectional properties:

$$f_1 = \frac{366{,}300}{L^2}\sqrt{\frac{I}{W}}$$

where,

f_1 = Fundamental frequency, Hz
I = Pipe section moment of inertia, in^4
W = Pipe weight, lb/in
L = Span length, in

Thus, the Fourier energy spectrum may be generated using the following equation:

$$E(t) = \frac{F(\omega)^2}{\sqrt{f_1}} \cdot \frac{I^{.25}\,W^{.75}}{1.277}$$

The maximum pipe span strain energy can also be expressed in terms of span frequency for specific material properties, load distribution, and boundary conditions. These strain energies for carbon steel fixed-fixed pipe spans subjected to uniformly distributed loads are provided below.

At first yield of the pipe at the supports:

$$U = \frac{10{,}670}{\sqrt{f_1}} \; \frac{I^{1.25}}{W^{.25}D_o^2}$$

At collapse of the span:

$$U = \frac{107{,}490}{\sqrt{f_1}} \; \frac{I^{1.25}}{W^{.25}D_o^2}$$

For a selected piping cross-section, piping span strain energy capacity and Fourier energy for various earthquake motions may be plotted simultaneously to show relative energy magnitudes as a function of frequency. Figure 5-4 shows the absorbed strain energy spectra at first yield and at collapse for 8-inch Schedule 40 pipe spans versus earthquake energy spectra for a typical in-building motion and the Parkfield earthquake record. These comparisons can be used to verify that the strain energy to collapse is greater than the available earthquake energy. To provide additional assurance that adequate margin against collapse exists, a factor of 2.0 will be maintained in this evaluation between strain and earthquake energy.

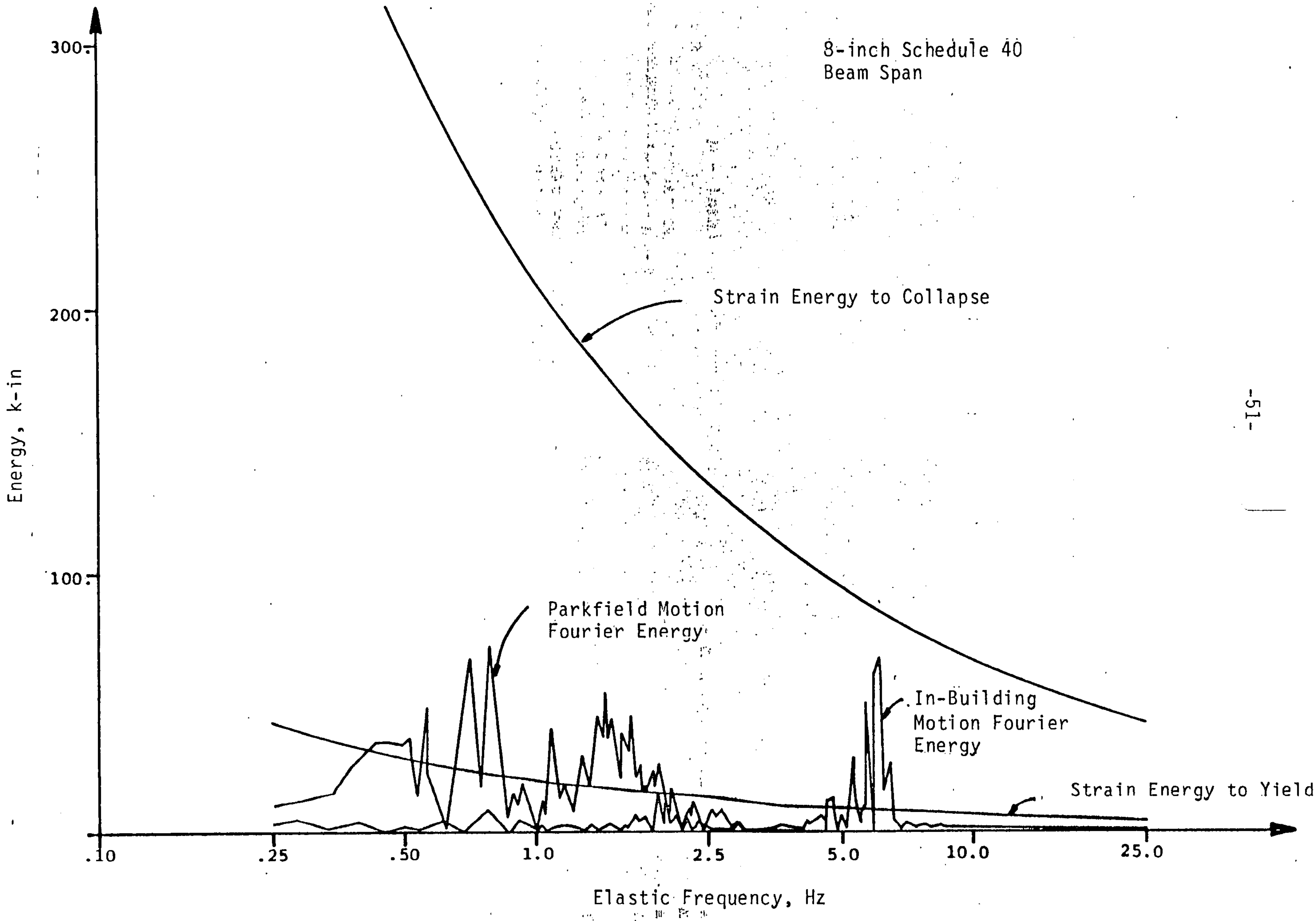


FIGURE 5-4 Comparison of Strain Energy and Earthquake Energy

c. Functionality Criteria for Equipment

The criteria for equipment (including valves, pumps, nozzles, etc.) will be in accordance with the BOPMEP Criteria.

5.2.5 Inelastic Criteria for Structural Members

The evaluation of structures for the effects of piping loads will be performed in accordance with the "Balance of Plant Structures Seismic Reevaluation Criteria," dated February 17, 1981. The evaluation will be limited to structural steel elements since the design capacity of concrete structural elements is not governed by the effects of piping loads. This is due to the fact that these structures are generally governed by shielding requirements and, therefore, tend to be quite massive with considerable reserve capacity.

The evaluation will be as follows:

1. All affected structural members will be evaluated for piping loads within the return-to-service scope.
2. All main structural framing members will satisfy plastic evaluation and design concepts of the AISC.
3. All connections associated with the members will be evaluated to assure adequate capacity.
4. Inelastic response of secondary steel members will be permitted. The limits on permissible ductility will vary from 3.0 for secondary members with contributing axial loads to 5.0 for members in pure tension.

The ductilities cited above represent a small amount of yielding in structural members. Typically, for a 10 foot span of an 18WF45 member under major axis bending a deflection of less than 1.0 inch would be expected. For the case of biaxial bending a deflection of about 1.5 inches would be expected. This is approximately equal to the magnitude of the deflections due to seismic anchor movements. Since the piping systems are designed to withstand anchor movement displacements and since this yielding will reduce the anchor movement stresses and loads, ductilities of this magnitude do not affect the integrity of the piping systems.

Approximately 2/3 of the complete scope of piping loads on structures have been subjected to preliminary screening calculations for loads based upon all pipe supports being installed to the BOPMEP Criteria. The results of these screening calculations show that only about 10% of the structural members required modifications to satisfy the Balance of Plant Structures Seismic Reevaluation Criteria. It is concluded that these conservatisms will not be significantly different when the structures are evaluated for the loads developed from the criteria in Section 5.2. It should also be noted that the ductility ranges to be used for secondary steel members are conservative when applied to an individual structural element and that it is not expected that exceedance of this limit would be unacceptable. Therefore, the implementation of modifications required to meet the ductility limits may not be completed prior to return-to-service. It is, however, anticipated that the analysis of these structural elements for the loads associated with the pipe supports will be completed prior to return-to-service.

5.2.6 Qualification of Small Piping

This section describes the methodology and criteria to be applied for the qualification of small bore piping as part of the program to return San Onofre Unit 1 to service. A more detailed description is provided in Appendix E. The intent of the approach is to take cognizance of the observed behavior of small piping systems in actual earthquakes and dynamic testing in developing a specific set of acceptance criteria applicable to SONGS 1. Conformance to this acceptance criteria will then be documented by field walkdowns.

The original design of piping systems at San Onofre Unit 1 was based on the 1955 version of the ANSI (formerly USAS) B31.1 Code for Power Piping. The fundamental basis of the 1955 version of the B31.1 Code is to develop a piping system that has a balance of flexibility and control. It is this concept of controlled flexibility that is in use today in the design of power plant piping. An inherent property of piping systems designed with controlled flexibility is the ability to absorb large amounts of energy such as is created by seismic ground motion.

Historically, piping systems designed similar to San Onofre Unit 1 have performed well when subjected to severe shaking from earthquakes of significant magnitudes. Several surveys have been made which document the satisfactory performance of welded carbon steel pipe. Two of the more authorative works on this subject were published by Cloud and by Murray, Nelson, et. al., (see Appendix E). Both of these studies concluded that for the particular earthquakes studied the performance of piping systems considerably exceeded the design basis.

In addition to the observed performance of piping systems in actual earthquakes, the performance of piping systems during dynamic tests has provided strong evidence of the substantial margins in the current design practice. It should be noted that these tests have not been limited to piping systems supported to the stringent requirements of current regulatory practice. In the case of the recent tests by ANCO engineers for KWU, very flexible piping systems similar to San Onofre Unit 1 piping systems were subjected to seismic inputs that exceed the spectra for San Onofre Unit 1. These tests were generic and formed the basis for the acceptance by the German regulatory authority of nuclear safety related small bore piping. In addition to the KWU tests, a number of other tests have been performed on small bore piping. These have supported the conclusion of excess capacity substantially beyond the design limits and even substantially beyond yield. The results of these tests are described in Section 3.0 of Appendix E.

The proposed program for qualification of SONGS 1 piping includes, in addition to the demonstration of design similarity to the KWU testing as described in Appendix E, the specific review of all SONGS 1 small bore piping required for safe shutdown. This review will apply criteria based upon good industry practice to ensure adequate lateral restraint, sufficient flexibility to provide for thermal growth, support for valves with eccentric masses, and adequate spacing of vertical supports to minimize dead weight and operating stresses. An approach to small bore piping that is similar to this is currently under consideration by a PVRC subcommittee.

To assure that the support configuration of the existing small bore piping and tubing at SONGS 1 (i.e., partially upgraded and partially original supports) will perform in accordance with tests and actual earthquake experience a field walkdown will be conducted to document the following:

1. Dead weight spans meet industry practice.

2. Valves with eccentric masses have supports adjacent to them.

3. Horizontal supports are placed at intervals approximately equal to 3 times the dead weight spans.

4. U-bolt nuts and pipe clamp nuts are properly tightened and have lock nuts where appropriate.

Modifications will be completed before return to service to assure that the "as-is" condition of SONGS 1 small bore piping and tubing comply with the above criteria.

5.3 Implementation Procedure

The criteria described in Section 5.2 will be applied to the SONGS 1 safe-shutdown piping systems. This is to assess the functional integrity of these systems for design basis earthquake loads. The assessment will be based on the existing pipe support configurations. Piping which satisfies the criteria is judged to be functionally acceptable. For piping which does not satisfy the criteria, installation of additional supports will be required.

The procedure for the application of the functionality criteria for piping and pipe supports (see Subsection 5.2.4) is described in the following subsections.

5.3.1 Piping Evaluation Procedure

The general procedure used to evaluate the piping consists of five steps. They are:

1. Review analyses and support configurations
2. Identify current stress state in piping
3. Evaluate stresses in as-installed piping
4. Evaluate component loads
5. Provide recommendations to meet functionality

a. Step 1 - The first step is to collect and review the as-built evaluations, (in support of the April 1982 submittal) the as-designed evaluations (in support of the Systematic Evaluation Program), and the as-installed support configuration.

b. Step 2 - The second step is to identify the current status of the stress state in the pipe, based on the analysis results and the support configurations for the three cases: as-built, as installed, and as-designed piping. If the as-installed support configuration match the as-built or as-designed support configurations, then a stress evaluation for the as-installed case is available and a functionality check is performed directly with these analysis results. If the as-installed support configuration is different from the as-built or the as-designed configuration, then Step 3 is implemented.

c. Step 3 - In this step, the piping is analyzed with the as-installed support configuration using either hand calculations or computer depending on the complexity of the problem and the level of conservatism in the hand calculations. The computer analyses are performed using the QUICKPIPE program.

The piping is checked for functional assurance based on the stress evaluation. If the pipe does not meet the criteria, supports are added to the piping model and a new evaluation is performed. This process is continued until the pipe is shown to meet the functionality criteria.

The hand calculation procedure consists of two general phases: model development and model analysis. The model development requires some knowledge of the piping stresses. With this information, the complexity required for the model can be quickly determined. For piping which is severely stressed, a more accurate model is required to eliminate as much unnecessary conservatism as possible.

In the model development phase, the first step is to decide how to treat branch line connections. For branch lines whose nominal diameter is less than or equal to 1/3 of the run line nominal diameter, the branch line is decoupled from the run line. That is, the run and branch lines are considered separately. For evaluations of the run line, the effect of the branch line is ignored. For evaluations of the branch line, the connection to the run line is treated as an anchor.

To assure that any failure downstream of the safe shutdown piping will not impair the functionality of the safe shutdown piping, the functionality evaluation is carried out beyond the safe shutdown boundary to (1) an anchor, or (2) a second support in each of the three orthogonal directions, or (3) a point where failure beyond that point would not jeopardize the functionality of the subject piping.

Once the boundary conditions are defined, the next step is to develop a multi-span beam model of the pipe. The spans of the model are based on a projection of the piping in the direction of the load. As an example, Figure 5-5 shows a model of a branch line between nodes 135 and 235. The mass of equipment and lines parallel to the direction of projection (e.g., line between nodes 210 and 215) is lumped onto the beam model.

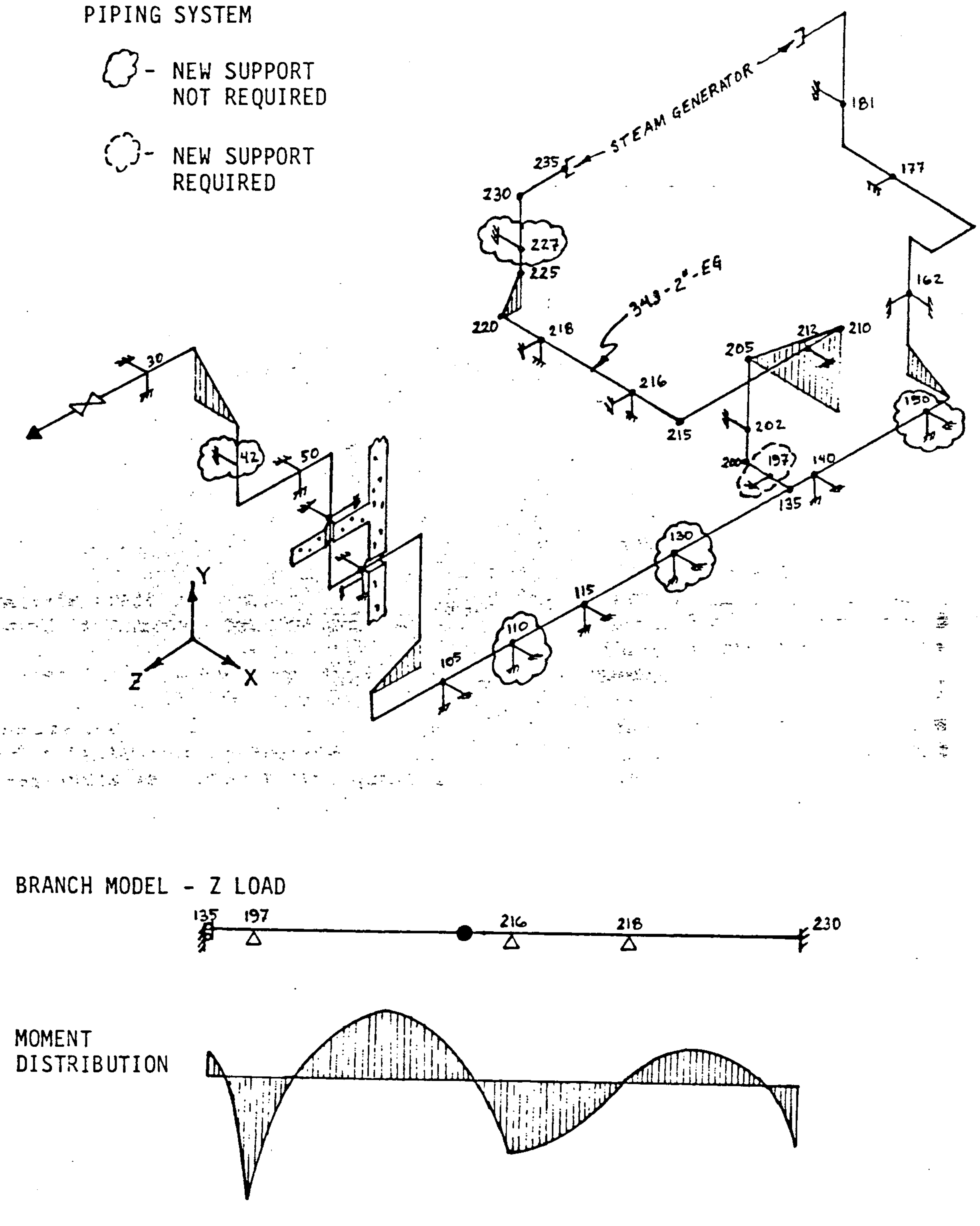


FIGURE 5-5 Problem MS-06 with Mathematical Model of Branch

The next step is to estimate the piping stresses. Based on the as-built and/or the as-designed analyses results, the level of refinement required in the analyses is determined. If the stresses are relatively low, then a more simplified approach, e.g., using single span beam model is used. In all cases, only static equivalent analyses are performed. To perform a simplified conservative evaluation, the DBE floor response spectra peak acceleration is used in conjunction with a 1.5 multiplier to account for higher mode participation and multifrequency input. In cases where a more refined approach is warranted to limit the stress level in the system, the spectra acceleration corresponding to the pipe fundamental frequency is used instead of the peak value. However, if the pipe segment fundamental frquency is less than the frequency corresponding to the peak acceleration frequency, the peak acceleration is used. Unless justified otherwise, the 1.5 multiplier is used to account for multiple frequency and higher mode participation.

Using the model developed and the determined acceleration level, a static equivalent analysis of the piping system is performed. It should be noted that several different models are required to conservatively compute the stresses. This is due to the different participation of the various pipe segments for the various load directions. The seismic stresses are combined with the pressure and gravity stresses for a functionality check using the criteria described in Section 5.2. The gravity stresses are obtained from the "as-built" analyses, if available, otherwise they are computed. The pressure stresses are computed using the design pressure, if known, otherwise the maximum operating pressure is used.

The computer analyses are performed using QUICKPIPE, which is based on standard production piping analysis techniques. This approach is used for piping where there are a large number of un-installed supports. This way, a refined analysis using a support optimizer is used to specify a minimum set of supports required for the piping to meet functionality.

<u>Step 4</u> - Once the piping is shown to meet the functionality limits, the forth step is to consider the component loads. These loads include the accelerations for pipe-mounted active valves and the nozzle loads. If the component loads meet allowables, then the pipe support configuration used to qualify the piping is deemed acceptable. If the component loads exceed allowables, then this is flagged as an open item.

Step 5 - The fifth and final step is to define action which should be taken for piping which exceeds the allowables. This action typically will include specific supports and, if applicable, a refined analysis to demonstrate that additional supports are not required.

5.3.2 Pipe Support Evaluation Procedure

In addition to evaluating the piping and its components, the pipe supports are also evaluated to demonstrate that functionality of the piping system will be maintained. Pipe supports are evaluated only to ensure global stability of the piping system. It is not necessary to assure functionality of each and every support during the earthquake to demonstrate piping functionality. Therefore, adequacy of the existing support configurations is evaluated through the following process:

a. The status of each support is reviewed to determine its potential for failure during the DBE event. Supports are classified in three categories based on their allowables, design, and predicted design load.

   A. Adequate -- sufficient capacity to carry load throughout the DBE. Generally, these supports either satisfy code criteria or can maintain their load carrying capacity despite partial failure or yielding of the support.

   B. Partially Inactive -- insufficient capacity to carry load throughout the DBE, but only inactive for brief periods of time. As an example, a steel strut may not absorb any added load after yielding but will continue to act as a support upon load reversal.

   C. Fully Inactive -- supports likely to catastrophically fail during the DBE. An example would be anchor bolt pull-out of all bolts.

b. Supports of type "A" are assumed to function throughout the DBE while type "B" and "C" supports are assumed to fail.

c. The span lengths between remaining type "A" supports are determined. A multiple-hinge collapse mechanism is postulated and the necessary strain energy in the pipe to develop that mechanism is calculated.

d. The earthquake energy available in the design spectra is then calculated as discussed in Section 5.2.4.b.

e. Where the available earthquake energy is less than the energy necessary to create the collapse mechanism, system functionality is assumed. If the energy to collapse is less than the earthquake energy, support modifications are identified to reduce the span length.

f. Where type "C" supports exist, load redistribution to adjacent supports is computed. Based on the redistributed load, the adjacent supports are checked to determine if they are reclassified as type "C" under the new load. If so, then failure is assumed and the process repeated.

5.4 Piping Review Examples

Two example piping problems are described here to illustrate the analyses used to evaluate the functionality of the SONGS 1 safe shutdown piping systems. The first problem is an example of a hand calculation analysis and the second problem is an example of a computer piping analysis. Both of these examples are based on actual safe shutdown piping analyzed for SONGS 1. All the other problems evaluated for functionality will be analyzed in the same manner.

5.4.1 Hand Calculation Analysis

The main steam line designated as MS-06 and depicted in Figure 5-5 is described here as an example of a hand calculation analysis. The portion of the line considered in this problem extends from the containment vessel (penetration C-1C) to the shield wall and then onto steam generator B. The piping was analyzed in 3 separate segments:

1) From the containment penetration to the shield wall.

2) From the shield wall to the steam generator nozzle, node 190.

3) From the branch node 135 to the steam generator nozzles, node 235.

Since no as-built analyses were available for the piping, and the as-installed and as-designed support configuratons were quite different, hand calculation analyses were performed on the as-installed configuration. The hand calculation method used was to determine the static equivalent stresses in the piping using the peak response spectral acceleration. Two sets of response spectra were used. For the piping inside the shield wall, the envelope spectrum, Figure 5-6 of the steam generator and building spectra was used, as this part of the piping was controlled by the relatively high steam generator response. For the piping outside and attached to the shield wall, the building spectra were used.

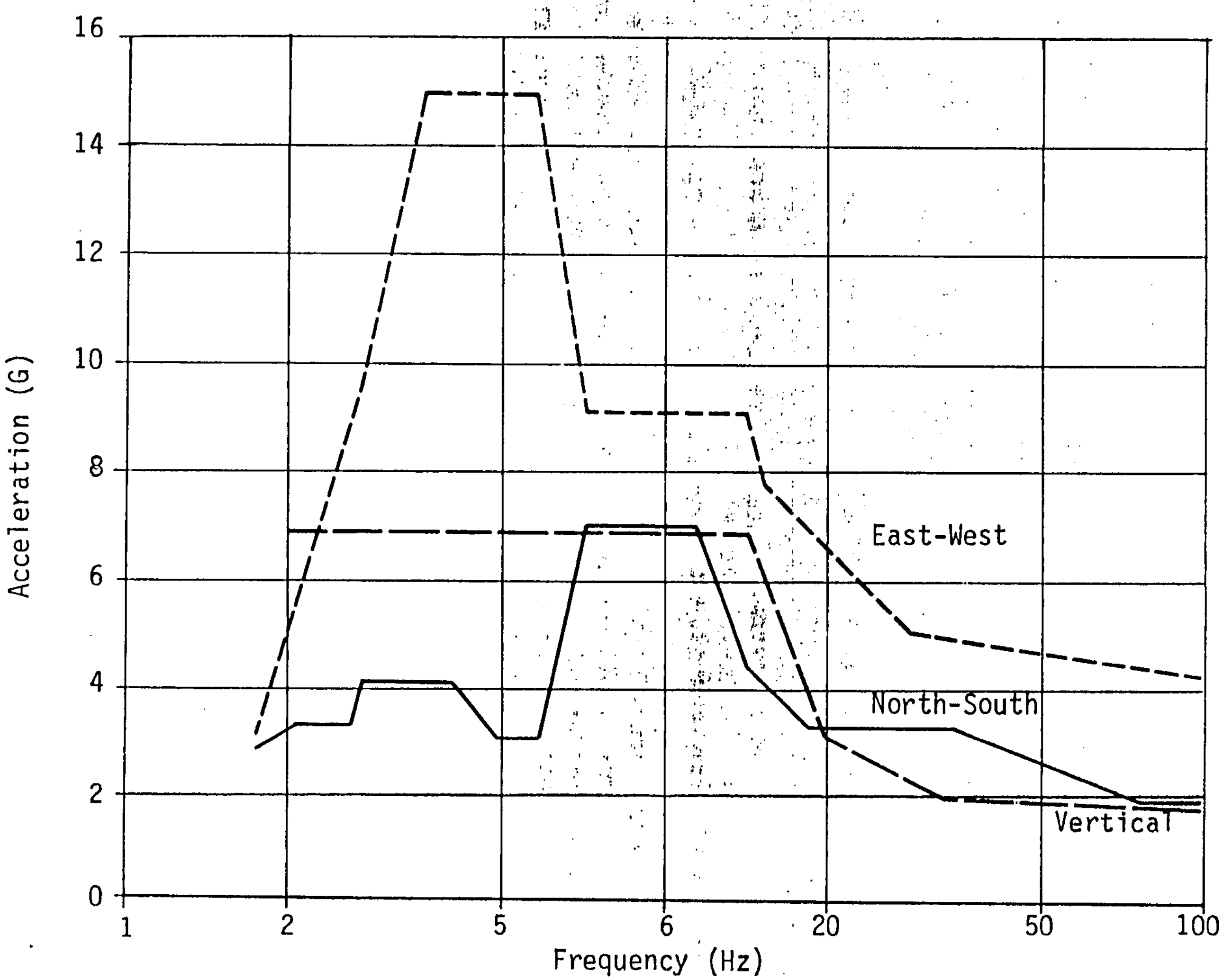


FIGURE 5-6 Envelope Spectrum Used for Piping from Steam Generator to Shield Wall

Continuous span beam models were generated using span projection methods. Figure 5-5 provides an example model. Lumped weights were added to account for equipment and for piping excited to displace in a rigid body mode. The models were necessarily different for each direction load. Each direction of earthquake input was considered separately and the results were combined using the square root of the sum of the squares (SRSS) method. The calculation of moments in the continuous beam models was done using the moment distribution method of joint relaxation to solve the statically indeterminate problem. The loads to compute the moments were directed to produce the highest moments possible of the piping. All the static load equations were obtained from Reference 25 and the frequency equations were obtained from Reference 26.

The response spectra corresponding to 4% damping were used for these particular evaluations. Based on Figure 5-2, a damping of at least 4% can be use for piping systems with frequencies up to 13 Hz. From the as-designed analyses, five frequencies were identified below 13 Hz. Since the as-installed system is more flexible than the as-designed system and the peak in the response spectra (which controls the peak response) is below 13 Hz, 4% damping is justified.

The piping stresses were computed using the section modulus and stress intensification factors given in ANSI B31.1, 1980 edition through Winter 1980 Addenda. A stress intensification factor (SIF) of 2.1 was computed and used for the tee and the fillet weld. This factor of 2.1 bounded the SIF for the long radius elbows. The total primary stress on the piping was computed using an SRSS of the moments due to SSE loads, moments due the the gravity load and stress due to pressure (1210 psi). These stresses were combined as per the criteria equation given in Section 5.2.4 and compared with the piping allowable based on 2.0 times the piping yield stress at design temperature (545°F). The most highly stressed section of the pipe occurred at node 235, which had a stress of 96% of the allowable. There were no active valves in this problem and the highest nozzle load was at steam generator node 190. The nozzle loads were combined using SRSS for the applied moments and SRSS for the applied forces. The nozzle loads were within allowables thus the piping functionality check controlled which supports are to be added. Adding one of the four un-installed supports was found necessary for the piping to meet the functionality limits.

5.4.2 Computer Analysis

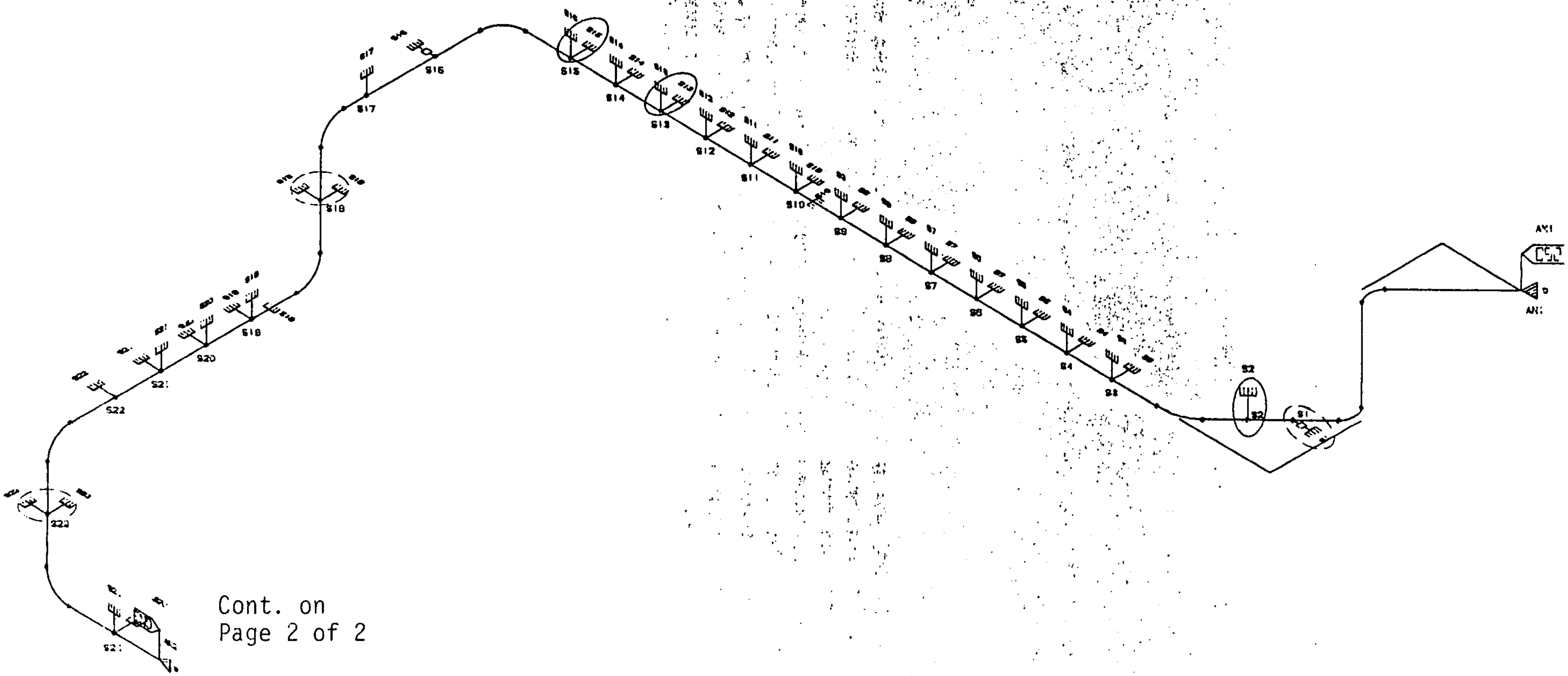
The main steam piping depicted in Figure 5-7 and designated as problem MS-358 was analyzed in two parts. Part 1 included the piping from the containment penetration (B-12) to an embedded pipe location. Part 2 included piping from the embedment to the steam generator nozzles. The entire piping is a 3/4 inch schedule 40S pipe size of material A-312-TP304. The design conditions for the system are temperature 545^{o}F and pressure 1210 psig.

A computer analysis using the QUICKPIPE program was performed for both parts of the piping separately. Since the line was attached to the steam generator, two sets of response spectra were used just as for the hand calculation problem. The pipe ends at the steam generator nozzle (anchors 2 and 3 on page 2 of Figure 5-7) had nozzle spectra applied while other portions of the line had floor spectra applied. The analysis was performed using standard production piping response spectra analysis for the DBE load. The DBE induced moments were computed for this problem using 4% damping and were combined using SRSS. Then a gravity analysis was performed using QUICKPIPE. This was then combined with the SSE and pressure stress and compared to the stress allowable. There were no active valves on this line and the nozzle loads were shown to be within allowables. Hence, the pipe stress controlled the functionality check of the piping. The results showed that 3 of the 12 un-installed supports were required. Based on this support configuration a maximum stress of 72% of the allowable was obtained at node 51W, thus showing a considerable amount of margin still exists based on the functionality criteria.

● ng System

(⁐) - New Support Not Required

○ - New Support Required



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Page 2 of 2

FIGURE 5-7 MATHEMATICAL MODEL FOR PROBLEM MS-358

(Page 1 of 2)

• Pip[illegible]ystem
( ) - New Support Not Required
◯ - New Support Required

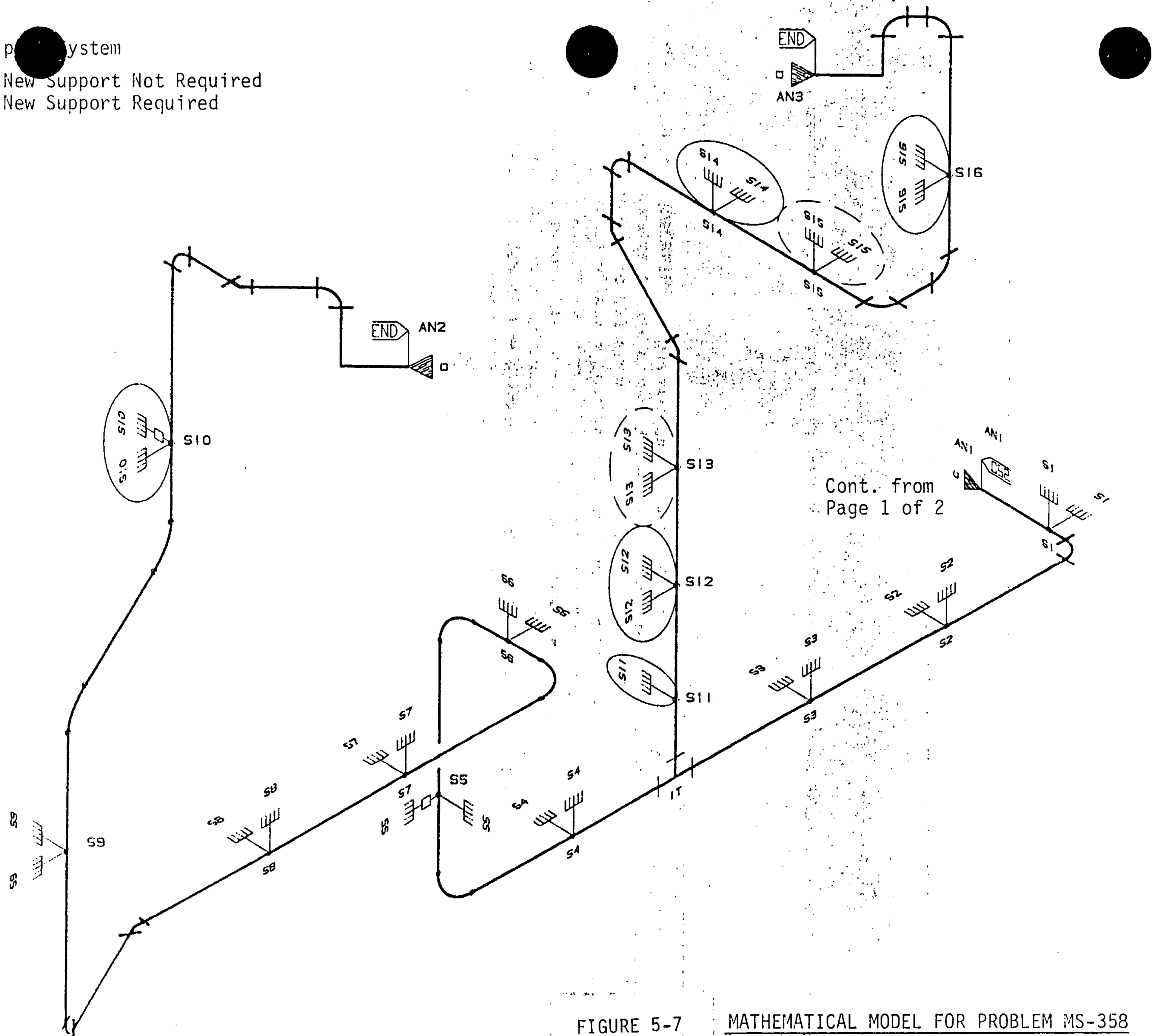

FIGURE 5-7 MATHEMATICAL MODEL FOR PROBLEM MS-358
(Page 2 of 2)

6.0 CONCLUSION

This report has provided the details of the various aspects of the return to service plan. As indicated earlier, the basic premise of the plan is that all structures and systems whose failure could cause an accident and/or whose function is required to obtain and maintain a hot standby condition will be available following a 0.67g earthquake. The plan consists of two main aspects: seismic hazard and safe shutdown.

The discussion of the seismic hazard aspect of the plan in Section 4.0 of this report identified significant conservatisms in the design basis ground motion for the SONGS site and also significant conservatisms in seismic design which are applicable to the SONGS 1 structures and components. In addition, it was shown that virtually every dominant contributor to seismic core melt frequency has been eliminated at SONGS 1 and that a conservative estimate of the seismic risk at SONGS 1 is that it is low and comparable to that at other plants. Based on these considerations, the seismic hazard should no longer be a significant concern for SONGS 1.

Although the seismic hazard is not a concern for the plant as a whole as discussed in Section 4.0, a specific evaluation discussed in Section 5.0 will be performed for those structures, systems and components required to obtain and maintain a hot standby condition to ensure their availability following a 0.67g earthquake. This evaluation involves an item by item review of the safe shutdown systems in accordance with identified acceptability criteria. The evaluation will be completed prior to return to service.

Based on the assessment of the seismic hazard and the evaluation of the safe shutdown systems, including the implementation of any required modifications, SONGS 1 can return to power without undue risk to the health and safety of the public even considering the possibility of a major earthquake at the plant site.

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APPENDIX A

SEISMIC HAZARD STUDY

SAN ONOFRE UNIT - 1

NEW MEXICO ENGINEERING CONSULTANTS, INC.
3936 GARCIA, N. E.
ALBUQUERQUE, NEW MEXICO 87111
(505) 296-7756

September 13, 1983

Mr. J. H. Hutton
CE Power Systems
Combustion Engineering
1000 Prospect Hill Road
PO Box 500
Windsor, Connecticut
06095-0500

Dear Mr. Hutton:

I have upon your authorization studied the seismic hazard situation at the San Onofre site. In this work, I have drawn heavily upon the technical expertise of the staff of Southern California Edison Company (SCE), and I have met with the staff of Lawrence Livermore Laboratory (LLL), to determine what would best meet their needs. As a result of that meeting, the scope of the work was increased beyond that contemplated in my proposal letter to you, dated 25 May, 1983. By performing a quick mid-course correction, SCE and I were able nevertheless to complete the work. The basic computational work was done by TERA Corporation (TERA) and Woodward-Clyde Consultants (WCC).

I have considered only Instrumental Peak Ground Acceleration, IPGA, because the LLL staff indicated that those would be appropriate and adequate for their studies.

For your reference, those values are: best estimate (mean), about one-third gravity (g); 84th percentile (mean plus one standard deviation,hereafter, 1-sigma), about one-half g; 2-sigma, about two-thirds g; and 3-sigma, about one g (which includes all of the instrumental strong-motion records in the world). Regardless of the statistics, the latter value is probably an upper limit for a site located about 8 km from a major event, regardless of the precise magnitude of the earthquake: based on physical principles, both Brune and McGarr have developed physical arguments for upper-limit IPGAs of less than about two g directly at the fault; and, using just about anyone's relatioship between IPGA, Magnitude, and distance, the maximum IPGA will have attenuated to about half its at-fault value, for a Magnitude 7 earthquake, by the time the waves have propagated out about 8 km. Thus it would be difficult to postulate any IPGA at this site greater than about one g or so.

The resulting hazard curves, expressed as Return Period as a function of the IPGA are given in Fig. 1. The TERA and WCC curves differ somewhat because the two groups made different assumptions and weightings in their treatment of the IPGA data. The closeness of the two results gives confidence that the results would not be significantly different from those shown in Fig. 1 for any other reasonable set of assumptions. It would therefore be my recommendation that curves lying smoothly at about the average of the two calculations be used; and I have sketched such central curves in Fig. 2. The three curves on Figs. 1 and 2 are mutually exclusive: If you wish to assume that the truncation within the bounds of plus-minus one sigma is correct for the return period you are considering, then the other two truncations are by definition impossible. At the LLL meeting, I was asked if I could give some further information on the distribution of IPGA at fixed return periods. That is, what would the distribution curve look like if one took a horizontal slice across the graph? To do that calculation rigorously would require redoing the TERA and WCC calculations , but I do not feel that would be either necessary or justified, as I shall now explain. In the original studies by TERA and WCC, the conditions which were varied for the probalistic aspects of the work included the size of the earthquake, the location of the fault, among other factors. The studies clearly showed that the nearby Offshore Zone of Deformation (OZD), postulated to lie 8 km from the site at the closest point, dominates the probabalistic results. Thus I assume that the principal contribution to the uncertainties is the truncation level chosen for the attenuation distribution. Thus I do not recommend redoing the work to include all of the probalistic factors which might be of importance. Instead, I recommend utilizing the curves of Fig. 2 as the bases for introducing some judgement into the situation, to furnish engineering approximations to the desired hazard curves.

The hazard curves will now be developed. It is assumed that the probabilities are about 100 percent that the site will experience an IPGA in excess of zero g over a long period of time, say 10^6 years. Thus zero g (and probably something greater) would fit the general notion of an upper bound of hazard curves. On the other hand, the existing data and the physical arguments advanced by leading seismologists, and as discussed above, suggest that a truncation on the order of plus-minus 3-sigma, for a site at 8 km from the earthquake, might reasonably fit the notion of an upper bound: that is, a hazard curve which has 100 percent probability of not being exceeded. The work necessary to fill in between these 0 percent and 100 percent bounding hazard curves can be done in several ways, all of which must involve some subjectivity and weighting. I have, with considerable help from Dr. C Mortgat of TERA, chosen to do those weightings in a way which is testable and reproducible, and have defined from the resulting band of curves the central one which I will suggest to you for your work as I understand it. We have 50 points in the bounds of the plus-minus 1-sigma, 66 points within the bounds of plus-minus 2-sigma and 67 points within the bounds of the plus-minus 3-sigma, we can compute the sample liklehoods by assuming a binomial process on the attenuation relationship, and that the uncertainty in recording is similar to the uncertainty in the attenuation relationship. Then we make some subjective judgements: on the one hand, we assume that, over a very long time, there is only a 10 percent chance that the plus-minus 1-sigma deviation truncation will be correct, but we assume 30 and 60 percent, respectively, that the plus-minus 2- or 3-sigma levels will be the correct truncation; then, on the other hand, we assume values of 10, 60, and 30 for the same truncations. The results of the calculations flowing from those assumptions are given in Fig. 3, in the form of probabilities of exceedance as a function of the IPGA which is being postulated to be exceeded over a very long time. Even considering the rather different assumptions regarding the truncation levels, the two curves are not very different: they do not differ greatly in value, and their shapes are consistent. It is for that reason that I suggest the central curve sketched on Fig. 3 for use in positioning hazard curves for very long times for this site, for the work you will be doing as I understand it. the resulting family of hazard curves are given in Fig. 4. One conclusion which which could be drawn from Fig. 4 is that, for the assumptions made, the design PGA of 0.67g (which is itself about a 2-sigma value) has about a 90 percent chance of being exceeded over an indefinitely long time.

I trust that these results will be useful to the LLL staff in their studies. If I may be of further assistance in this or other similar work, please do not hesitate to call.

Very truly yours,
NEW MEXICO ENGINEERING CONSULTANTS

By--------------------
Robert L. McNeill

cc: SCE, Mr. H. G. Hawkins
TERA, Mr. L. H. Wight
WCC, Dr. K. Sadigh

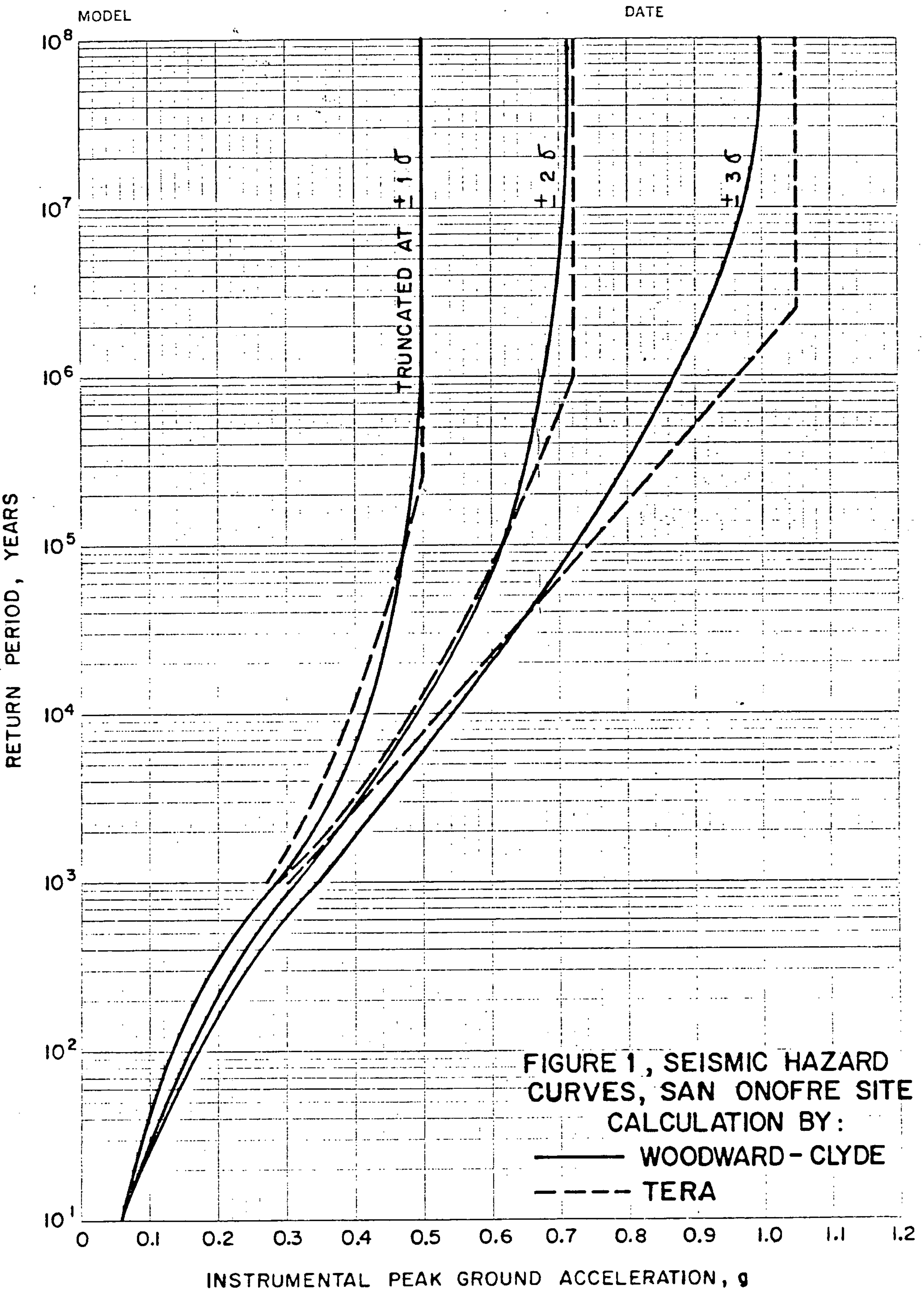


FIGURE 1, SEISMIC HAZARD CURVES, SAN ONOFRE SITE

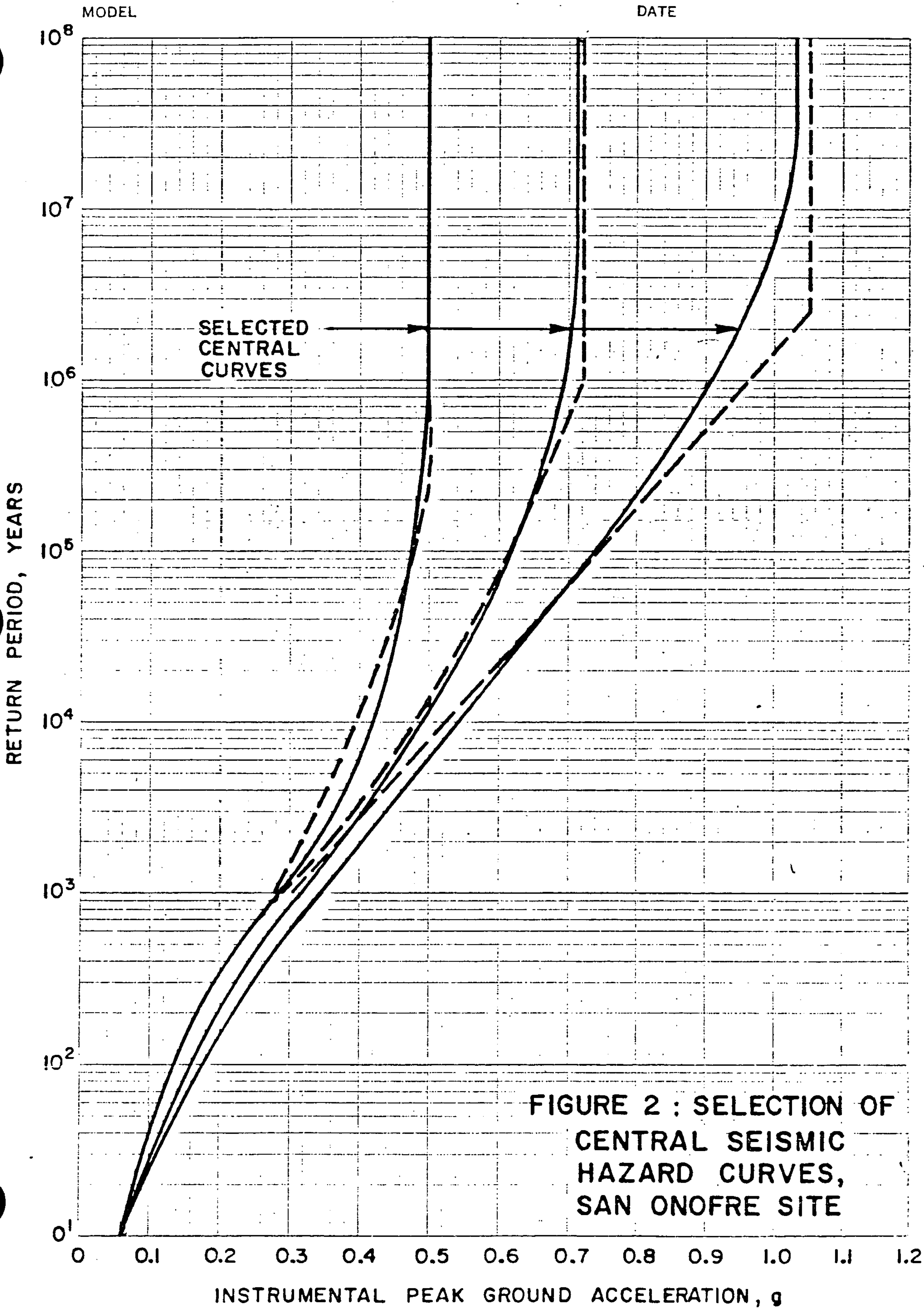


FIGURE 2 : SELECTION OF CENTRAL SEISMIC HAZARD CURVES, SAN ONOFRE SITE

46 6463

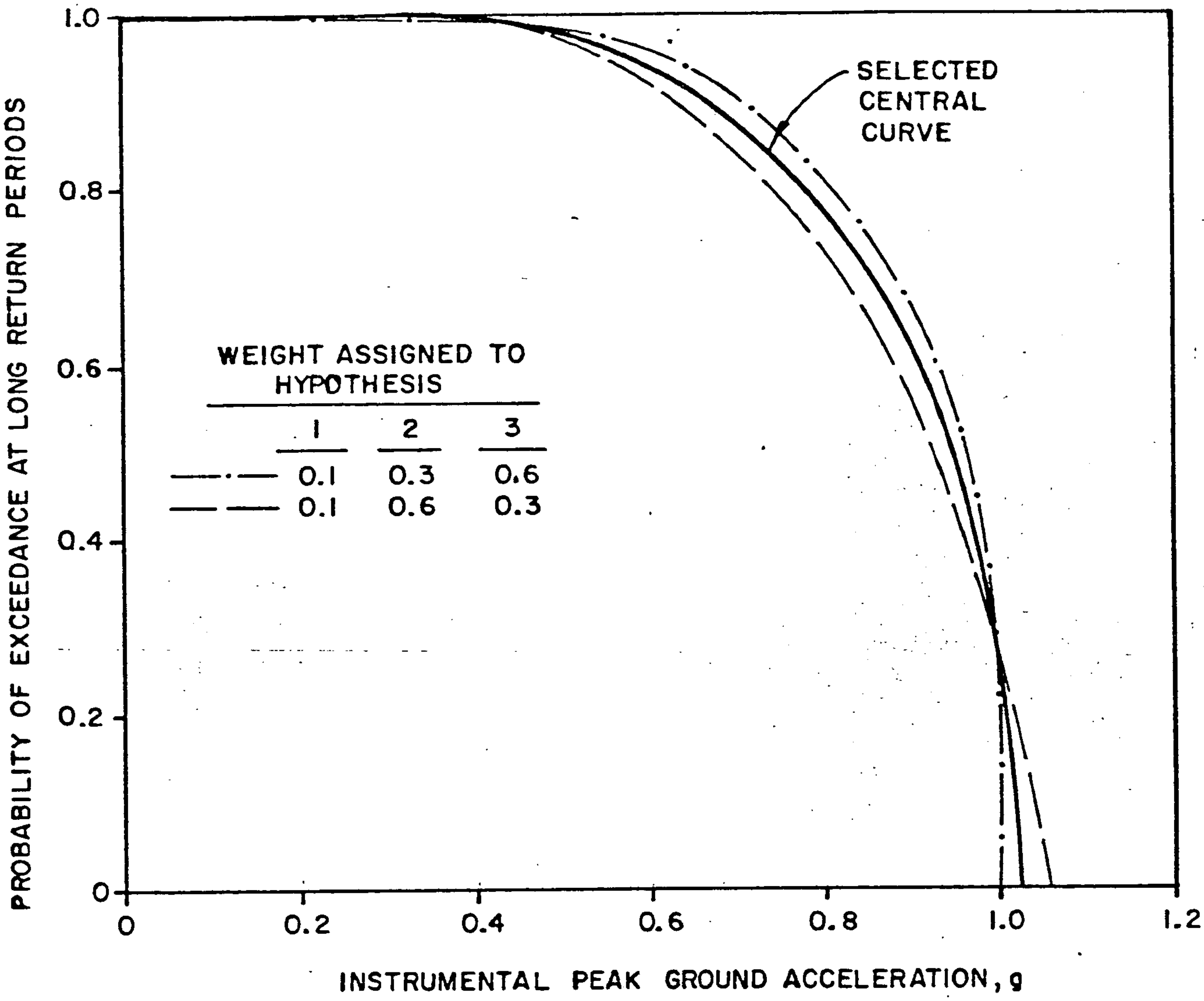


FIGURE 3: EFFECTS OF ASSIGNING WEIGHTS TO HYPOTHESIS

MODEL DATE

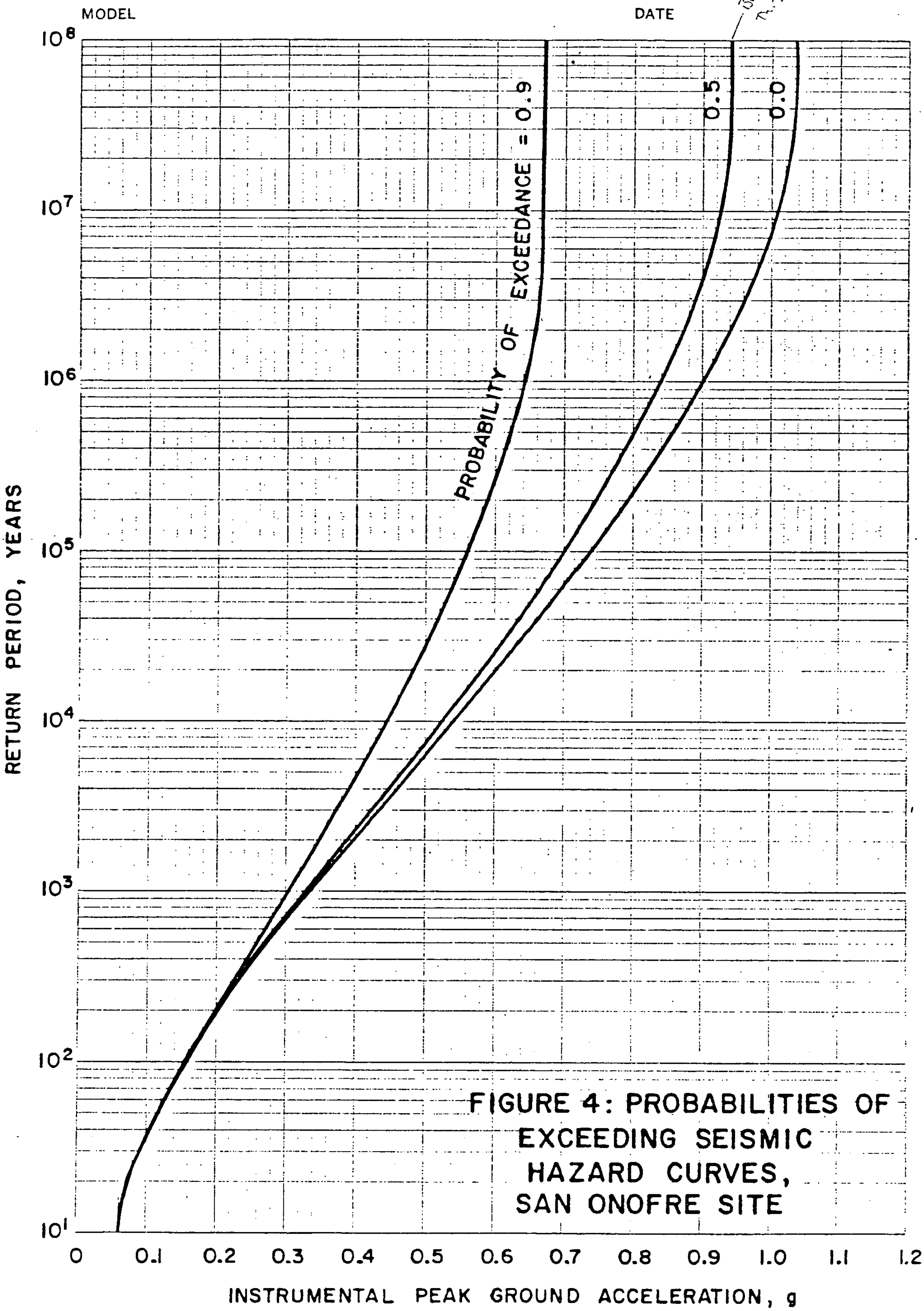


FIGURE 4: PROBABILITIES OF EXCEEDING SEISMIC HAZARD CURVES, SAN ONOFRE SITE

46 6463

SEMI-LOGARITHMIC 7 CYCLES X 60 DIVISIONS
KEUFFEL & ESSER CO.

APPENDIX B

LESSONS LEARNED FROM SEISMIC
PROBABILISTIC RISK ASSESSMENT STUDIES

SAN ONOFRE NUCLEAR GENERATING STATION

UNIT 1

DECEMBER 23, 1983

TABLE OF CONTENTS

1.0 INTRODUCTION

A number of probabilistic risk assessments (PRAs) for nuclear power plants have become available in recent years. Over a dozen of these PRAs have estimated the contribution of seismic events to the predicted annual frequency of core melt. The purpose of this report is to compare the SONGS 1 design against the dominant contributors to seismically induced core melt frequency as identified by the seismic risk portions of PRAs prepared for other nuclear plants. The evaluation consisted of three steps:

1. Obtaining a clearer understanding of the relative importance of various types of seismically induced failures to core melt frequency;

2. Providing a checklist of dominant contributors to seismically induced core melt frequency;

3. Evaluating the existing design of SONGS 1, including modifications already performed, relative to the seismic dominant contributors of plants for which seismic PRAs have been performed.

2.0 METHODOLOGY

Detailed seismic PRAs have been completed for about 15 nuclear power plants. As of November 1983, only a few of these PRAs have been published and submitted for NRC review. The first step in the evaluation presented herein was a review of three published PRAs (Zion, Indian Point Units 2 & 3, and Limerick) to attempt to characterize the dominant contributors to seismically induced core melt frequency. After developing an initial list of major contributors, consultations were held with two firms that have conducted many of the published and yet-to-be published seismic PRAs (Pickard, Lowe, & Garrick Inc., and Structural Mechanics Associates, Inc). These consultations served to verify and supplement the list of dominant contributors to core melt as well as identify seismically induced failures which are major contributors to offsite consequences. The SONGS 1 design was then evaluated to assess the current plant status for each dominant contributor category.

3.0 DOMINANT CONTRIBUTORS TO SEISMICALLY INDUCED CORE MELT FREQUENCY

Review of three published seismic PRAs and discussions with experts extensively involved in the preparation of many published and yet-to-be published seismic PRAs resulted in the identification of dominant contributors to seismic core melt frequency. These dominant contributors, whose relative importance may vary from plant to plant, include:

A. Failures of reactor coolant system components, usually by loss of supports or failure of an enclosing structure. In one BWR plant, loss of the upper stabilizer brackets for the reactor pressure vessel support was inferred to cause a major breach of the reactor coolant system pressure boundary through the breakage of all main steam lines. In a PWR plant, the pressurizer and associated piping were shown to be vulnerable to the seismically induced collapse of the pressurizer enclosure roof.

B. Failures of offsite and onsite electric power sources. In the PRAs reviewed, non-seismically designed switchyard equipment (e.g., insulators) are identified as being vulnerable to failure at relatively low seismic excitations. Thus, offsite power is usually lost following an earthquake of even moderate magnitude. Seismically vulnerable elements of emergency onsite power sources identified in prior seismic PRAs include the diesel generator essential support systems; associated electrical and controls equipment (particularly improperly anchored control cabinets and battery racks); and, in a few instances, failure of electrical conduits and bus ducts.

C. Losses of essential cooling water supplies. These losses are predominately associated with failure of large storage tanks. Tankage failure is most commonly initiated by loss of anchorage that in turn leads to compressive buckling of the tank wall. In one seismic PRA, both the refueling water storage tank and condensate storage tank were shown to be susceptible to seismically induced failure.

D. Failure of equipment supports and anchorage. Several examples of this type of failure are cited above.

E. Localized or complete failure of structures; the latter usually occurring only at ground accelerations well in excess of the design basis earthquake. Failures of structures can cause failure of essential systems and equipment attached to the failed structure or otherwise damaged by falling debris. An example involving a pressurizer and associated piping is cited above. Failures of masonry walls in the vicinity of essential equipment has also been identified as a potentially important contributor to seismic core melt frequency in some PRAs.

The preceding list of dominant contributors to seismic core melt frequency has been compiled from seismic PRAs for predominately east coast plants. Although most of these plants were designed to less stringent seismic design criteria than SONGS 1, the ground accelerations considered in estimating their actual seismic capacities range from zero up to (and in some cases, slightly higher than) the 0.67g seismic reevaluation basis for SONGS 1. It should also be noted that the seismically induced failures identified as important contributors to seismic core melt frequency for these plants occurred at median ground accelerations usually at least 2 to 3 times higher than the design basis earthquake.

Another significant conclusion from these seismic PRAs (as well as from damage assessments following historical earthquakes) is that ductile steel piping systems with butt welded joints have a high seismic withstand capability from the standpoint of inertial effects. Seismic PRAs show that the median ground acceleration capacities for ductile steel piping systems are sufficiently high that the fragility curves predict a negligible probability of failure at ground accelerations comparable to the 0.67g seismic reevaluation basis for SONGS 1.

The only types of piping system failures that have been identified as potentially important contributors to seismic core melt frequency for ground accelerations in the range of 0.67g and lower include:

1. Failure of non-ductile (e.g., buried cast iron) piping

2. Failure of threaded joints

3. Failure of piping routed between structures with separate foundations, where seismic anchor movement (SAM) loads caused by differential building movements become excessive.

The above-listed failures were predicted to occur only at ground accelerations well in excess of the design basis earthquake for those plants in which these types of failures were identified.

4.0 SONGS 1 EVALUATION

A list of dominant contributors to seismic core melt frequency is presented in Section 3.0. This list was compared with the existing SONGS 1 design to determine whether or not these same types of failures might contribute significantly to the probability of seismically induced core melt in the event of a 0.67g seismic event at San Onofre.

The following subsections present the findings of the analysis for each of the dominant seismic core melt contributors identified in Section 3.0. The discussion for each begins with a brief description of the nature and significance of the failure mode. An evaluation is then presented of the SONGS 1 design to indicate whether or not that failure mode is likely to represent a significant contributor to seismic core melt frequency for seismic events up to 0.67g at San Onofre.

4.1 Reactor Coolant System

A. PRA Significance

Failures of reactor coolant system (RCS) components, control rod drive mechanisms, and/or reactor internals have been identified as major contributors to seismic core melt frequency. Failures of major components are usually associated with either the loss of supports or a failure of an enclosing structure.

B. SONGS 1 Design

During the period from 1972 to 1977, the reactor coolant loop piping, reactor vessel, steam generators, pressurizer, reactor internals, CRDMs, and supports were reevaluated and upgraded as necessary to ensure their ability to withstand a 0.67g seismic event. Included in this effort was a concurrent reevaluation of the containment sphere and reactor building to ensure that the supporting structures for the RCS can also withstand a 0.67g seismic event. The results of this effort (known as the Seismic Backfit Project) have been reviewed by the NRC.

As a result of the aforementioned analyses and modifications, the SONGS 1 reactor coolant system has been upgraded to effectively eliminate this item as a major contributor to seismic core melt frequency for ground accelerations up to 0.67g.

4.2 Electric Power Systems

A. PRA Significance

Electric power systems are classified as either offsite or onsite. Loss of offsite power has typically occurred at low ground accelerations and was usually initiated by damage to switchyard equipment. A loss of offsite power results in a loss of plant load and subsequent turbine trip/reactor trip. If offsite power is lost, the onsite emergency power systems must be available to bring the plant to a safe shutdown.

Three general categories of seismically induced failures in onsite power systems which have been identified as important contributors to seismic core melt frequency at ground accelerations up to 0.67g include:

1. Diesel generator support systems

2. Electrical & controls equipment

3. Raceways (cable tray & conduit)

The seismic fragilities of systems supporting the diesel generators (e.g., cooling, air start, fuel oil) are usually more limiting than the diesels themselves. For example, in one PRA, diesel availability was lost due to the loss of cooling water resulting from the failure of service water pumps. The pumps failed functionally, either directly, or indirectly through the structural collapse of the roof of the building in which they were enclosed.

Electrical and controls related items that have been identified as important contributors to seismic core melt frequency, at ground accelerations in the range of interest, are associated with inadequate anchorage of control cabinets and battery racks, and various types of structural failures leading to loss of control (e.g., collapse of the control room ceiling).

Failure of conduits and cable trays associated with power and control circuits have not appeared as dominant contributors to seismic core melt frequency for ground accelerations in the range of interest except for one plant. In that case, cable tray fragility was based on failure of cable tray supports for a rod supported system with no lateral bracing. Unistrut-supported or lateral-braced systems have higher seismic capacities.

B. SONGS 1 Design

Major additions and modifications to the SONGS 1 offsite and onsite power systems have been performed over the past ten years. These, coupled with extensive structural evaluations, have significantly enhanced SONGS 1 seismic capabilities.

After the 1971 San Fernando valley earthquake, an evaluation and upgrade program was initiated for switchyards in the SCE system. As a result of this effort, the San Onofre plant switchyard was designed to a 0.5g design basis. This is significantly greater than a typical power plant design and reduces the likelihood of a seismically induced loss of offsite power. Accordingly, the contribution to the SONGS 1 seismically induced core melt frequency from a loss of offsite power has been significantly reduced.

During the 1976-77 outage of the plant, an entirely new safety-related diesel generator onsite power system was added to enhance the ability of SONGS 1 to safely withstand a 0.67g seismic event. This effort (known as the Standby Power Addition project) included the addition of two new 6000 kw diesel generators complete with their own dedicated support systems. New cooling systems independent of existing plant systems were added along with new fuel oil, air start, HVAC, and switchgear. These new systems and hardware were enclosed in a new building separate from existing plant structures. This new onsite power system was designed to withstand a 0.67g seismic event. Accordingly, these items are effectively eliminated from being an important contributor to seismic core melt frequency for ground accelerations up to 0.67g.

In response to NRC letters dated January 1, 1980 and July 28, 1980, a seismic reevaluation and upgrade program was initiated in 1980. The scope of this effort was the support and anchorage of all safety-related electrical equipment (e.g., panels, racks, MCCs, switchgear, inverters, etc.). Also included was the anchorage of non-seismic Category I ancillary items which could damage the safety-related items identified if the ancillary items were to fail during a seismic event.

The reevaluation was concluded and results submitted to the NRC by letters dated March 25, 1981 and May 29, 1981. Modifications have been completed on all items. Therefore, failure of electrical and controls equipment by loss of anchorage is effectively eliminated from being an important contributor to seismic core melt frequency for ground accelerations up to 0.67g.

One structurally related loss-of-control failure identified was initiated by collapse of the control room ceiling. The SONGS 1 control room ceiling has been replaced with a new 0.67g seismically designed ceiling. Therefore, this item has been effectively eliminated as an important contributor to seismic core melt frequency for ground accelerations up to 0.67g.

SONGS 1 does not utilize the types of conduit and cable tray supports that were identified as being vulnerable to seismically induced failure at ground accelerations in the range of interest. Moreover, as part of the current seismic reevaluation and upgrade program, a total of approximately 1100 cable tray supports and 7300 conduit supports were evaluated, and the majority of identified modifications, over 1700, have been implemented. In addition, actual testing of raceways similar to those at SONGS 1 has demonstrated their capability to safely withstand a 0.67g seismic event. Therefore, failure of conduits and cable trays are effectively eliminated from being an important contributor to seismic core melt frequency for ground accelerations up to 0.67g.

4.3 Essential Water Supply

A. <u>PRA Significance</u>

Failures of essential water supplies have been identified as a dominant contributor to seismic risk. These failures are primarily associated with large tankage required for either primary or secondary systems makeup for heat removal; e.g., the refueling water storage tank (RWST) and condensate storage tank (CST). Tank failures are usually a result of anchorage or support skirt failures which lead to compressive buckling of the tank wall and subsequent loss of the tank contents.

B. <u>SONGS 1 Design</u>

A seismic reevaluation and upgrade program for the balance of plant and mechanical equipment and piping (BOPMEP) has evaluated the seismic adequacy of the SONGS 1 CST and RWST.

The evaluation identified a need to modify or replace the CST in order to establish seismic adequacy for a 0.67g seismic event. Rather than modify the CST, a new seismically qualified auxiliary feedwater storage tank (AFWST) has been designed and constructed to withstand a 0.67g seismic event.

To supplement the RWST, an additional source of borated water for primary side makeup will be designed and constructed to withstand a 0.67g seismic event. This source will consist of a crosstie from the spent fuel pool (SFP) to the charging pumps. Accordingly, these items are effectively eliminated from being an important contributor to seismic core melt frequency for ground accelerations up to 0.67g.

4.4 Structures

A. <u>PRA Significance</u>

Structural failures of buildings housing, or in close proximity to, essential equipment have been identified as a major contributor to seismic risk. Structures are generally assumed to fail functionally when inelastic deformations under seismic load are sufficient to interfere with the functioning or support of safety-related equipment. Actual collapse of a structure is assumed to result in a common cause failure of all safety-related equipment or systems housed within the failed portion of the structure. Examples of structural failure causing loss-of-function of essential systems and equipment have been cited in earlier sections. The possibility of damage caused by impact between two structures during a seismic event has been identified in seismic PRAs for some plants. Other structural failures identified as important contributors are the collapse of masonry walls.

B. SONGS 1 Design

During the Sphere Enclosure Project, Standby Power Addition Project, Seismic Backfit Project, as well as part of the current seismic reevaluation program, safety-related structures in the plant have been analyzed to assess their adequacy to withstand a 0.67g seismic event. The evaluation has included the following structures:

- Containment Sphere
- Sphere Enclosure Building
- Reactor Auxiliary Building
- Turbine Building
- Control Building
- Diesel Generator Building
- Fuel Building
- Intake Structure
- Ventilation Equipment Building

Where required modifications were identified, they have been implemented such that all structures required to ensure safe shutdown are capable of withstanding a 0.67g seismic event. Separation of structures was also evaluated. The evaluation demonstrated that impact between structures will not occur at ground accelerations up to 0.67g.

To assess the seismic capacity of existing masonry walls in the plant, extensive nonlinear analyses were performed. Then, a testing program was conducted to verify the results of the nonlinear analyses. The evaluation showed that the existing walls will all withstand a 0.67g seismic event without collapse. In addition, a few masonry walls whose deflections may have affected attached electrical tray and conduit raceway have been modified to limit their deflections during a 0.67g seismic event. Therefore, this item has been effectively eliminated as an important contributor to seismic core melt frequency for ground accelerations up to 0.67g.

Except for limited portions of the Control Administration building, Ventilation equipment building and isolated footings of the Turbine building, all plant structures are founded upon native San Mateo sand that is not susceptible to failure at 0.67g. For the three areas of the plant site where in-situ soils are a concern, evaluations have been performed and (where necessary) modifications have been initiated or completed to ensure that structures in these areas required for safe shutdown will withstand a 0.67g seismic event. The only affected essential systems and equipment required for safe shutdown are the auxiliary feedwater pumps foundation and 480 V switchgear room slab. These areas will be addressed prior to return to service to ensure that consequences of settlement of the slabs will not impair the integrity of the supported equipment.

As a result of the extensive analysis and modifications at SONGS 1, the essential structures have been demonstrated to be adequate or upgraded to effectively eliminate these items from being major contributors to seismic core melt frequency for ground accelerations up to 0.67g.

4.5 Piping

Seismically induced piping system failures have generally not been found to be a dominant contributor to seismic core melt frequency except in special circumstances noted in Section 3.0. These special circumstances are discussed below with respect to the design of SONGS 1.

A. Welded Ductile Steel Piping

With the exception of a small amount of cast iron pipe, all SONGS 1 safety-related piping systems are welded ductile steel in nature. Historical experience has shown that such systems have a very high seismic withstand capability. This experience is reflected in seismic PRA fragility data, all of which indicate that the median ground acceleration capacities of welded ductile steel piping systems are sufficiently high that - even considering uncertainties - the probability of failure at ground accelerations of 0.67g and lower is small and not a major contributor to seismic core melt frequency even for piping systems which were designed for an SSE much lower than 0.67g.

B. Cast Iron Piping

SONGS 1 contains a limited amount of buried cast iron pipe associated with the salt water cooling (SWC) system. Cast iron pipe is known to be susceptible to failure under seismic load (refer to Section 3.0). To ensure the safe shutdown capability of SONGS 1, an alternate means of heat removal will be provided for systems required for safe shutdown.

C. Process Piping with Threaded Joints

SONGS 1 utilizes no threaded joints in process piping for essential systems. Therefore, this item is eliminated as a potential contributor to seismic core melt frequency for SONGS 1.

D. Piping Spanning Buildings on Separate Foundations

SONGS 1 contains some essential piping spanning buildings on separate foundations. Some of the piping system failures identified in seismic PRAs were attributable to excessive differential building movement during an earthquake (Section 4.0). The evaluation of piping spans between structures that are required to attain safe shutdown will be specifically addressed prior to return to service. Thus this item will be eliminated as an important contributor to seismically induced core melt frequency for SONGS 1.

5.0 CONCLUSIONS

This report has evaluated the SONGS 1 design status in relation to dominant contributors to seismically induced core melt frequency identified from seismic PRAs for other nuclear plants. The upper bound of ground accelerations considered in the seismic PRAs which were used to identify dominant contributors to seismically induced core melt, are comparable in magnitude to the 0.67g seismic reevaluation criterion for San Onofre. There are no obvious features of the SONGS 1 design which would suggest that SONGS 1 is fundamentally different than these other plants insofar as the most important contributors to seismically induced core melt. Based on this evaluation, it is concluded that the seismic upgrade work completed to date on SONGS 1 has concentrated on the most important contributors to seismic core melt frequency. Virtually every dominant contributor category has been eliminated by work already completed.

On the basis of these comparisons it is concluded that SONGS 1 would have a low probability of seismically induced core melt for peak ground accelerations up to and including 0.67g.

APPENDIX C

EVALUATION OF SAN ONOFRE NUCLEAR GENERATING STATION

(SONGS) UNIT 1 SEISMIC RISK

December 23, 1983

EVALUATION OF SAN ONOFRE NUCLEAR GENERATING STATION (SONGS) UNIT 1 SEISMIC RISK

INTRODUCTION:

This report provides a summary of an assessment of the frequency of seismic event induced core damage for San Onofre Nuclear Generating Station Unit 1 - SONGS 1. The assessment has been performed by Nucon Inc. with technical assistance of Dr. H. Lambert, an associate of Nucon Inc. and Dr. Ron Polivka of URS Blume Associates. This evaluation rests heavily on the Seismic Safety Margin Review Program (SSMRP) and data generated by Lawrence Livermore Laboratory. Key features of SONGS 1 capability to preclude core damage from earthquake has been modeled in sufficient depth to include known important features. The analysis includes substantial conservatism. Where required, estimates of SONGS 1 specific response have been included, particularly estimates of location specific accelerations. Failure probabilities have been estimated from SSMRP directly for piping, inferred from SSMRP fragility for components, and estimated conservatively for SONGS 1 structures specifically.

The results of the study indicate that SONGS 1 has a seismic risk level very near Zion as stated in SSMRP and below the safety goal. The conservatisms included are substantial. The uncertainty is unestimated but is noted to be large in all such studies.

BACKGROUND:

SONGS 1 was originally constructed with capability to withstand a .5g earthquake . Over the past several years a number of modifications have been made to the plant so that a .67g earthquake could be survived. This is a conservative earthquake level for design purposes as documented elsewhere. Specific plant upgrades include structures, wall, control room ceiling, diesel generators, and critical systems. However, to date the modifications required to completely upgrade SONGS 1 to .67g are not complete. Dominant, risk sensitive, features have been nevertheless largely upgraded. In light of the substantial margin included and the status of upgrades, Southern California Edison Company has initiated a return to service plan. This study has been undertaken to evaluate the seismic risk for the unit based upon a combination of data from other studies (SSMRP) and a plant specific fault tree model of risk sensitive features. The frequency of earthquakes is taken to be higher than Zion (the study plant of SSMRP) for SONGS 1 and plant features of SONGS 1 are then evaluated conservatively to determine the effect of these plant features and their design level on composite risk.

GENERAL APPROACH TO ANALYSIS OF SONGS 1:

The general approach involves performing a seismic risk evaluation utilizing a mixture of inferred and site/plant specific data. A fault tree model similar to that of SSMRP was constructed and quantified. The result is a single value or point estimate most closely representing a mean value. The component parts of the analysis are shown below and discussed in the following sections:

- The hazard curve utilized is a .5 probability of exceedence curve for SONGS 1 site specific
- System features and fault trees are SONGS 1 specific
- Location specific accelerations are estimated for SONGS 1 specific data
- Component fragilities are from SSMRP
- Structural failure is assumed to occur as a step function for the earthquake band above .67g, the design value for structures
- Piping failure probabilities are estimated from SSMRP directly
- Random failure rates are from WASH 1400 and NUREG CR 1278
- Uncertainty is not estimated but is large

The result is a conservative estimate of SONGS 1 Seismic Risk. The estimate is derived from fault tree analysis for six different discrete earthquake levels and summed to approximate the total risk.

SEISMIC HAZARD - FREQUENCY OF EARTHQUAKE:

The frequency of severe earthquakes is assumed to be higher for SONGS 1 than for eastern or midwestern sites. The values for SONGS 1 are taken from work by Dr. Robert McNeill. The values adopted are from the 0.5 exceedence curve (figure 4 on the following page). Due to the methods of Dr. McNeill, it is not possible to define the curve as a median or mean value. It is felt to represent most closely a mean value and is taken to be the mean. Given the proximity of the .5 curve to the 0.0 curve, this may be conservative.

Since the current study is a comparative analysis using SSMRP, and hence Zion, as a base; it is appropriate to compare the curve used here with the values for Zion. This comparison is shown on the table following the exceedence curves. It should be noted that for earthquake levels 3 and 4, the greatest difference is assumed. These are generally important earthquake intervals, thus this difference is an indication of the conservatism.

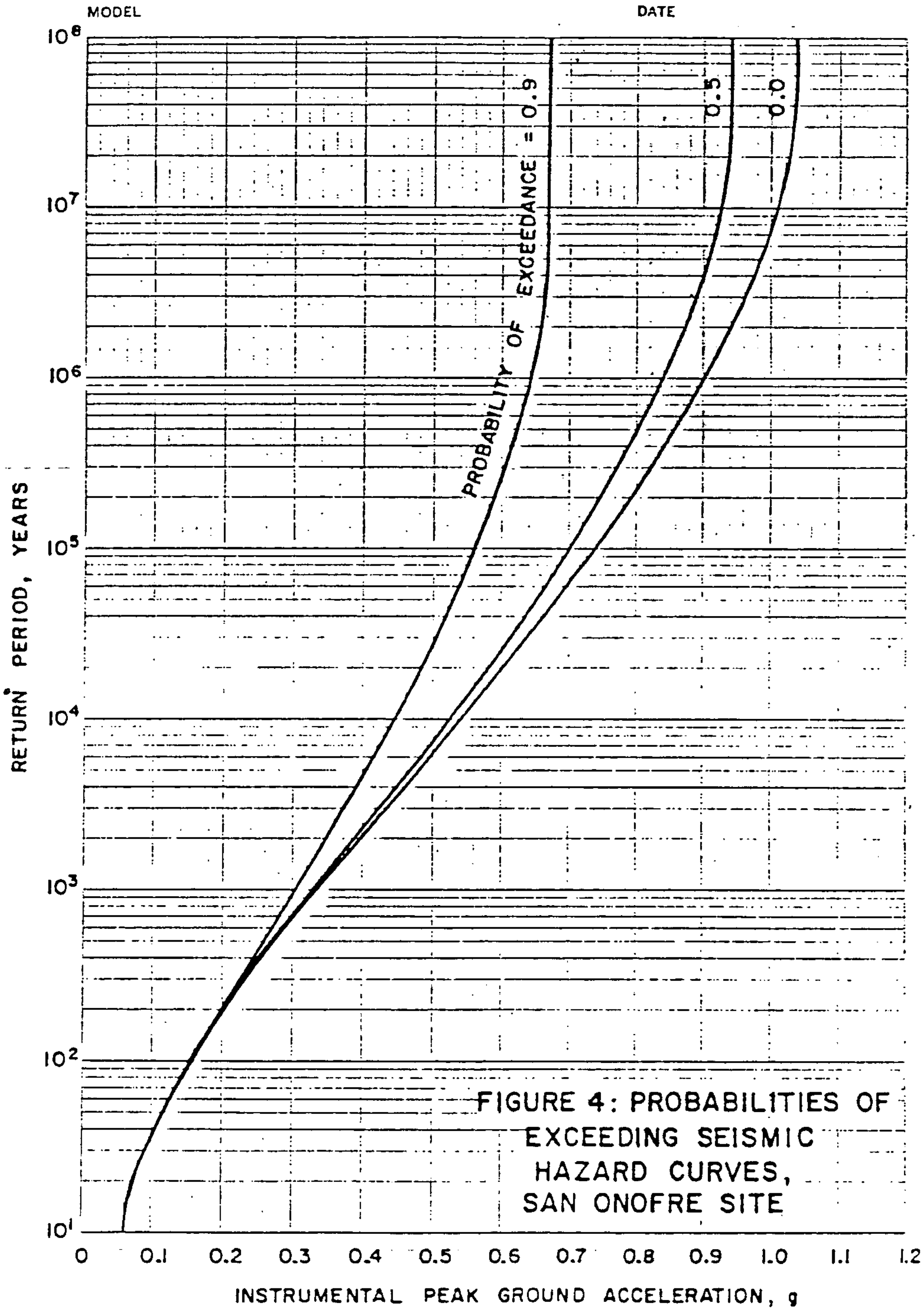


FIGURE 4: PROBABILITIES OF EXCEEDING SEISMIC HAZARD CURVES, SAN ONOFRE SITE

I Zion Versus SONGS 1 Hazard

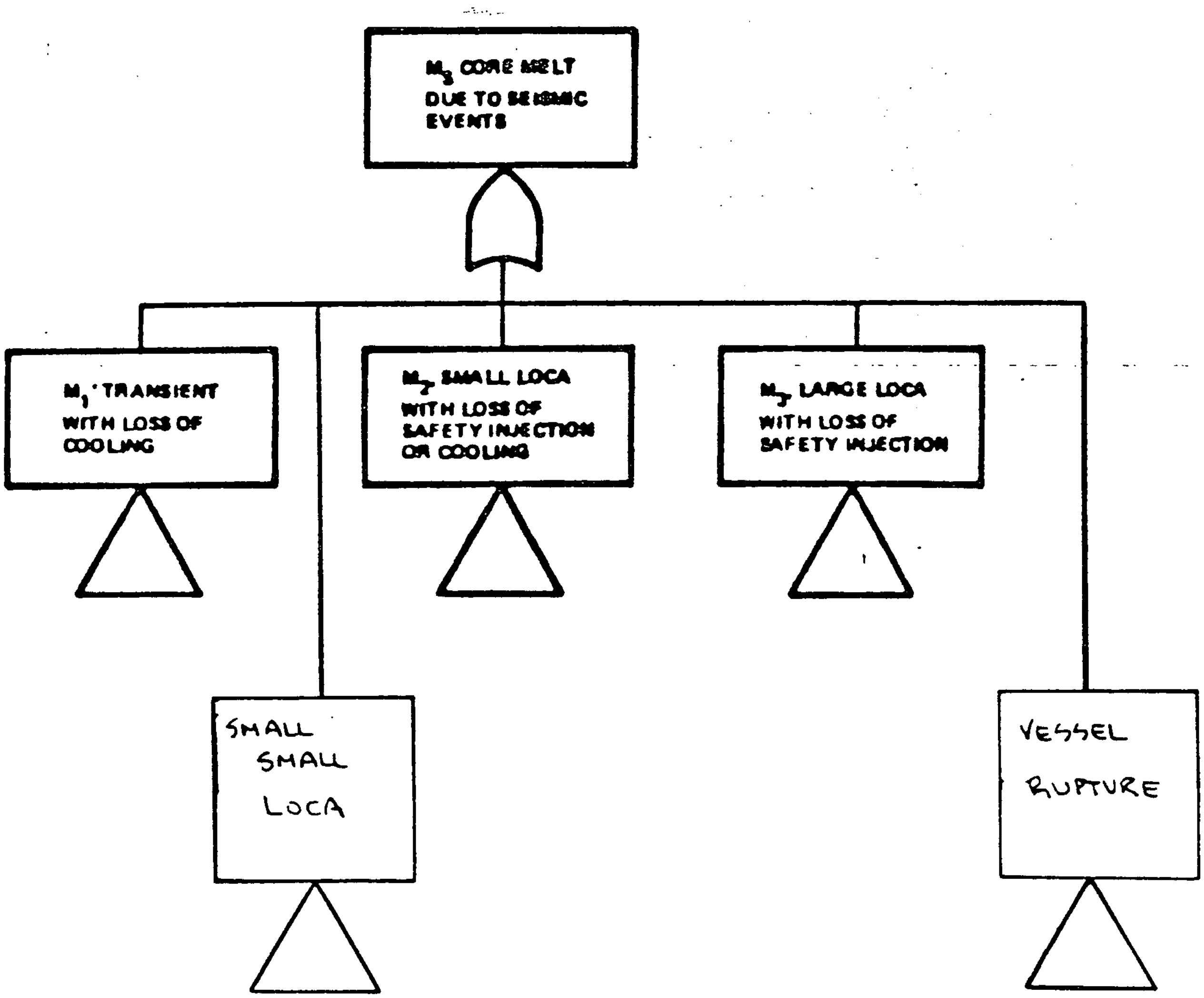
ZPGA Earthquake Level	Zion[1] Frequency #/Year	SONGS 1[2] Frequency #/Year
.15 - .30g	2.52(-4)	4.5(-3)
.3 - .45g	4.55(-5)	5.7(-4)
.45 - .6g	6.57(-7)	1(-4)
.6 - .75g	1.61(-7)	1.4(-5)
.75 - .9g	5.31(-8)	1.25(-6)
.9 - 3.0g	4.1(-8)	10^{-7}

1 - NUREG/CR - 2015 Vol. B

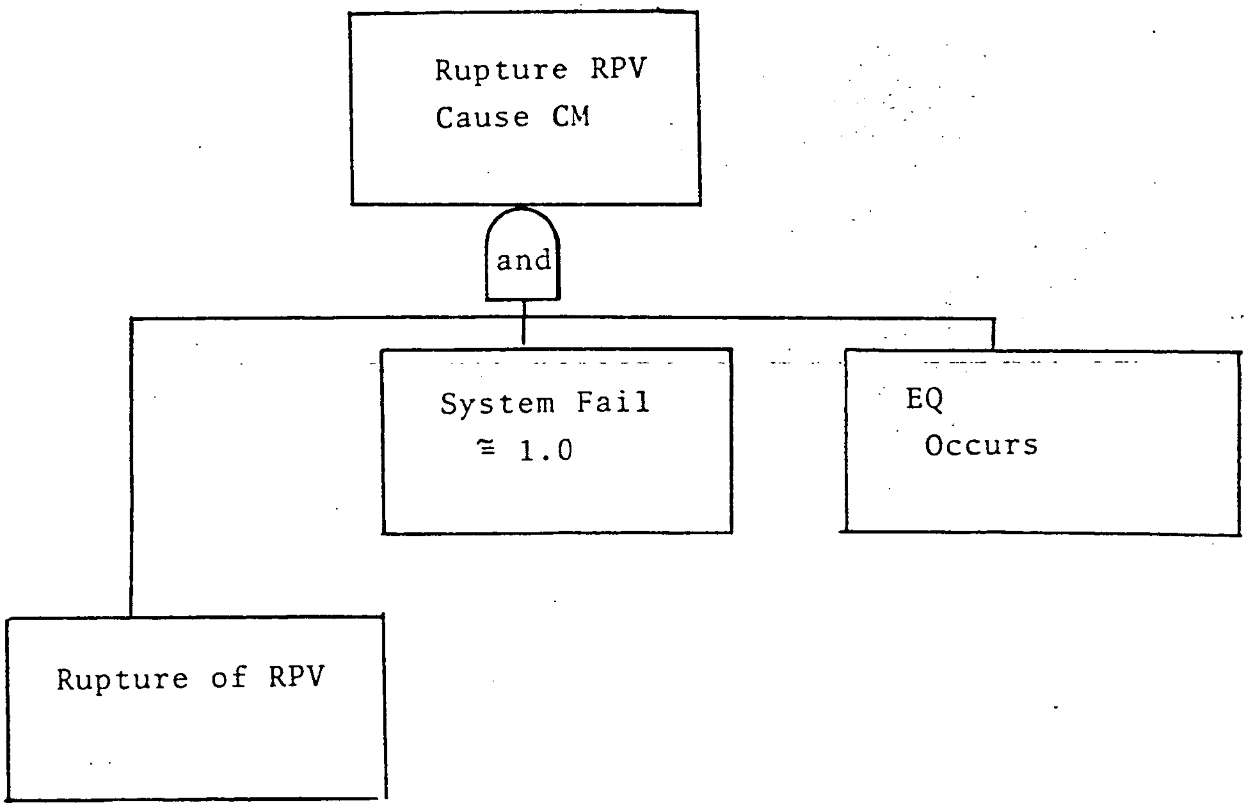
2 - McNeill/50% Exceedence Curve

PLANT MODEL - TOP LEVEL:

The following pages present the top level fault tree formulation for this study. As in SSMRP, the risk of earthquake is evaluated for five different types of events for six different earthquake levels. Subsequent analysis focuses on critical systens and their failure probability given an earthquake has occurred. In this analysis, dominant failure modes of SSMRP were evaluated to aid in analysis. For vessel Rupture and Large LOCA, this led to a presumption that the event occurred with a probability of 0.0 below .67g , the structural (dominant failure mode) design value and that failure was assumed at the next earthquake band, probability set equal to 1.0. This is felt to conservatively bound these events probability/frequency. Similarly small samll LOCA is given a small frequency due to the low likelihood of a stuck open valve. The two most likely cases are Transient and Small LOCA. For these the likelihood of the earthquake (each of six) causing the event is calculated by evaluating dominant failure paths. The dependent system failure probability is determined explicitly for each event and the top level event frequency determined.

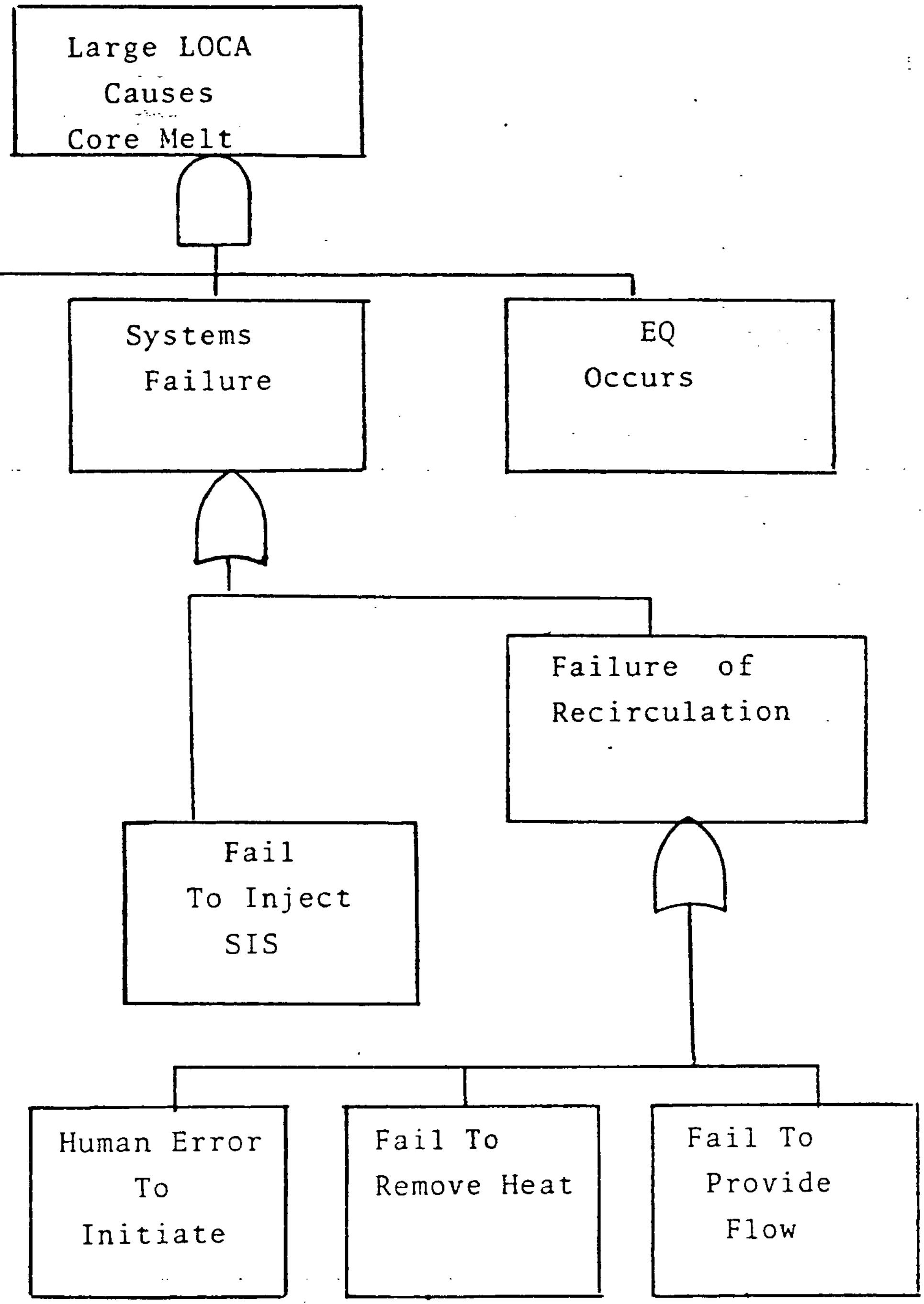


SONGS 1 MASTER FAULT TREE



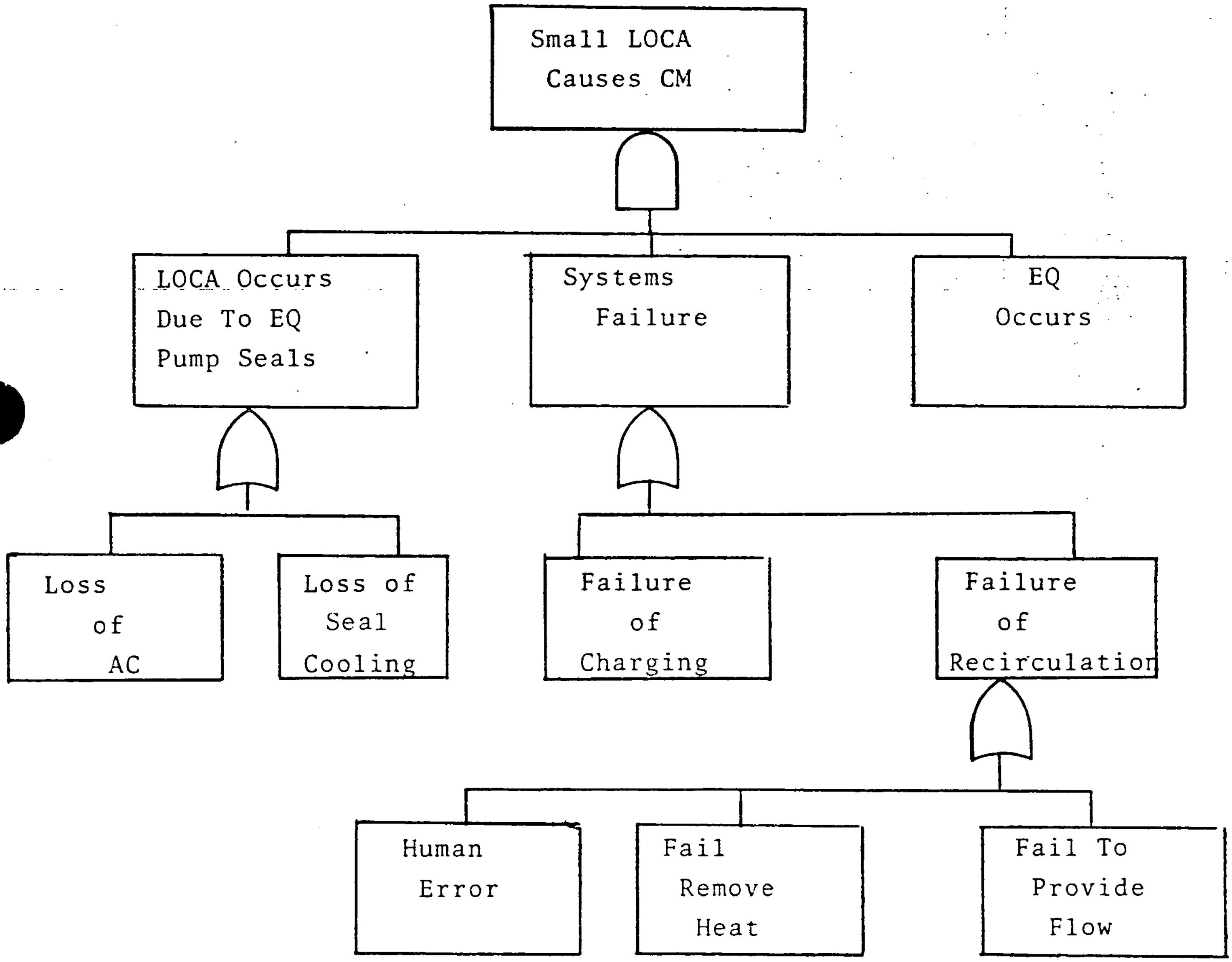
DOMINANT FAILURE CAUSES:

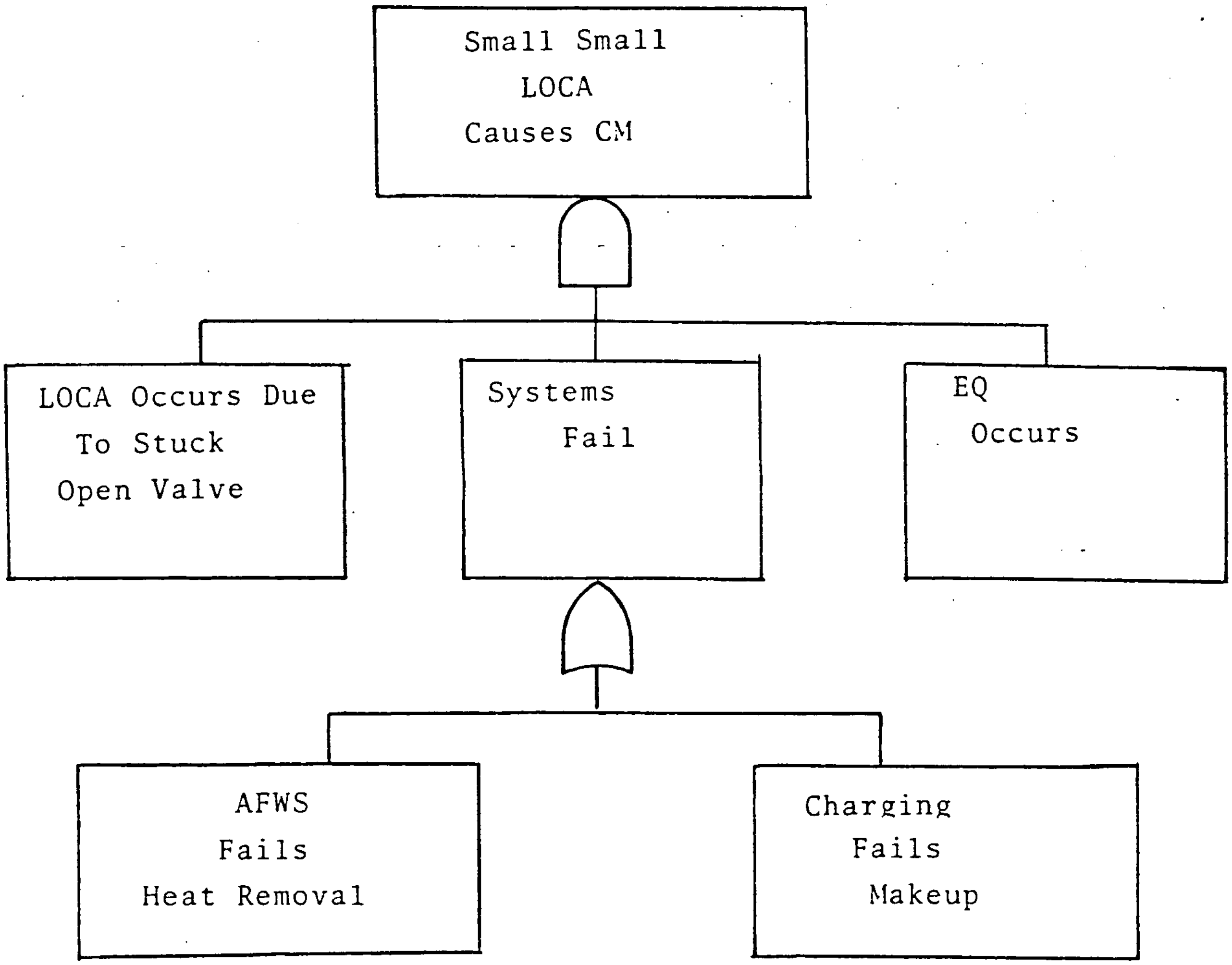
- TWO OR MORE FAILURES OF SUPPORTS FOR STEAM GENERATOR OR COOLANT PUMPS

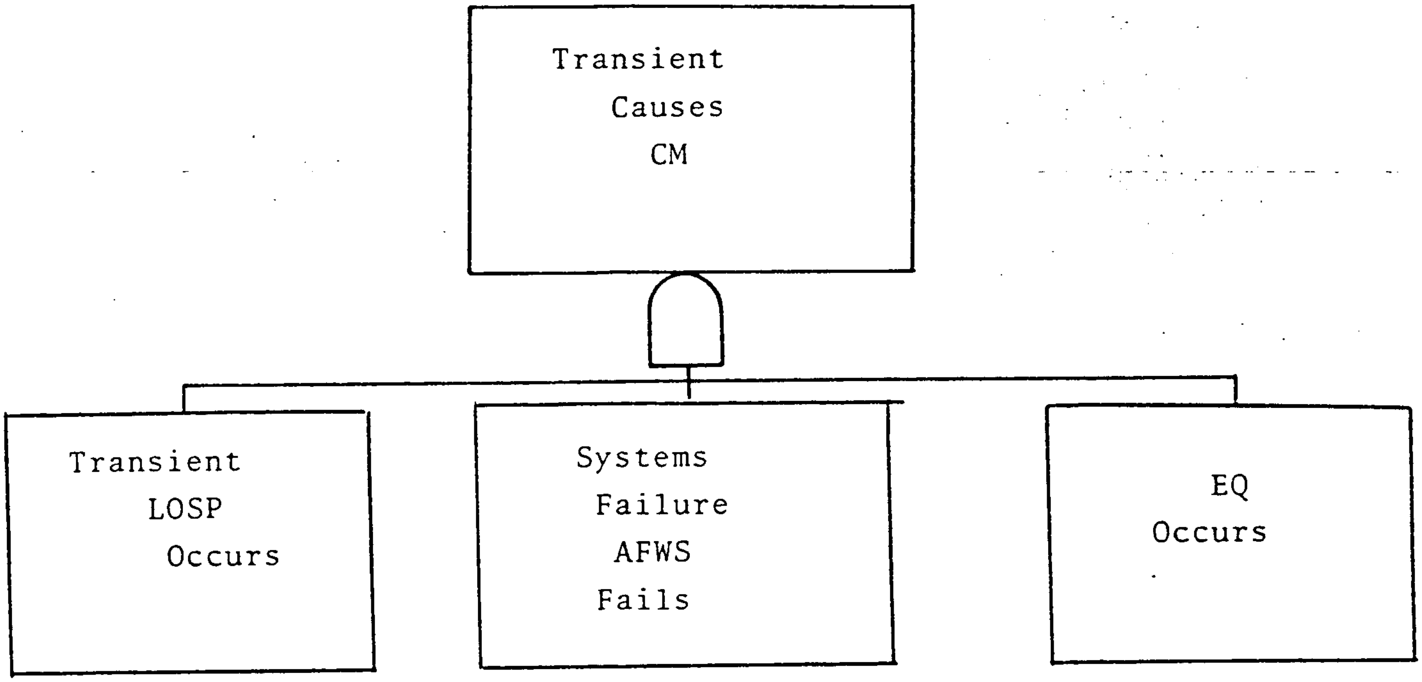


DOMINANT FAILURE CAUSES:

- FAILURE OF STEAM GENERATOR SUPPORT
- FAILURE OF REACTOR COOLANT PUMP SUPPORTS


Small LOCA Causes CM
LOCA Occurs Due To EQ Pump Seals
Systems Failure
EQ Occurs
Loss of AC
Loss of Seal Cooling
Failure of Charging
Failure of Recirculation
Human Error
Fail Remove Heat
Fail To Provide Flow


Small Small
LOCA
Causes CM
LOCA Occurs Due
To Stuck
Open Valve
Systems
Fail
EQ
Occurs
AFWS
Fails
Heat Removal
Charging
Fails
Makeup


Transient Causes CM
Transient LOSP Occurs
Systems Failure AFWS Fails
EQ Occurs

SYSTEM MODELS:

SONGS 1 specific system fault trees were constructed for Auxiliary Feedwater System, Charging System, and Safety Injection. Important dependencies were included. Special emphasis was placed on modeling piping segments due to both the on-going piping program at SONGS 1 and the relative significance of piping failures in SSMRP, particularly those that penetrate buildings with a potential for differential movement of fixed anchor points. The AFWS model was the most detailed to account for electrical dependencies since Transient Loss of Offsite Power is the main challenge of AFWS. A measure of the detail of the model is the number of cut sets or failure combinations analyzed. For SONGS 1 approximately 20,000 cut sets were defined and evaluated. The system drawings are shown in the following pages. Numbers in circles refer to pipe segments analyzed for failure effect and probability.

A number of plant system features are designed for .67g. In most cases these were not credited in the analysis due to lack of available data. This is a conservatism. Key assumptions for the systems are as follows:

AFWS

- The turbine driven pump is self cooling

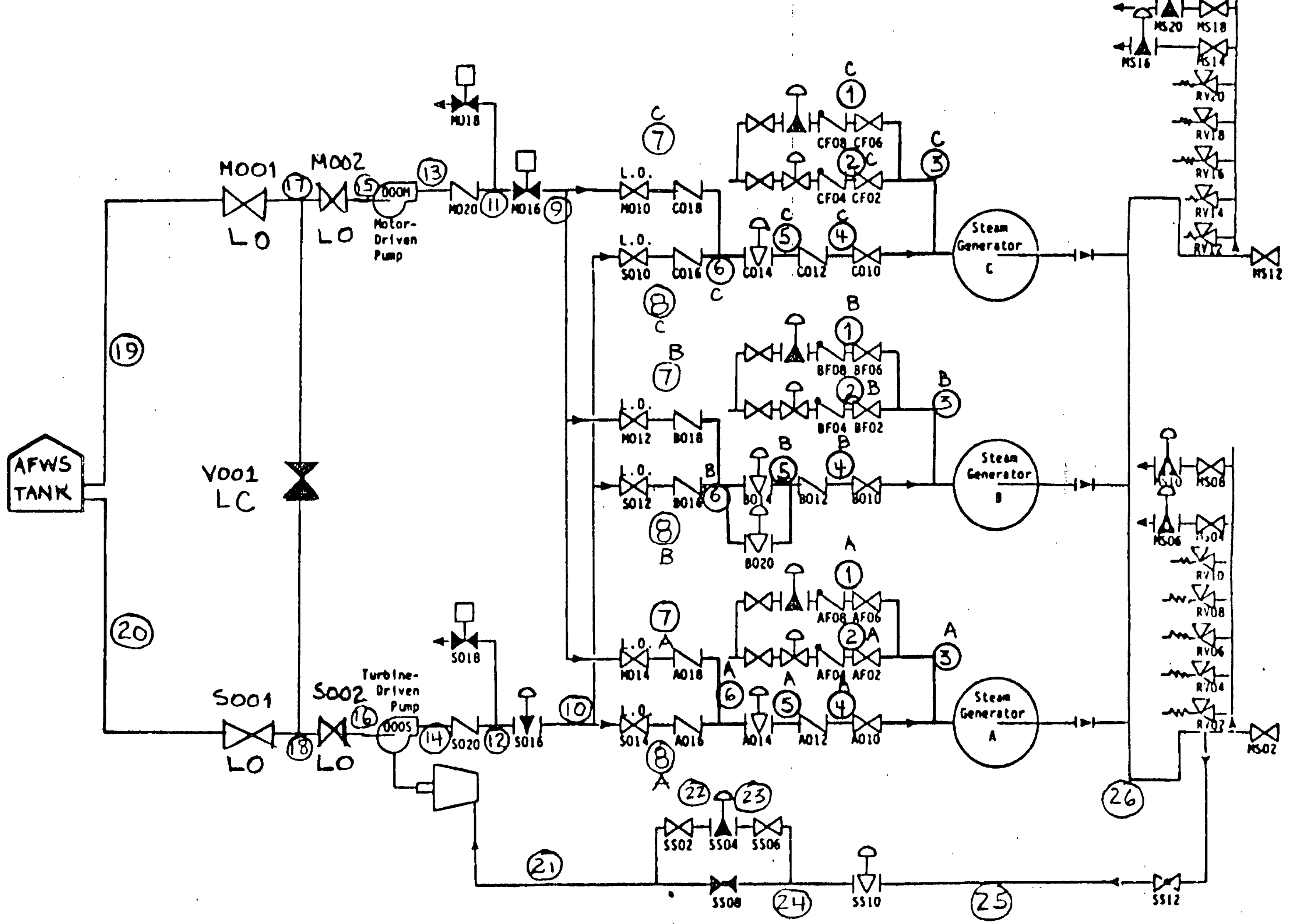
- In the event of a Transient given failure of AFWS and Feed and Bleed, core damage will start to occur after one hour

CHARGING

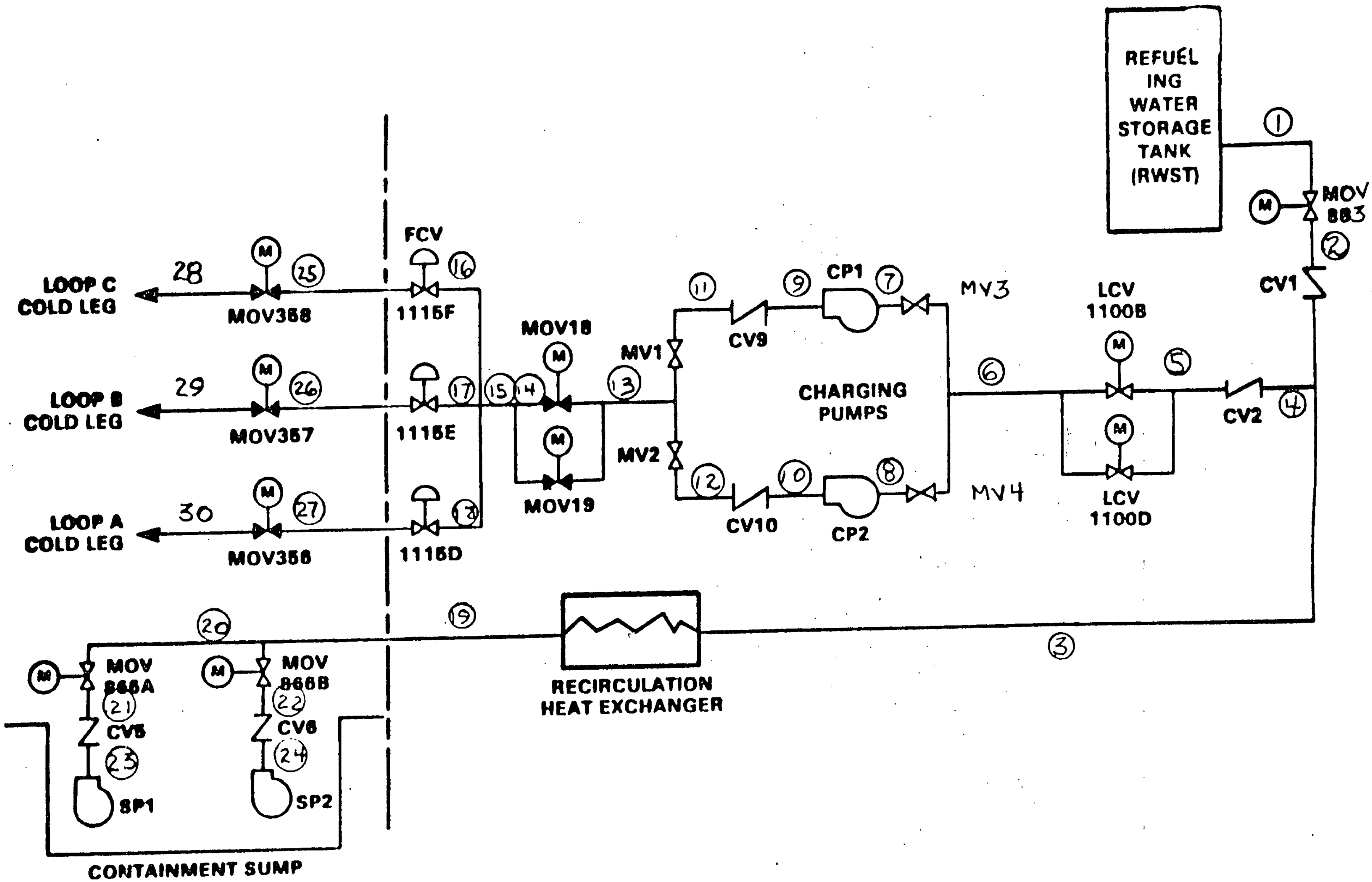
- Pumps can be self-cooled with cooling fans.

COMPONENT COOLING WATER SYSTEM

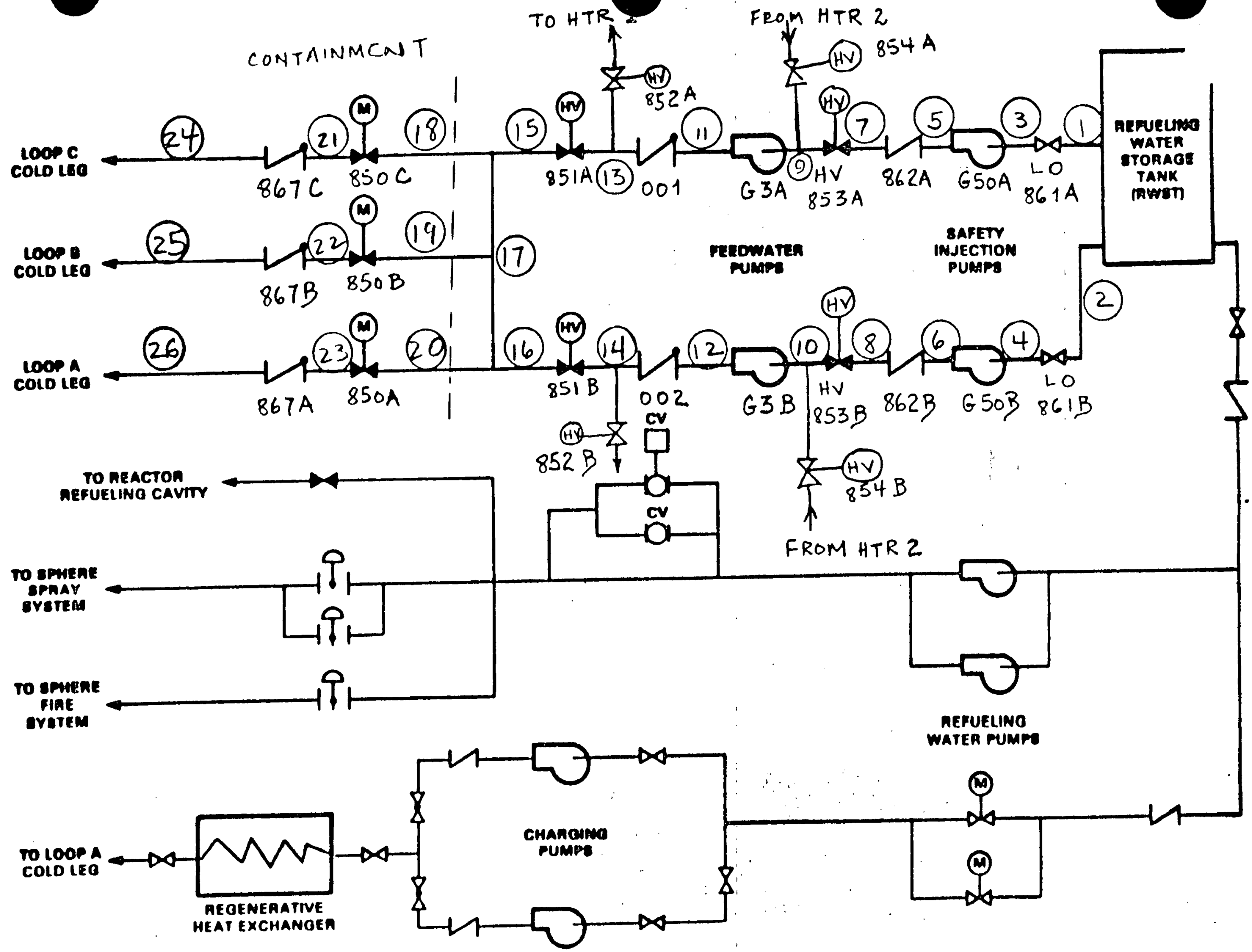
- In the event of a LOCA, the CCWS serves as an intermediate loop between the recirculation heat exchanger and the salt water cooling system. If heat removal fails, belated core melt will occur within 20 hours due to cavitation of the sump pumps.



San Onofre Auxiliary Feedwater System



SIMPLIFIED SCHEMATIC OF THE LONG TERM CORE COOLING SYSTEM



SIMPLIFIED SCHEMATIC OF THE EMERGENCY CORE COOLING INJECTION SYSTEM

LOCATION SPECIFIC RESPONSE - LOADINGS:

A major input to the computer analysis of the fault tree is the probability of failure due to the postulated earthquake. It is noted that buildings tend to amplify earthquake levels at higher elevations. Since SONGS 1 is largely a single floor plant local response would be expected to be less than for a multifloor plant. The seismic input was taken from Bechtel analyses of various locations and buildings of SONGS 1 at 0.67g. (Instructure Response Spectra, July 1982) The location response for structures for response modes in the rigid range were selected after a review of response mode frequency. These responses were combined by SRSS. For equipment, a review of expected response modes confirmed the validity of rigid range values.[1] The SONGS 1 exposure for .67g was determined (estimated) and exposure for other earthquake intervals scaled linearly from these values. The resultant values are ahown on the following table (in g's).

---

1 Piping treatment is discussed in a subsequent section.

EQUIPMENT RESPONSE

COMPONENT	ZION FRAGILITY	SONGS 1 EXPOSURE .15-.3g	.3-.45g	.45-.6g	.6-.75g	.75-.9g	.9g+
TANKS	2.01	.34	.56	.79	1.01	1.24	1.66
HX	1.85	.34	.56	.79	1.01	1.24	1.66
V. PUMPS	2.21	.39	.65	.91	1.16	1.43	1.90
M. PUMPS	3.19	.39	.65	.91	1.16	1.43	1.90
MOV'S	4.83	.39	.65	.91	1.16	1.43	1.90
REL/MAN. V'S	8.9	.50	.84	1.2	1.5	1.85	2.5
BATT. RACKS	2.3	.39	.65	.91	1.16	1.43	1.90
MCC'S	7.6	.39	.65	.91	1.16	1.43	1.90

FRAGILITY DATA:

In performing the analysis it is necessary to define equipment fragility. This has been done for Zion in SSMRP and shown on the following pages. This data was used directly in the analysis for components. It is noted that Zion equipment is the basis for the table. This equipment is designed to 0.17g, a value smaller than the 0.50g design values of SONGS 1 and the currently discussed 0.67g design value. It is likely that failure modes involving supports or structural failures are overstated by using these values. Fragility scaling was considered but not performed due to the fact that components with different design values may nevertheless, be identical.

Table 4. Summary of component fragilities.

Category	Fragility Median	β_R	β_U	Load Parameter	Frequency (Hz)	Damping % of Critical	Failure Mode[a]
Reactor core assembly	2.06	0.24	0.32	Spectral accel. g	5-15	5	Deformation of guide tubes
Reactor pressure vessel	3.83	0.23	0.39	Spectral accel. g	5	5	Fracture of RPV outlet nozzle
Pressurizer	2.00	0.40	0.34	Spectral accel. g	20	5	Failure of support skirt bolting
Steam generator	2.45	0.24	0.37	Spectral accel. g	5	5	Support failure
Reactor coolant pump	2.64	0.24	0.37	Spectral accel. g	5	5	Support failure
Piping (master fragility)	2.44×10^6	0.18	0.33	Moment in - lb	--	--	Plastic collapse
Large vertical vessels w/formed heads	1.47	0.20	0.35	ZPA g	Rigid	--	Failure of anchor bolts
Large vertical tanks w/flat bottom	2.01	0.25	0.29	ZPA g	Rigid	--	Failure of anchor bolts
Large horizontal vessels	3.90	0.30	0.53	Spectral accel. g	12-20	5	Failure of anchor bolts
Small to Medium vessels & heat exchangers	1.85	0.25	0.45	Spectral accel. g	20	5	Failure of anchor bolts
Large vertical centrifugal pumps with motor Drive	2.21	0.22	0.32	Spectral accel. g	5	5	Failure of support connection
Motor driven pumps & compressors	3.19	0.21	0.27	Spectral accel. g	7	5	Impeller deflection

Table 4. (Continued)

Category	Fragility Median	β_R	β_U	Load Parameter	Frequency (Hz)	Damping % of Critical	Failure Mode[a]
Large motor operated valves	4.83	0.26	0.60	Piping peak accel. g	Rigid	--	Distortion of extended operator
Large motor-operated valves	14.40	0.28	0.56	Piping peak	Rigid	--	Structural failure
Small motor operated valves	9.84	0.26	0.60	Piping peak accel. g	Rigid	--	Distortion of extended operator
Large hydraulic and air actuated valves	7.61	0.31	0.34	Piping peak accel. g	Rigid	--	Loss of control air
Large relief, manual, and check valves	8.90	0.20	0.35	Piping peak accel. g	Rigid	--	Internal damage
Miscellaneous small valves	12.50	0.33	0.43	Piping peak	Rigid	--	Internal damage
Horizontal motors	12.10	0.27	0.31	ZPA g	Rigid	--	Binding of rotating parts
Generators	0.65	0.25	0.31	Spectral accel. g	22	5	Shutdown valve trip
Battery racks	2.29	0.31	0.39	ZPA g	Rigid	--	Failure of battens
Switchgear	2.33	0.47	0.66	Spectral accel. g	5-10	5	Spurious operation of a protective relay
Dry transformers	2.78	0.28	0.30	Spectral accel. g	10	5	Failure of anchor bolts
Control panels and racks	11.50	0.48	0.74	Spectral accel. g	5-10	5	Dislodging or malfunction of component
Auxiliary relay cabinets	7.63	0.48	0.66	Spectral accel. g	5-10	5	Breaker trip

Table 4. (Continued)

Category	Fragility Median	β_R	β_U	Load Parameter	Frequency (Hz)	Damping % of Critical	Failure Mode[a]
Local instruments	7.68	0.20	0.35	Spectral accel. g	5-35	5	Loosening of fasteners
Motor control centers	7.63	0.48	0.74	Spectral accel. g	5-10	5	Breaker trip
Communications equipment	5.00	0.33	0.35	Spectral accel. g	10-50	5	Dislodging of components
Light fixtures	9.20	0.14	0.14	Spectral accel. g	20-30	2	Dislodging of components
Inverters	15.60	0.26	0.35	Spectral accel. g	5-10	5	Relay trip
Cable trays	2.23	0.34	0.19	ZPA g	Rigid	--	Support system failure
Circuit breakers	7.63	0.48	0.74	Spectral accel. g	5-10	5	Breaker trip
Relays	4.00	0.48	0.75	Spectral accel. g	5-10	5	Relay chatter
Ceramic insulators	0.20	0.25	0.25	PGA g	2-8	5	Fracture of porcelain
Air handling units	2.24	0.27	0.31	Spectral accel. g	5	5	Rubbing of fan on housing
Instrument racks and panels	1.15	0.48	0.66	Spectral accel. g	5-10	5	Relay chatter
Duct work	3.97	0.29	0.46	Spectral accel. g	5-10	7	Structural failure
Hydraulic snubbers and pipe supports	1.46	0.22	0.49	ZPA g	Rigid	--	Weld failure

[a]Only the most likely failure mode is listed, although the fragility may be based on a combination of modes.

FAILURE PROBABILITIES:

The component failure probabilities were determined for each earthquake level by converting the median of the fragility to a mean, and integrating the fragility curve with the response.

As was assumed in the SSMRP, component fragilities and local responses were assumed to be log normal random variables. Failure probability is computed by calculating the overlap of the tails of the two distributions. Examining the SSMRP data for Zion, all cases relevant to SONGS 1 analysis have the standard deviation of the response very much smaller than the standard deviation of the fragility. For this reason, the response is treated as a deterministic random variable. (This result is consistent with the sensitivity analysis conducted by the SSMRP which showed that varying the response had little effect on the core melt probability at Zion.)

It was further assumed that fragilities were statistically independent. In all cases but one, the statistical dependence of fragilities was not an issue because single pipe ruptures as analyzed dominated the system failures. One exception was the simultaneous rupture of all three discharge headers in the main feedwater system which was the dominant cut set for the AFWS. However, this piping is qualified to .67g. Since the highest rupture probabilities from the Zion AFWS study were used, the analysis is conservative.

COMPONENT	COMPONENT FAILURE PROBABILITY					
	.15-.3g	.3-.45g	.45-.6g	.6-.75g	.75-.9g	.9g+
TANKS	ε	ε	ε	ε	ε	.05
HX	ε	ε	ε	ε	.003	.23
V. PUMPS	ε	ε	ε	ε	.006	.05
M. PUMPS	ε	ε	ε	ε	ε	ε
MOV'S	ε	ε	ε	ε	ε	ε
REL/MAN. V'S	ε	ε	ε	ε	ε	ε
BATT. RACKS	ε	ε	ε	ε	.004	.03
MCC'S	ε	ε	ε	ε	ε	ε
BURIED PIPE	ε	.008	.07	.2	.38	.68

STRUCTURAL FAILURE:

All structures at SONGS 1 are designed for a 0.67g earthquake. It is noted that fragilities (median) of 3 to 4g are quoted in SSMRP for Zion structures designed for 0.17g. Determining rigorously the failure probability for such structures is beyond the scope of this analysis. A conservative estimate was derived by assuming a probability of zero or epsilon for earthquake bands up to and including the 0.67g design value. For earthquake interval five and six a failure probability of 1.0 was assumed. This level of conservatism is included as a measure of core damage potential Offsite effects measures of significance should not be based upon or inferred from this assumption due to its believed significant conservatism.

PIPING FAILURE:

Piping failure in SSMRP is determined from detailed analyses of movement and response. For this assessment the values of SSMRP for failure probability were used directly. Review of SSMRP data indicated that failure probabilities had the following properties:

- Highest failure rates occurred for pipes with near fixed anchor points and differential movement

- Other failure rates varried over ten orders of magnitude with highest probabilities approximately 0.1 times the highest values

Based upon this data, the highest failure rates for SSMRP were adopted for pipes penetrating the containment and 0.1 times these values were used for all others. It is noted that pipes penetrating the containment generally do not have nearby anchor points making this a conservative assumption. Other pipes generally enter the turbine building through loose penetrations without fixed anchors causing high stress due to differential movement.

PIPE DATA

FIXED PENETRATIONS X1.0/WITHIN BUILDING X0.1

	SOURCE	.15-3g	.3-.45g	.45-.6g	.6-.75g	.75-.9g	.9g
3-4 in.	SSMRP	3(-3)	9(-3)	0.1	0.26	0.34	.53
6-10 in.	ESTIMATED	2(-3)	9(-3)	.095	.23	.33	.5
16-24 in.	SSMRP	1.3(-3)	1(-2)	9(-2)	0.2	.33	.46

RANDOM FAILURE AND HUMAN FAILURE PROBABILITIES:

Generally component random failure probabilities were taken from WASH 1400. Human error rates were from Swain Guttman as shown on the following chart. Diesel Generator failure rates were used as shown on the following page. It is noted that the plant specific experience with diesel failures is very good and exceeds the values used in the analysis.

HUMAN ERROR RATES

- SWAIN-GUTTMAN NUREG CR 1278 WITH JUDGEMENT
- POST LARGE LOCA .1 @ 30 MIN. .01 @ TWO HOURS
- FOR ACTIONS AT OR BEFORE 1 HOUR: .1 USED IF REMOTE TO CR
  .05 USED IF IN CR
- FOR ROUTINE ACTIONS AT 1/2 HOUR .1 USED
- FOR 20 HOUR CROSS CONNECT TO RFWST .01 USED (CONSERVATIVE)

FOR DG FAILURE:

PLANT DATA:	1 DIESEL: 5E-3
	2 DIESELS: NO FAILURES IN 420 TRIES
INDUSTRY DATA:	1 DIESEL: 5E-2
	2 DIESELS: 9E-3
VALUE USED	1 DIESEL: 1E-2 (2/3)[1] = 6.6E-3
	2 DIESELS: (1/420)(2/3)(2/3) = 1E-3

---

1 PROBABILITY OF NO REPAIR IN TWO HOURS
(NRC DIESEL REPAIR DATA)

RESULTS OF ANALYSIS:

The following charts provide the results. System failure probabilities are shown first, followed by a small LOCA and Transient frequencies. The values shown are for each of the six postulated earthquake intervals. The total Core Damage Assessment presents the total result and sums all contributors. The core damage frequency is conservatively estimated to be $2x10^{-5}$/year, without Feed and Bleed. A lower value is derived with credit for Feed and Bleed.

These values are below the safety goal ($1x10^{-4}$/year) and roughly equivalent to SSMRP values for Zion. Given the conservative nature of this assessment, the result suggests that the seismic hazard at SONGS 1 is small.

	EQ1	EQ2	EQ3	EQ4	EQ5	EQ6
EVENT (per yr)	4.5E-3	5.7E-4	1.0E-4	1.4E-5	1.3E-6	1E-7
AFWS	6.7E-4	2.6E-3	4.6E-2	2.0E-1	3.6E-1	6.9E-1
CHG.	1.2E-2	2.8E-2	2.2E-1	4.9E-1	6.3E-1	8.7E-1
SIS	8.0E-3	3.3E-2	3.1E-1	6.3E-1	7.7E-1	9.2E-1
FEED & BLEED	4.5E-3	1.3E-2	1.2E-1	3.2E-1	4.2E-1	8.1E-1

SMALL LOCA

	EQ1	EQ2	EQ3	EQ4	EQ5	EQ6
EQ (per yr.)	4.5E-3	5.7E-4	1.0E-4	1.4E-5	1.5E-6	1.0E-7
SMALL LOCA	1.2E-2	4E-2	3.8E-1	7E-1	8E-1	9.5E-1
SYSTEMS FAILURE	1.2E-2	2.8E-2	2.2E-1	4.9E-1	6.3E-1	8.7E-1
CORE DAMAGE	6.5E-7	6.4E-7	8.4E-6	4.8E-6	7.6E-7	8.3E-8

TRANSIENT

	EQ1	EQ2	EQ3	EQ4	EQ5	EQ6
EQ (per yr.)	4.5E-3	5.7E-4	1.0E-4	1.4E-5	1.3E-6	1.0E-7
TRANSIENT	2.6E-1	7.6E-1	6.2E-1	3E-1	2.0E-1	5E-2
SYSTEMS FAILURE	6.7E-4	2.6E-3	4.6E-2	2.0E-1	3.6E-1	6.9E-1
CORE DAMAGE W/O F&B (per yr.)	7.8E-7	1.1E-6	2.9E-6	8.4E-7	9.4E-8	3.5E-9
CORE DAMAGE W/F&B (per yr.)	ϵ	1.4E-8	3.4E-7	2.7E-7	3.9E-8	2.8E-9

TOTAL CORE DAMAGE ASSESSMENT

EQ LEVEL	RUPTURE	LOCA	SMALL LOCA	SMALL SMALL LOCA	TRANSIENT
1	ϵ	ϵ	6.5E-7	ϵ	7.8E-7
2	ϵ	ϵ	6.4E-7	ϵ	1.1E-6
3	ϵ	ϵ	8.4E-6	ϵ	2.9E-6
4	ϵ	ϵ	4.8E-6	ϵ	8.4E-7
5			7.6E-7	ϵ	9.4E-8
	1E-6				
6			8.3E-8	ϵ	3.5E-9
TOTAL	1E-6 (1)		1.5E-5 (2)	ϵ	5.7E-6 (3)

CORE DAMAGE = 2.17 E-5
W/O F&B (1+2+3)

CORE DAMAGE = 1.6E-5
W/ F&B (1+2)

ALL VALUES PER YEAR

COMPARISON OF RESULTS:

The above results can be compared to the value of core damage for Zion for SSMRP. The SONGS 1 analysis is felt to be conservative rather than a best estimate, nevertheless,

SSMRP	Zion	Median	=	$3x10^{-5}$/year
SSMRP	Zion	Mean	=	1 to $2x10^{-5}$/year
SONGS 1 Mean		Estimate	=	$2x10^{-5}$/year

Due to the estimating technique of this study it is not possible to provide a distribution for the estimate. Nevertheless, it is clear that the seismic risk for SONGS 1 is estimated to be approximately equal to the seismic risk of Zion.

We estimate the uncertainty to be large and dominated by the uncertainty in the hazard curve. SSMRP calculated the median core melt probability for Zion to be 3 E-5 per year, and the upper 90% confidence value to be 8 E-4 per year,ie. a factor of 26 between the median and the upper 90% confidence level. Since much data from SSMRP was used, estimating such a factor for SONGS 1 is not unreasonable.

LIMITATIONS OF THE ANALYSIS:

The assessment reported herein contains numerous conservatisms. Rigorous analysis of these factors would tend to reduce the risk estimate and might cause some shifting of assessed contribution to risk for the five types of events. The likelihood of one or the other is limited by the fact that all cannot occur with a probability greater than unity given the occurrence of the earthquake. This factor tends to lend credence to the validity of an assessment but could introduce some error in the evaluation of competitive design features where choices are evaluated involving different damage scenarios. Thus caution is suggested regarding use of this analysis for prioritization of specific upgrades or modifications.

The analysis is performed with and without Feed and Bleed consistent with the general analytical approach of SSMRP. This does not represent a judgment regarding Feed and Bleed but is done to increase comparative capability.

The diesel generators and all supports are designed for 0.67g and enclosed in a 0.67g structure. This led to an assumption that random failures dominate diesel reliability. This is clearly true at lower earthquake levels and is felt to be only moderately effected at the highest level.

Localized soil failures are not included in the analyses as Southern California Edison is actively verifying the local soil properties.

The statistical approach is felt to be appropriate for an assessment - that is, the development of a conservative mean estimate and comparison of the result to SSMRP mean and median values. Nevertheless, the uncertainty is certainly large. The conclusion of the study is that Zion and SONGS 1 have similar earthquake risk. The former is dominated by structural and power failures. Both of these have been hardened for SONGS 1. The model of SONGS 1 conservatively assumes that the piping failure probability is equal to Zion's at the same earthquake level. Due to higher design g-value and partial upgrades to .67g at SONGS 1, this is conservative. Nevertheless, on this basis, piping failures dominate the current assessment.

CONCLUSIONS:

Based on this analysis it is concluded that the earthquake risk at SONGS 1 is approximately equal to the earthquake risk at Zion. In the review and system modeling, no specific area of concern was identified which Southern California Edison has not addressed or is not addressing as part of the return to service program. Much data from SSMRP has been adopted and used directly; in all cases there is strong reason to believe that such use is conservative. Finally, the uncertainty is large, but the results are consistent with SSMRP. Based on this analysis, there appears to be no reason for SONGS 1 not to be returned to service consistent with the upgrades already performed and included in the analysis.

APPENDIX D

FUNCTIONALITY CRITERIA FOR

PIPING SYSTEMS IN RESPONSE TO THE DBE EVENT

SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1

December 1983

SAN ONOFRE NUCLEAR GENERATING STATION
UNIT 1
FUNCTIONALITY CRITERIA FOR
PIPING SYSTEMS IN RESPONSE
TO THE DBE EVENT

Prepared for:

Southern California Edison

Prepared by:

EDS Nuclear Inc.

December 1983
EDS Report No. 04-0310-0063
Revision 2

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TABLES

FIGURES

1.0 INTRODUCTION

Southern California Edison (SCE) has been participating in the Systematic Evaluation Program (SEP) for San Onofre Nuclear Generating Station, Unit 1 (SONGS-1) for several years. As part of this program, SCE is demonstrating that all safety-related piping systems at SONGS-1 can withstand a design basis earthquake (DBE) at the site. The Nuclear Regulatory Commission (NRC) and SCE have agreed upon criteria for piping which, when satisfied, will assure the integrity and functionality of the piping systems during and after a DBE. These criteria are based on the ASME Boiler and Pressure Vessel Code, Section III (referred to as the Code) requirements, which are widely accepted as appropriately conservative design bases. However, it has been recently proposed that the Code requirements are overly conservative when applied to cases involving short-term dynamic loads, such as the DBE. Diverse nuclear industry groups, such as the ASME code committees, Electric Power Research Institute (EPRI), the NRC, and concerns in foreign countries (e.g. Japan and West Germany), have for the past few years been investigating this issue (References 1 through 8). The final resolution of the issue and changes to the Code are still several years away, however the trend in the results is clear - piping systems subjected to short-term dynamic loads can maintain integrity and functionality at stress levels well above Code allowables without a decrease in safety margins.

At SONGS-1, SCE is presently upgrading all piping systems to meet the SEP criteria. Due to the large amount of construction work involved in completing the upgrade program, the schedule for return to power may be extended. To allow a more expeditious schedule, SCE may instead complete partial modifications of the piping systems. The success of this approach depends upon revising the criteria for piping integrity and functionality to take credit for the observed higher margins against failure under short-term loads.

The new criteria, which we have called the functionality criteria, is shown to be applicable for SONGS-1 piping, and includes sufficient conservatism to cover any uncertainty in the seismic analysis procedures. EDS has developed such a functionality criteria in response to SCE's intention to return to power prior to completion of the seismic upgrade program for piping. Section 2 of this report describes the functionality criteria and its basis. Justification of the criteria requires demonstration of applicability to SONGS-1 piping systems. The analysis program used for this justification is outlined in Section 3. The program involves selecting representative piping systems at SONGS-1 (Section 4) and demonstrating through nonlinear analysis that integrity and functionality are maintained at the criteria stress limits (Section 5). The conclusions of the study are summarized in Section 6. The functionality criteria is justified for SONGS-1 without regard for its intended application.

However, the arguments in support of the criteria may differ if they are applied in low probability situations (e.g. occurrence of a DBE over a short time period) or to systems with less safety significance (e.g. accident mitigation systems as opposed to safe shutdown systems).

2.0 DEVELOPMENT OF THE FUNCTIONALITY CRITERIA

The objective of the seismic upgrade program being performed at SONGS-1 is to demonstrate the ability of the unit to successfully shutdown following a 2/3 g level earthquake. The design requirements to assure safe shutdown are those requirements imposed by the NRC under the SEP. This design basis originates from the ASME Code requirements for piping systems. However, operability and functionality (i.e. the capability of a system to function immediately after an earthquake until safe shutdown is achieved) can often be established using less restrictive criteria.

The criteria requires that a system have the capacity to function during and immediately after an earthquake. This level of system performance is consistent with less restrictive load limits than those specified by the NRC for the faulted condition -- limits that allow permanent deformations of a finite nature. The bases of these limits are general functionality and plastic limit analysis considerations. For piping systems, the criteria allows an increase in the primary stress allowable to twice yield for carbon steel components and 2.2 times yield for stainless steel components. These piping stress allowables, which are compared to stresses calculated from a linear elastic analysis, reflect the added capacity of a piping system beyond Code limits when subjected to short-term seismic loading. These allowables are justified through the discussion and analyses that follow.

The SONGS-1 functionality criteria were developed to include significant identifiable conservatisms inherent in ASME Code piping analysis procedures. These conservatisms include:

- Damping values range between 2 and 5 percent for DBE loading as specified by Reg. Guide 1.61 [9]. Observed damping values for piping systems at high stress levels are much higher due to effects such as gaps in supports and flexible boundary conditions. Current Pressure Vessel Research Council (PVRC) task group activities are investigating redefining damping to higher, more reasonable values [10].

- Strain rate effects are neglected in the criteria. These effects can significantly increase the yield stress in dynamic loading cases.

- Stress intensification and flexibility factors considered are extremely conservative as defined in the linear elastic analyses. These factors result in greater susceptibility to yielding under smaller loads in components such as elbows and tees; however, there is no consideration for load redistribution to other components following initial yield.

- Pressure effects which increase the ultimate load capacity of components are not taken credit for in the criteria, although pressure stresses are included in the evaluations.

- Component thicknesses are normally greater than the nominal dimensions specified. This increase in thickness can have a significant effect on component capacity.

- Actual material strengths are generally at least 10 percent greater than Code specified minimums.

Current Code allowables for dynamic loading are also recognized as extremely conservative, especially for seismic motion. For the elastically-calculated stress levels of 2.0 S_y for carbon steel piping and 2.2 S_y for stainless steel piping, actual yielding of the piping systems are expected to be of a limited local nature. This prediction is based on the characteristics of seismic motion as well as the nonlinear behavior of piping systems:

- The energy in any seismic motion is finite. As a piping system yields locally, much of the input energy is absorbed as strain energy, and the kinetic energy of the system is reduced.

- Nonlinear damping effects significantly decrease the response of a system after some amount of yielding.

- The inertial effects of a typical piping system limit the deformations and hence the extent of local yielding.

- The system redundancy allows yielding at multiple locations. In this manner, system collapse due to formation of a mechanism is highly unlikely, and loading will be redistributed to different components such that excessive yielding will not occur in any one component.

Additional qualitative insight into the dynamic behavior of piping can be obtained from operating plants which have experienced strong ground motions. The El Centro Steam Plant [11], Lawrence Livermore Laboratory, and the Hamaoka Units in Japan have all been subjected to earthquake motion without disruption of operation. SRV discharge piping systems in BWR plants have also been subjected to dynamic loads without damage, where conventional analysis indicates dynamic stresses well above current Code allowables.

Recently, the PVRC Task Group on Dynamic Loading has undertaken a program to develop more rational criteria for the evaluation of piping systems under transient loading [12]. This group recognizes the conservatism in current ASME Code practice, and is sponsoring research into the behavior of typical piping systems under dynamic loads to direct Code considerations towards the actual response and failure modes of those systems.

Several experimental programs to investigate the yielding of piping systems have recently been completed or are currently underway. ANCO Laboratories has performed two sets of dynamic tests on Kraftwerk Union piping systems in West Germany [1,2,3]. One set utilized nine typical small-bore piping configurations of up to 300 feet in length with a variety of components and standard piping support systems. These systems were subjected to both low and high frequency loads of various amplitudes corresponding to seismic and aircraft impact loads, respectively. The maximum low frequency excitations with a maximum peak acceleration of 12 g were applied for durations of ten seconds. The maximum high frequency excitations with a maximum peak acceleration of 24 g over the 20 to 40 Hz frequency range were applied for durations of approximately one second. Peak acceleration response of 50 g, peak displacements of 50 cm, and plastic strains in excess of 0.6 percent were reported. Linear elastic analysis predicted dynamic stresses over four times ASME Code allowables. Even for these extreme loads, there was no observed failure due to plastic collapse, leakage, or loss of pressure-retention capability. This program was presented to West German licensing agencies to justify existing installations without backfitting for dynamic loads, and to provide licensing support for the elimination of primary stress requirements for these loads on small bore (less than 2-inch diameter) piping.

High-excitation testing to benchmark dynamic nonlinear analysis methods for piping [4] is currently being conducted for EPRI. One test has been completed on a 4-inch Schedule 40 ferritic steel piping system. This system has a length of 20 feet and consists of two elbows and three runs of piping. The system was designed to ASME Class 2 rules. The system was pressurized to design allowables and subjected to various dynamic excitation levels corresponding to seismic events. The primary purpose of this initial test was to demonstrate the feasibility of dynamically exciting piping systems to levels far in excess of current Code allowables. The maximum dynamic excitation level corresponded to seven to eleven times a typical SSE spectra for a plant in a low to moderate seismic region. This excitation level results in stresses which exceed Level D Code allowable stress limits by a factor greater than three. Permanent and visible deformations were observed, but there was no plastic collapse or loss of structural integrity in the pressurized piping. Input accelerations were greater than 14 g, and response accelerations were greater than 21 g in one elbow. Plastic strains greater than 1.5 times the yield strains were recorded.

A limited amount of dynamic component testing has also been conducted [5,6,7,8]. Straight pipe test data on fixed and pin-ended spans were developed in a joint Lawrence Livermore National Laboratories/Sargent and Lundy study. Strain levels with corresponding stresses up to 130% of yield were observed. A Japanese experimental study tested carbon and stainless steel elbows and tees well into the plastic range with harmonic excitation. No failure or structural instability was observed in any of these tests.

These dynamic tests on piping systems indicate that typical piping systems can withstand extreme seismic loading conditions without plastic collapse. Therefore, it is justifiable to develop functionality criteria which allows reasonable deformation of a piping system but still ensures that a safe shutdown can be achieved following a DBE.

Justification of the functionality criteria for piping subjected to a DBE is therefore provided by:

- Inherent conservatisms in standard piping system properties and design techniques.

- Demonstrated functionality of typical nuclear plant piping systems subjected to seismic events and high-excitation dynamic testing.

- Extremely low probability of occurrence of a DBE with the plant in the present design condition.

Final justification of the functionality criteria limits for SONGS-1 is achieved through a nonlinear analysis program on representative systems from the plant. This provides program measurable evidence of the adequacy of the criteria. The analysis program approach is outlined in the following section. It is noted that a similar approach was used to successfully license the 2.0 S_y stress limit for seismic response of piping at Commonwealth Edison's Dresden and Quad Cities plants as part of their IE Bulletin 79-14 program [13].

3.0 ANALYSIS PROGRAM APPROACH

The purpose of the nonlinear analysis program is to show that typical piping systems at SONGS-1 remain functional at elastically-calculated functionality criteria stress limits. The load combination considered in the criteria is Gravity + Pressure + DBE. Thermal expansion was not considered as part of the criteria.

Two representative piping systems were selected for the functionality study. Numerous piping systems were reviewed to choose these two systems. It was desirable to choose systems typical of most of the piping at SONGS-1, and to provide a variety of material and component parameters.

Elastic analyses were then performed on the two systems. Gravity, pressure, eigenvalue, and seismic analyses (both response spectrum and time history methods) were performed. These elastic analyses provided the following information:

- Gravity, eigenvalue, and time history analysis results were used to provide correlation of results with the nonlinear analyses. This insured proper development and accuracy of the nonlinear analysis models.
- Gravity and pressure analyses were performed to assess the magnitude of those stresses compared to the total elastic stress levels required for the functionality study. The gravity and pressure stresses were negligible and were excluded in the nonlinear functionality analyses. This assumption is discussed in detail in the analysis section.
- Response spectrum analyses were performed to identify the critical direction of seismic input motion for the nonlinear analyses. The results of the reponse spectrum analyses were also used to determine the scale factor on the input motion needed to produce maximum stresses at the required functionality limits.

Seismic time histories were developed for each piping system which enveloped the required SONGS-1 design response spectra. The base motion used was 10 seconds of an El Centro 1940 acceleration time history record. An iterative process was used to adjust the response across the frequency range of interest, such that the final response spectra generated from the time histories closely matched the SONGS-1 design spectra.

Nonlinear analysis models were developed such that the components which comprise the model were correlated to experimental behavior. Static loading analyses were performed on those components, and their material properties were adjusted such that their global response closely matched that of a similar experimental component. This modeling technique provided increased accuracy in the piping responses predicted by the nonlinear analysis.

After extensive modeling checks were performed to verify the accuracy of the nonlinear analysis models, direct time integration analyses were performed with the scaled design time histories. Response time histories of the critical components were obtained, and maximum moments and strains were reviewed to assure the functionality of the systems. Finally, the results were used to make conclusions regarding the adequacy of the SONGS-1 functionality criteria for DBE loading.

4.0 CHOICE OF REPRESENTATIVE PIPING SYSTEMS

Approximately twenty piping systems at SONGS-1 were selected for initial review. These twenty systems were those having stresses reported in excess of the SEP allowable stress level in the as-built configuration. The April 1982 submittal by SCE to the NRC [14] was used to obtain the stress levels to select these twenty systems. Available support installation status information was then reviewed to determine the number of supports requiring installation or modification for the final design and the number of those supports that had been installed at the time. The selection of systems was based on the following considerations:

- Location and magnitude of overstress
- Support requirements for final design condition
- System function
- Material properties
- System geometry, variety of components

Based upon the review, two piping systems were chosen for the functionality study. These systems are designated problem numbers AC-19 and MW-01. AC-19 consists of 2-1/2-inch and 1-inch lines which carry water to cool the primary shield wall. All AC-19 piping is carbon steel A-53 Type B. MW-01 consists of 8-inch and 6-inch lines anchored at recirculation heat exchangers. All MW-01 piping is A312 Type 304 stainless steel. Figures 4.1 and 4.2 show the portions of AC-19 and MW-01 piping included in the analyses.

Problem AC-19 was selected because it requires the completion of much of the support work to meet SEP criteria. The final design condition requires the addition of three lateral supports and one vertical support. The piping is small diameter carbon steel piping with relatively low diameter-to-thickness ratios. Therefore, the components are fairly stiff, with low stress intensification factors. The geometry of the system is complex, with a great variety of different components such as 2-1/2-inch long-radius elbows, 5-D bends, and 2-1/2x1 tees. A relatively even distribution of high stresses was anticipated for these lines.

Problem MW-01 was chosen to complement the system parameters investigated in the AC-19 analysis. MW-01 piping is made of stainless steel with a high diameter-to-thickness ratio. The 8-inch piping is Schedule 10S; therefore, the components are flexible and have high stress intensification factors. The system also has a variety of different component types. High stresses were expected at a few local areas.

The two problems selected from the variety of systems at SONGS-1 provide a good representation of the various piping component, material, and system types present in the plant. Both carbon and stainless steel materials are represented, as well as piping components of different size and flexibility. The systems both have typical run configurations with a mix of various component types. Although the seismic stress levels in the systems were not at the functionality stress limits, the input motions were increased to obtain the desired maximum elastic stress.

5.0 PIPING SYSTEM FUNCTIONALITY ANALYSIS

This section describes the analysis methods used to demonstrate component and system functionality at the maximum elastic stress limits specified by the functionality criteria. Preliminary elastic analyses are first discussed. Input time history generation is described. Nonlinear analysis methods, assumptions, and results are then presented. Overall conclusions are discussed in Section 6.0.

5.1 Elastic Analysis

Mathematical models of each piping system were first developed. These models include standard ASME flexibility factors and stress intensification factors for the components. Material properties were obtained from the ASME Code Appendix I [15] for the design temperature of each system.

All elastic analyses were performed with the EDS computer program SUPERPIPE [16]. Gravity analyses for each system were first performed. Low stresses were observed, as these systems are well supported in the vertical direction. The functionality study considers seismic loading in one horizontal direction, and although the load combination Gravity + Pressure + Seismic is addressed by the study, gravity stresses were omitted for the following reasons:

- Gravity stresses represented only 5% of the total stress in critical components when the maximum system stresses were increased to 2.0 S_y for carbon steel or 2.2 S_y for stainless steel.

- Since the earthquake load is scaled such that the maximum stresses equal the functionality stress limit, omitting gravity stress causes this scale factor to be greater.

- During horizontally-applied seismic motion, piping components are stressed in different locations around the pipe circumference than when gravity loading is applied. By omitting the gravity loading, the effects of the seismic loading are maximized, producing conservative strain data in the nonlinear analysis.

Pressure stresses were also calculated and found to be insignificant. Pressure loading was not included in the functionality analysis because low to moderate levels of pressurization have a beneficial effect on piping response, in that it stiffens the piping system and increases the bending resistance of the components. By neglecting pressure effects, strains are slightly overpredicted. Unpressurized piping is also more susceptible to ovalization of its cross-section and reduction of flow area.

Eigenvalue analyses were performed to determine the fundamental frequencies of system response. Tables 5.1 and 5.2 list the modes and frequencies below 33 Hz for AC-19 and MW-01. Seismic response spectrum analyses were then performed using the SONGS-1 design spectra for each global axis direction to determine the critical direction of seismic input for the nonlinear analyses.

For problem AC-19, the global X-direction response spectrum analysis produced an even distribution of high stresses in many components. Thus, when the seismic motion in the X-direction is scaled such that the maximum stress level is 2.0 S_y, extensive yielding of the system should result. For problem MW-01, results for the X-direction response spectrum analysis also predicted overstress in more than one location.

Therefore, the seismic load was applied in the global X-direction for the nonlinear analysis of both lines.

Tables 5.3 and 5.4 summarize the stresses in critical components from the X-direction response spectrum analyses for problems AC-19 and MW-01, respectively. These tables also show the final factored stress levels for the systems. These factored stresses and their related bending moments, system support loads, and accelerations are used to compare results with the nonlinear analyses later in this section. The X-direction seismic time history and response spectrum scale factors are 2.68 for AC-19 and 7.85 for MW-01. These scale factors were determined such that stresses in the critical elbow elements were at the functionality limits. Thus, stresses in a few other components exceeded the functionality limits.

From the results of the elastic analyses, it was observed that there are areas of low stress in the piping systems. To minimize the cost of the nonlinear analyses, it was desirable to eliminate as many piping degrees of freedom as possible. Runs of pipe were removed from both systems in areas remote from the critically stressed piping. The removed piping was modeled in the reduced system by specifying lumped masses and stiffnesses at the cutoff points. The reduced models are shown in Figures 5.1 and 5.2 with the node numbering scheme used in the nonlinear analysis. All elastic analyses previously discussed were performed on the reduced models with excellent correlation. Critical frequencies were maintained in the reduced models, which assured an accurate and cost-efficient model for use in the nonlinear analyses.

5.2 Time History Generation

To perform nonlinear seismic analyses of the two piping systems, it was necessary to obtain input time histories to meet the SONGS-1 seismic design requirements. An iterative process was used to adjust the response at different structural frequencies such that the SONGS-1 design response spectra were properly enveloped by the response spectrum generated from the time histories.

A ten-second record from the El Centro 1940 earthquake motion was used as the base motion. One time history was generated for each analysis problem. The SONGS-1 design response spectra used to match the time history response were envelopes of the two horizontal design spectra (N-S and E-W) which were used for the SEP analyses of AC-19 and MW-01.

The Fourier components of the El Centro motion were scaled such that the final time history produced an acceleration response spectrum close to that used in the SONGS-1 design. The EDS computer programs FREAK [17] and RESPEC [18] were used in the iterative process to obtain the design time history. When the time history-generated response spectra and the design response spectra were matched, the resulting displacement time histories were baseline-corrected to remove the drift in the motion. The final acceleration time histories are plotted in Figures 5.3 and 5.4. The resulting response spectra are compared to the original design spectra in Figures 5.5 and 5.6.

To verify the adequacy of the generated time histories, a linear time history analysis was performed on problem AC-19 and compared with the results of the response spectrum analysis. The time history analysis produced stresses slightly higher than those calculated in the response spectrum analyses. This step showed that the time history generated conservatively predicted the system response.

5.3 Nonlinear Piping Component Correlation

The computer program ANSYS [19] was used to perform the nonlinear analyses. The models were composed of elastic and plastic straight pipe elements and plastic elbow elements. To maintain functionality, the elbow, tee, and straight pipe elements must not distort excessively during the DBE event. To assure that the system models accurately predict the piping behavior in the field, the ANSYS elbow elements were correlated with measured response in experimental studies. Also, since ANSYS does not have a specific tee or branch connection element, an equivalent component was developed by adjusting properties of the four straight pipe elements used to model the branch connection. These tees were also correlated with experimental data.

In finite element analysis, certain geometric and material property relationships are idealized. In the ANSYS analyses, only a bilinear stress-strain relationship can be used for the non-proportional loading encountered in seismic analysis. This bilinear relationship is adjusted so the behavior of the elbow and tee elements closely matches the experimental results. In the element correlation task, it was found that it was not possible to obtain a good match for both the momentdeflection data and the moment-strain data for a particular element. In an elbow, this is attributed to additional ovalization modes not included in the ANSYS model. However, by matching the moment-deflection curves closely, the proper global response is assured. Additionally, by matching the moment-deflection curves, a conservative moment-strain relationship is produced. Thus the ANSYS-calculated strains can be considered an upper bound response of the component under the seismic load. Figures 5.7 and 5.8 show the moment-deflection and moment-strain curves for a carbon steel elbow.

To develop the ANSYS elbow models, the ORNL/NUREG-24 elbow study [20] was used. This study loaded 6-inch (nominal) commercial carbon and stainless steel elbows to produce predominately plastic response. One stress-strain curve for elbows was developed for each problem, since AC-19 is a carbon steel system and MW-01 is a stainless steel system. After these stress-strain relationships were determined, they were used directly to establish the elbow materials in each problem. The Karman flexibility factor was used to allow for changes in elbow size and cross-section.

To develop the ANSYS tee models, results of the study by Ellyin [21] were used. This study loaded tees of various run and branch sizes with in-plane and out-of-plane couples. Loading was applied to produce plastic distortion of the tees. To model the tee with ANSYS elements, standard pipe components were used for the run pipe. For AC-19 tees, the run pipe was predicted to remain elastic, and elastic pipe elements were used in the model. For MW-01, plastic pipe elements were used for the run pipe. Two plastic pipe elements were used for the branch pipe in each problem. The first was a relatively stiff element extending from the run pipe axis to the surface of the run pipe. The other element was relatively flexible. Deflections at the notch of the tee and at a point farther up the branch pipe were matched with experimental curves. Again, this produces an extremely conservative moment-strain relationship. Figures 5.9 and 5.10 show the moment-deflection and moment-strain relationships for the correlated carbon steel tee.

5.4 Nonlinear Analysis

The mathematical model for the nonlinear analysis was developed with elbow and tee components which closely match experimental behavior. Other straight-pipe components were modeled using the standard ANSYS pipe elements with ASME Code material properties at the design temperature. Damping for the DBE seismic event was taken to be 2 percent from the fundamental frequency of the system to 50 Hz. Alpha-beta damping using the current stiffness matrix was used.

Although the nonlinear analysis model was developed to closely predict actual behavior of the piping systems, they still contained inherent conservatisms.

- Actual material strengths are greater than Code-specified minimums. Code-specified minimums were used in the analysis.

- Component thicknesses are normally greater than nominal values. This increases the strength and moment-carrying capacity of the components.

- The boundary conditions specified for each problem are conservative.
- Actual system damping is higher than code-specified values. Two percent damping is used.
- Strain rate effects which enhance yield strength are conservatively neglected.
- Pressure effects increase collapse moments of components. These effects were conservatively neglected in the analysis.

The nonlinear analysis models created with ANSYS were verified by comparison with SUPERPIPE linear analyses. The previously created elastic SUPERPIPE models were modified to have the same material properties and flexibilities as the nonlinear model. Gravity, eigenvalue, and seismic time history analyses were performed. Both non-linear ANSYS models showed excellent correlation with the linear SUPER-PIPE models. The linear time history analyses were used to predict the time that each system would begin to yield and the time when system response would be maximized.

The predicted time of maximum response using the linear analyses gives an upper bound limit to time of significant response in the nonlinear model. Because of yielding and increased damping and energy absorption in the nonlinear systems, actual maximum response occurs earlier than in the elastic system. This was observed in the analyses of both AC-19 and MW-01. Thus the nonlinear analyses were not carried out to the end of the seismic time history. Instead, analyses were performed to a time just beyond the time of maximum reponse predicted by the elastic analyses.

5.4.1 AC-19 Nonlinear Analysis

The linear time history analysis of AC-19 predicted that first yielding would occur at 2.0 seconds at Elbow 3 and that maximum response would occur at 5.5 seconds. In the nonlinear analysis, first yield occurred in Elbow 3 (Refer to Figure 5.1 for designation of components) at about 2.24 seconds, slightly later than predicted. Soon after the elbow experiences yielding, the piping near the support at Node 16 yields, followed by Elbow 1. These components accumulate strain until strong motion starts at about 5 seconds into the earthquake. At this time, additional straight pipe segments yield (at Nodes 13, 14, etc.), and the maximum response is reached at 5.0 seconds. The analysis was run to 6.0 seconds, and it was seen that response was significantly decreased in that final second.

In the nonlinear analysis, moments in the critically stressed components were significantly reduced. Table 5.5 compares bending moments from the linear and nonlinear analyses for the more highly stressed components. Significant moments were reduced a minimum of 16%. The reduction in moment was mainly due to the "detuning" of the system as it yielded. The frequency of the system decreased as yielding occurred. For the AC-19 system, this caused the response to move away from the spectral peak, thus a decrease in response for the entire system was expected. The variation of moment reduction throughout the system is due to the redistribution of total load to the yielding components.

Strain data from the nonlinear analysis of AC-19 is reported in Table 5.6. Very low strains were calculated for the AC-19 piping system, with a maximum strain of 0.74 percent reported in Elbow 3. Maximum strain for a straight pipe section was 0.41 percent at Node 16.

The response of the piping in the area of the tees produced displacement-induced loads on the tees. The nonlinear analysis predicted artificially high moments in Tee 1 because of the stiff model used, which did not allow the required deflection of the piping. Strain energy methods were used to predict a maximum moment of 1.13 k-in, which is in the elastic range of behavior. Thus, the tees in AC-19 were not expected to yield under the applied loading conditions.

Functionality of the AC-19 piping system was assessed by comparing the maximum moments in each type of component (elbow, tee, and straight pipe) to ASME collapse moments and by comparing the calculated strains to measured strains in experimental studies. Table 5.7 compares theoretical collapse moments with calculated moments. All moments in the AC-19 system were below the collapse moments except the moments in Elbow 3 and the straight pipe adjacent to the support at Node 16. Moments at these two locations exceeded the collapse moment by 6 to 7 percent; however, due to the conservatism of the collapse moment determination and the low strain levels in the piping system, these moments were considered acceptable.

Strains for AC-19 were compared to the strains reported in the ORNL/NUREG-24 study used for the elbow correlations. For all carbon steel elbows tested in the study, the elbow strains calculated in the AC-19 analysis were in the range of measurement. Maximum ovality in the experiments was 6.5 percent. This ovality corresponds to a flow area reduction of about 0.3 percent, which is insignificant.

The piping of AC-19 was considered functional for the following reasons:

- The system and component models were conservatively developed as previously discussed, therefore response was overpredicted.

- The ASME collapse moments are extemely conservative. They correspond to a ductility of 2. Component test data show that piping is functional at moments in excess of the ASME collapse moment.

- The strains reported in the nonlinear analysis were conservative because of the material law used to match the global response. These conservatively calculated strains were well within the allowable strains reported in experimental studies and resulted in an insignificant flow area reduction.

The impact on support loads was also investigated. Table 5.8 compares support loads for the elastic and nonlinear analyses. Because of the frequency shift previously discussed, the loads on all supports were reduced in the nonlinear analysis.

5.4.2 MW-01 Nonlinear Analysis

The linear time history analysis of MW-01 predicted that first yielding would occur at 0.6 seconds at Elbow 4 and in the straight pipe at Node 19 (Refer to Figure 5.2 for designation of components). Maximum response was predicted to occur at 1.9 seconds. In the nonlinear analysis, very slight yielding of Elbow 4 occurred at 0.075 seconds; however, significant yielding of the system did not begin until 0.55 seconds into the seismic motion. At this time, Elbow 6 also yielded, followed by Elbows 2 and 5 in the next 0.2 seconds. The maximum moments were observed at 0.9 seconds, however maximum strains occurred at about 1.5 seconds. At this time, yielding was observed in Elbow 3 and the straight pipe at Node 19. The analysis was run to 4.0 seconds. Results showed no significant response after the maximum response at 1.5 seconds. Tee 1, which was very highly stressed in the linear response spectrum analysis due a high stress intensification factor, did not yield. This was due to the inherent flexibility of the branch connection which is not included in the ASME Code provisions for component modeling.

In the nonlinear analysis, critical moments in the highly stressed components were significantly reduced. Other moments of smaller magnitude increased due to load redistribution following yielding. Unlike AC-19, which has a very low fundamental frequency, MW-01 became "tuned" as the system yielded. The response moved closer to the spectral peak as yielding occurred. Table 5.9 compares the bending moments from the linear and non-linear analyses for the critical components. Figure 5.11 compares the linear and nonlinear moment response at Elbow 4 for the 4 seconds of applied seismic motion. This figure shows the reduction in moment and a slight frequency shift between analyses after the significant yielding occurs at 1.5 seconds.

Strain data from the nonlinear analysis is reported in Table 5.10. The highest strain in an elbow was 2.0 percent at Elbow 4. Maximum strain in a straight pipe was .07 percent at Node 19.

Functionality of the MW-01 piping system was assessed by comparing the maximum moments in each type of component experiencing plastic deformation (elbow and straight pipe) with ASME-defined and theoretical collapse moments and by comparing the calculated strains to strains reported in experimental studies. Figure 5.12 calculates the ASME collapse moment at twice the deflection at yield for the 8-inch elbow in the MW-01 system. The maximum resultant bending moment in Elbow 4 was 87 percent of the ASME collapse moment. The maximum resultant moment in the straight pipe at Node 19 was 75 percent of the theoretical collapse moment.

Strains for the elbows in the MW-01 system were compared with strains reported in the study of thin-walled elbows by Imazu, et al. [22]. In this study, for elbow strains of 2.0 percent, flow area reduction of 5 percent was reported, which is not significant.

The piping of MW-01 was considered functional for the following reasons:

- The system and component models were conservative as previously discussed, therefore response was overpredicted.

- All calculated moments were well below the theoretical collapse moments, which allow a ductility limit of 2.

- Strains in the critical elbow were conservatively calculated, yet resulted in a predicted flow are reduction of only 5 percent. This flow area reduction was considered to be insignificant.

The impact of nonlinear piping on support loads and accelerations was also investigated. Table 5.11 compares the support loads calculated for the linear and nonlinear analyses. Table 5.12 compares accelerations for selected nodes for the two analyses. Because of the frequency shift of the system and nonlinear load redistibution previously discussed, some suport loads and accelerations increased while others decreased.

6.0 CONCLUSIONS

The nonlinear analysis program supports the functionality criteria and shows that typical SONGS-1 piping systems remain functional at elastic stress levels of 2.0 S_y for carbon steel and 2.2 S_y for stainless steel. Although the nonlinear analyses made conservative assumptions in modeling and load definition, moment and strain levels in both systems were within experimentally verified functionality limits.

Critical moments in both systems were significantly reduced in the nonlinear analyses. AC-19 was a "detuned" system, such that the response was reduced after yielding occurred. For AC-19, all moments and support loads were reduced. This suggests that for lightly supported, flexible systems, functionality criteria which allow component yielding provide a rational method of evaluation. MW-01 was a "tuned" system, such that the reponse was increased after yielding occurred. Despite the increase in response, the yielding allowed a redistibution of load in the system such that functionality was maintained. This shows that for both "detuned" and "tuned" systems, redistibution of load following yielding provides the required load reduction to insure system functionality.

Therefore, the elastic piping stress limits of 2.0 S_y for carbon steel and 2.2 S_y for stainless steel specified in the SONGS-1 functionality criteria provide assurance that the piping systems are capable of withstanding DBE loads without loss of function. These criteria allow local yielding in components such that load redistribution reduces maximum moments and stresses, yet provides limits on the extent of yielding such that functionality of the system is maintained.

7.0 REFERENCES

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TABLE 5.1

AC-19 SYSTEM FREQUENCIES

Mode No.	Predominant Direction	Frequency, Hz	Period, Sec
1	X	2.0	0.513
2	X	3.7	0.267
3	Y	6.8	0.148
4	X	7.2	0.139
5	Y	9.0	0.111
6	Z	11.1	0.090
7	X	13.9	0.072
8	X	18.7	0.053
9	X	21.6	0.046
10	Y	22.9	0.044
11	X	25.8	0.039
12	Z	29.2	0.034
13	Y	30.5	0.033
14	Y	31.9	0.031

TABLE 5.2

MW-01 SYSTEM FREQUENCIES

Mode No.	Predominant Direction	Frequency, Hz	Period, Sec
1	X	7.7	.129
2	Z	9.4	.106
3	Y	12.9	.078
4	X	14.7	.068
5	Z	16.5	.061
6	Y	23.8	.042
7	Z	25.5	.039
8	X	27.0	.037
9	Z	27.5	.036

TABLE 5.3

AC-19 ELASTIC STRESS RESULTS

	Elastic Stress(2), ksi	
Location(1)	Design Spectrum	2.68 x Design Spectrum
Elbow 1	18.6	49.7
Elbow 2	14.8	39.5
Elbow 3	25.9	69.4 (2.0S_y (3))
Pipe @ Node 11	15.2	40.7
Pipe @ Node 12	12.8	34.1
Pipe @ Node 13	20.0	53.5
Pipe @ Node 14	20.1	53.7
Pipe @ Node 16	28.9	77.4 (2.2 S_y (3))
Tee 1	32.3	86.6 (2.5 S_y (3))
Tee 2	16.3	43.5

NOTES: (1) From model in Figure 5.1

(2) Elastic Stress = 0.75iM/Z, 0.75i $\geq$ 1.0
where i = ASME Class 2/3 Stress Intensification Factor
M = Resultant of two bending and torsional moments
Z = pipe section modulus

(3) For A-53 B Carbon Steel, S_y = 34.7 ksi at 110°F

TABLE 5.4

MW-01 ELASTIC STRESS RESULTS

	Elastic Stress(2),ksi	
Location(1)	Design Spectrum	7.85 x Design Spectrum
Elbow 2	2.75	21.6
Elbow 3	2.94	23.1
Tee 1	11.57	90.8 (3.6 S_y (3))
Elbow 4	7.01	55.0 (2.2 S_y (3))
Elbow 5	4.05	31.8
Elbow 6	3.71	29.1

NOTES: (1) From model in Figure 5.2

(2) Elastic Stress = 0.75i M/Z, 0.75i $\geq$ 1.0
where i = ASME Class 2/3 Stress Intensification Factor
M = Resultant of two bending and torsional moments
Z = Pipe Section Modulus

(3) For A312 TP304 stainless steel, S_y = 25.0 ksi at 200°F

Table 5-5

AC-19 LINEAR VS. NONLINEAR ANALYSIS RESULTS - MOMENTS

Location (See Figure 5.1)		Bending Moment, k-in Linear	Bending Moment, k-in Nonlinear	Percent Change
Elbow 1	In-Plane	37.9	18.5	-51
Elbow 2	In-Plane	33.8	25.8	-24
Elbow 3	In-Plane	69.4	53.2	-23
Pipe @ Node 14	About Vertical Axis	54.7	45.7	-16
Pipe @ Node 16	About Vertical Axis	82.4	53.9	-35
[illegible] 1 (1)	Out-of-Plane	12.0	7.0 (2)	-42

NOTES: (1) These moments are reported at the centroid of the tee element. Actual moments in the tee are somewhat lower.

(2) This moment is an upper-bound moment based on a stiff tee model. Actual moment is lower.

Table 5.6

AC-19 NONLINEAR ANALYSIS RESULTS - STRAINS

Location (See Figure 5.1)	Linear Analysis Stress, ksi	Nonlinear Analysis Maximum Strain, Percent
Elbow 1 @ Node 2	49.7	0.49
Elbow 2 @ Node 7	39.6	Remained Elastic
Elbow 3 @ Node 8	69.4 (2.0 S_y (1))	0.74
Pipe @ Node 14	53.8	0.21
Pipe @ Node 16	77.5 (2.2 S_y (1))	0.41
Tee 1	86.6 (2.5 S_y (1))	Remained Elastic (2)

Notes: (1) S_y = 34.7 ksi
(2) See text for discussion

Table 5.7

AC-19 NONLINEAR ANALYSIS RESULTS - COLLAPSE MOMENT COMPARISON

Location (See Figure 5.1)	Nonlinear Analysis Resultant Moment (1), k-in	Collapse Moment, k-in	Percent of Collapse Moment
Elbow 1 (Std. long radius)	25.5	26.4(2)	97
Elbow 3 (5-D bend)	53.6	50.4(2)	106
Pipe @ Node 16	53.9	50.4(2)	107
Tee 1	1.1(3)	2.8(4)	41

TES: (1) Resultant moment is taken as the SRSS of the two maximum bending moments.

(2) Theoretical collapse moment = $\frac{1}{6}\ (D_o^3 - D_i^3)\ \frac{S_y}{C_2}$

(3) Based on strain energy considerations for actual tee behavior. See text for discussion.

(4) ASME collapse moment at twice deflection at yield.

Table 5.8

AC-19 LINEAR VS. NONLINEAR ANALYSIS RESULTS - SUPPORT LOADS

Node(1)	Direction(1)	Support Load, k: Linear	Support Load, k: Nonlinear	Percent Change
11	Y	5.98	4.82	-19
14	Y	0.50	0.46	-10
15	Y	0.28	0.25	-11
16	X	1.88	1.31	-30
	Y	0.13	0.11	-15
20	Lateral	2.83	2.19	-23
	Y	0.23	0.20	-13
28	Y	0.84	0.61	-27
	Z	2.04	1.69	-17

Notes: (1) See Figure 5.1

Table 5-9

MW-01 LINEAR VS. NONLINEAR ANALYSIS RESULTS - MOMENTS

Location (See Figure 5.2)		Bending Moment, k-in Linear	Nonlinear	Percent Change
Elbow 3	In-Plane	57.7	69.1	+20
	Out-of-Plane	4.6	7.0	+52
Elbow 4 @ Node 14	In-Plane	118.3	96.2	-19
	Out-of-Plane	80.2	65.6	-18
Elbow 4 @ Node 16	In-Plane	30.3	28.1	-10
	Out-of-Plane	6.3	5.7	-7
Pipe @ Node 19	About Vertical Axis	306.9	198.1	-35
	About Horizontal Axis	53.1	39.0	-27
Tee 1	In-Plane	24.8	21.8	-12
	Out-of-Plane	5.5	6.6	+20

Table 5.10

MW-01 NONLINEAR ANALYSIS RESULTS - STRAINS

Location (See Figure 5.2)	Linear Analysis Stress, ksi	Nonlinear Analysis Maximum Strain, Percent
Elbow 3 @ Node 8	23.1	0.10
Elbow 4 @ Node 14	55.0 (2.2 S_y (1))	2.0
Elbow 4 @ Node 16	31.8	0.42
Pipe @ Node 19	38.9	0.07
Tee 1	90.8 (3.6 S_y (1))	Remained Elastic

Notes: (1) S_y = 25.0 ksi

Table 5.11

MW-01 LINEAR VS. NONLINEAR ANALYSIS RESULTS - SUPPORT LOADS

Node(1)	Direction(1)	Support Load, k Linear	Nonlinear	Percent Change
11	Y	2.00	3.37	+69
	Z	0.89	0.99	+11
19	X	11.30	8.31	-26
	Y	1.64	1.25	-24
20	X	4.64	2.59	-44
	Z	1.50	1.16	-23
24	Y	0.41	0.91	+125
	Z	2.09	2.47	+18
28	X	3.82	4.60	+20
	Z	1.38	1.40	+1

Notes: (1) See Figure 5.2

Table 5.12

MW-01 LINEAR VS. NONLINEAR ANALYSIS RESULTS - ACCELERATIONS

		Acceleration, g		
Node(1)	Direction(1)	Linear	Nonlinear	Percent Change
10	X	8.12	6.52	-20
	Y	0.05	0.09	+67
	Z	0.18	0.17	-1
12	X	8.12	6.51	-20
	Y	0.31	0.21	-31
	Z	0.96	0.53	-45
14	X	8.08	6.47	-20
	Y	1.62	1.73	+7
	Z	2.68	2.80	+5
21	X	5.21	5.71	+10
	Y	0.55	0.79	+42
	Z	1.74	1.72	-1

Notes: (1) See Figure 5.2

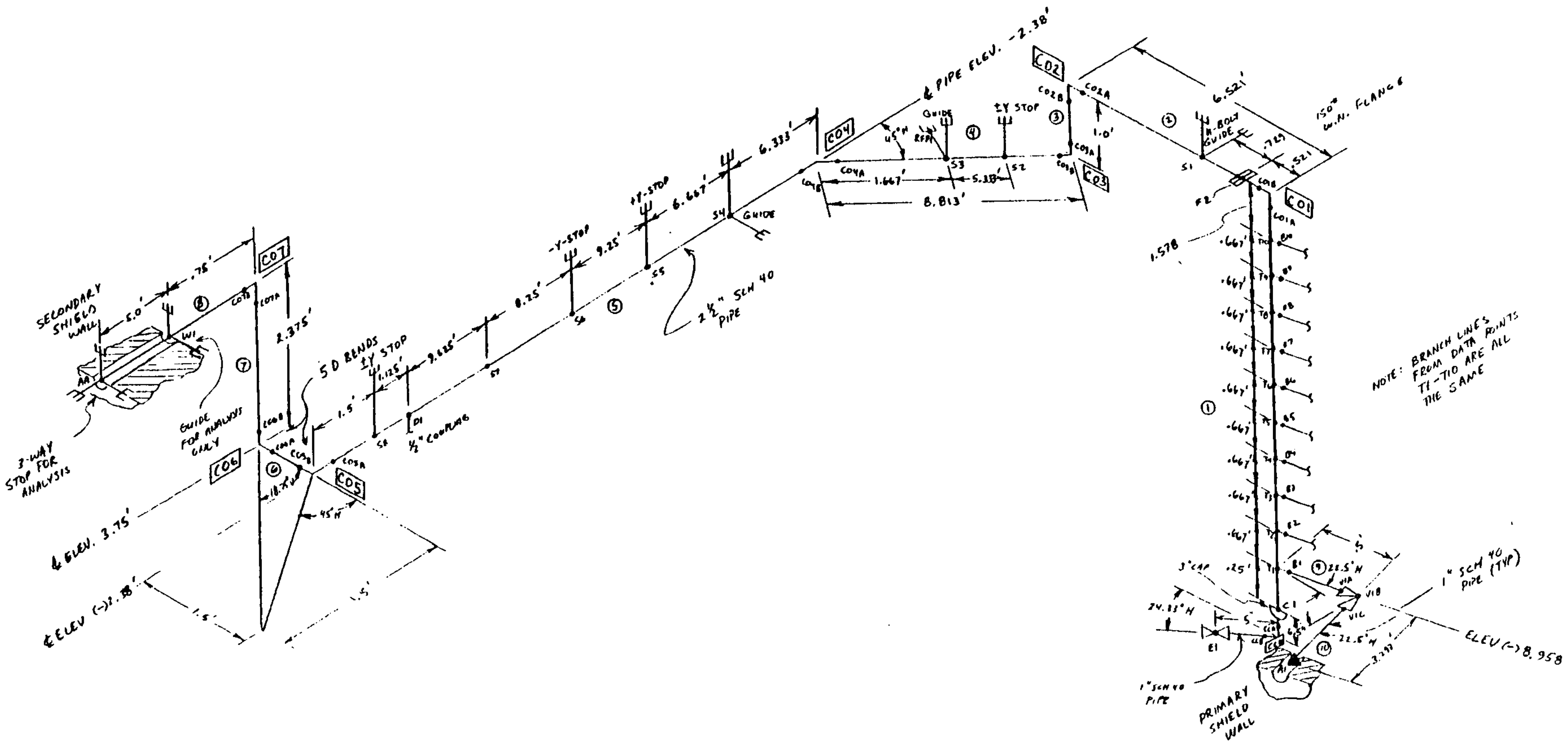

FIGURE 4.1 AC-19 Mathematical Model - Linear Analysis

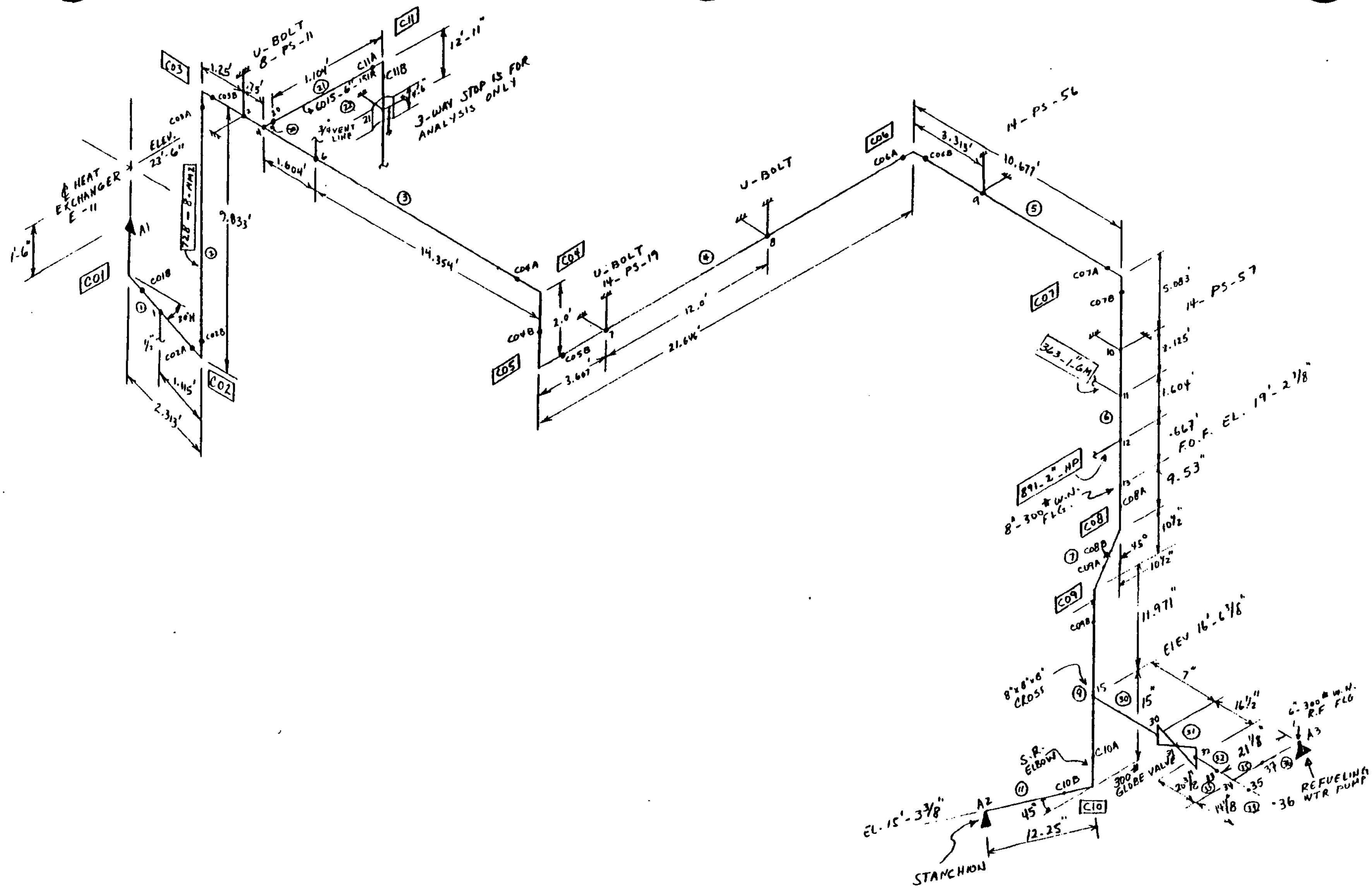

FIGURE 4.2 MW-01 Mathematical Model - Linear Analysis

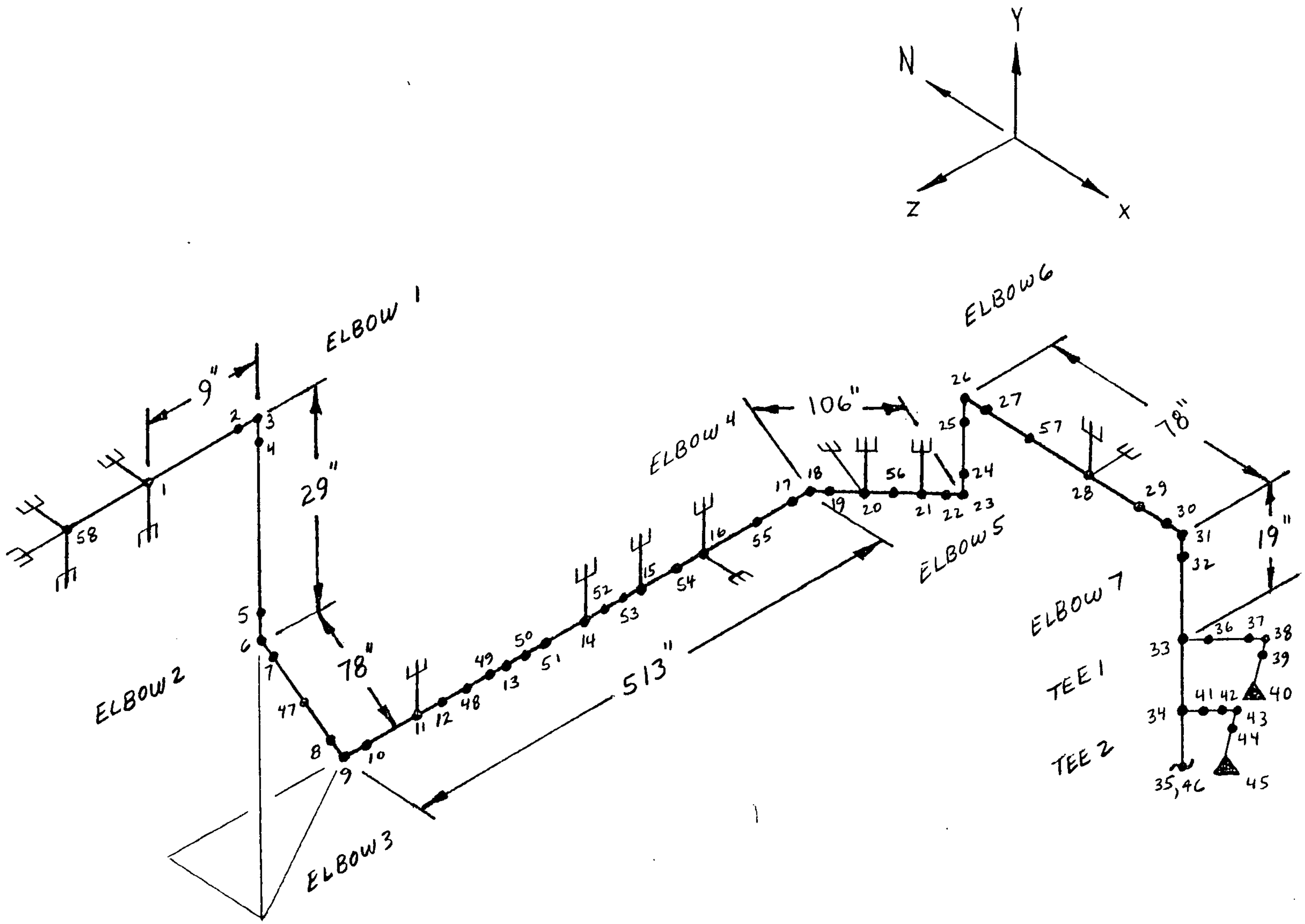

FIGURE 5.1 AC-19 Mathematical Model - Nonlinear Analysis

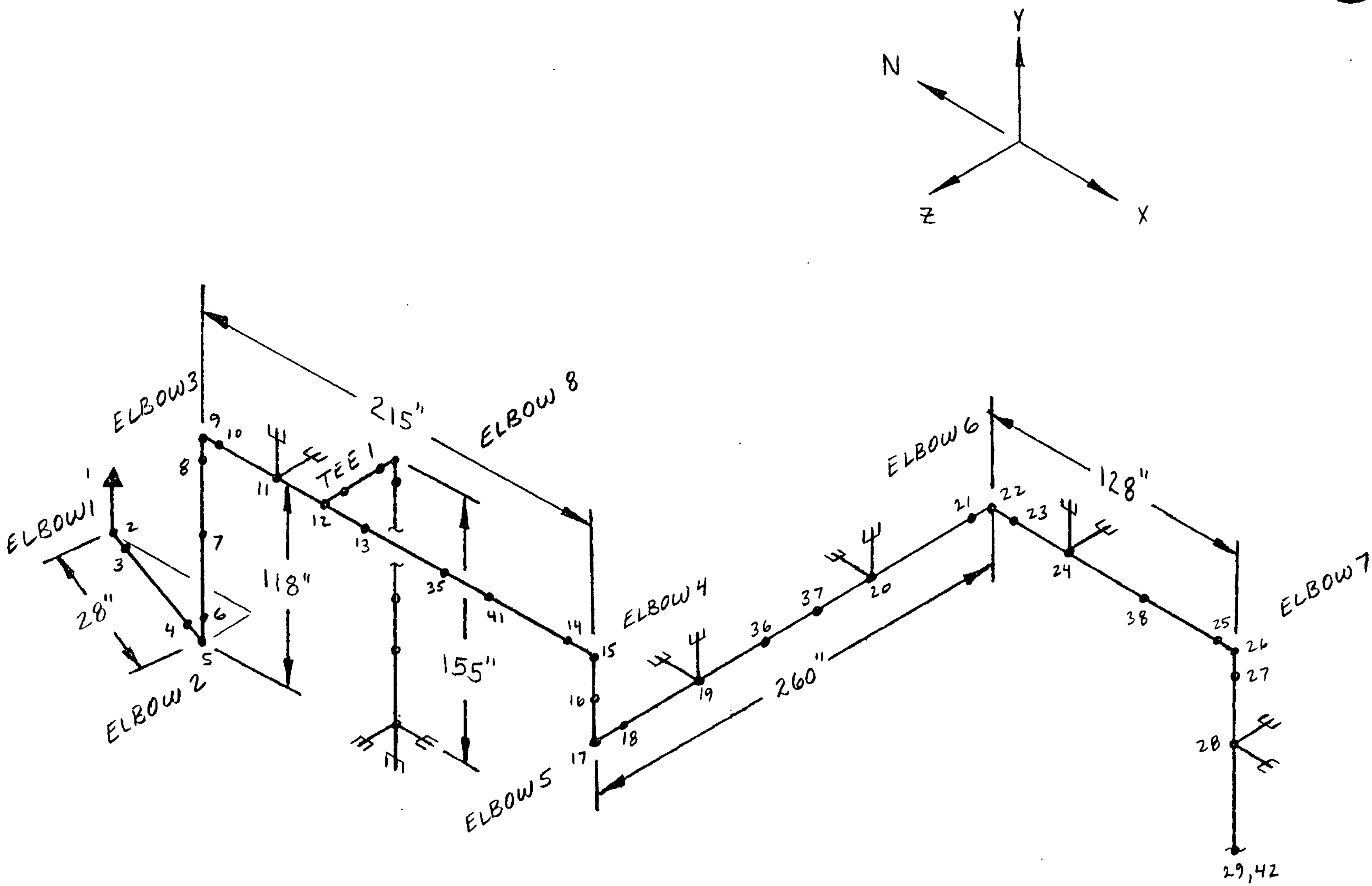

FIGURE 5.2 MW-01 Mathematical Model - Nonlinear Analysis

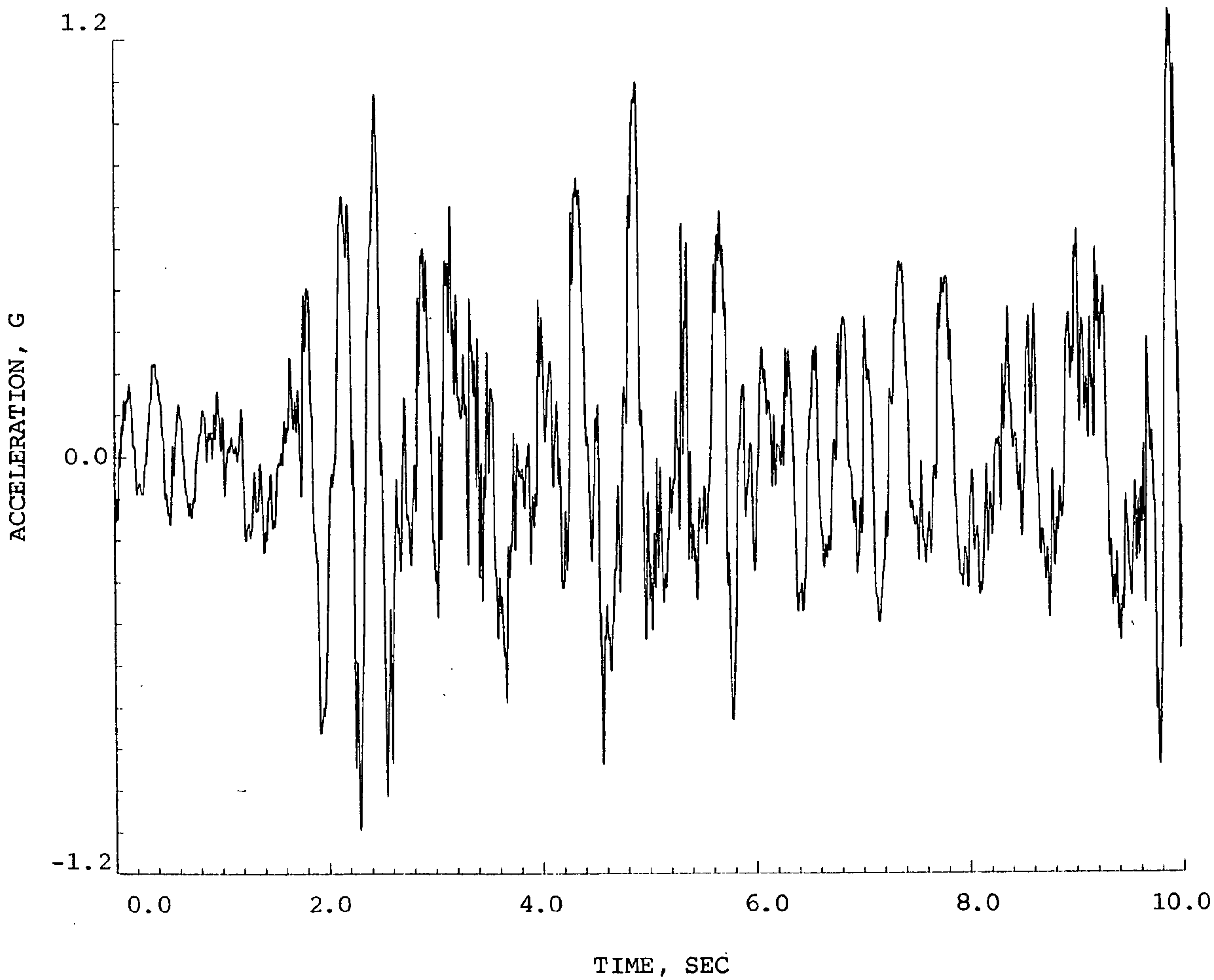

FIGURE 5.3 AC-19 Design Time History

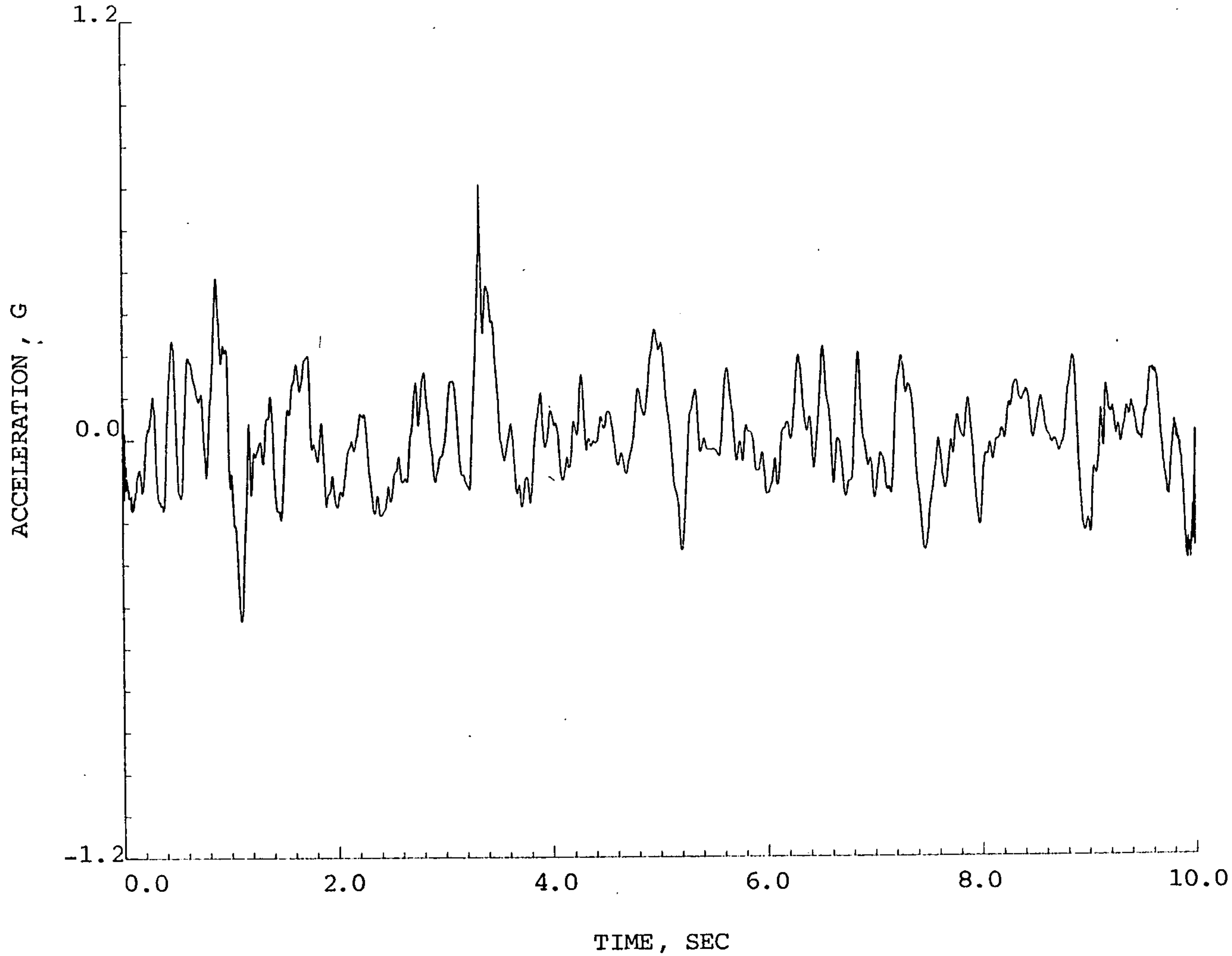

FIGURE 5.4 MW-01 Design Time History

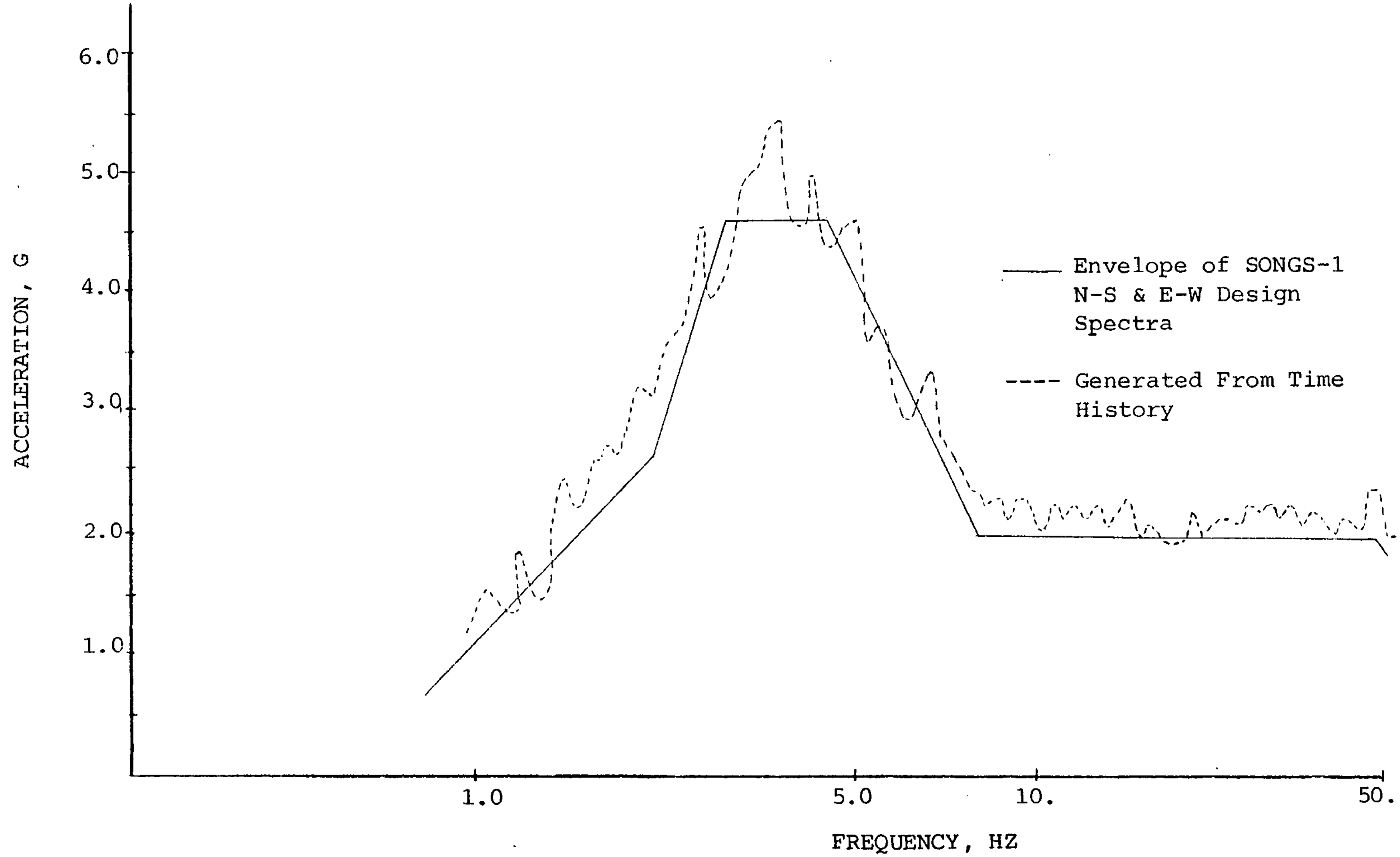

FIGURE 5.5 AC-19 Enveloped Response Spectra

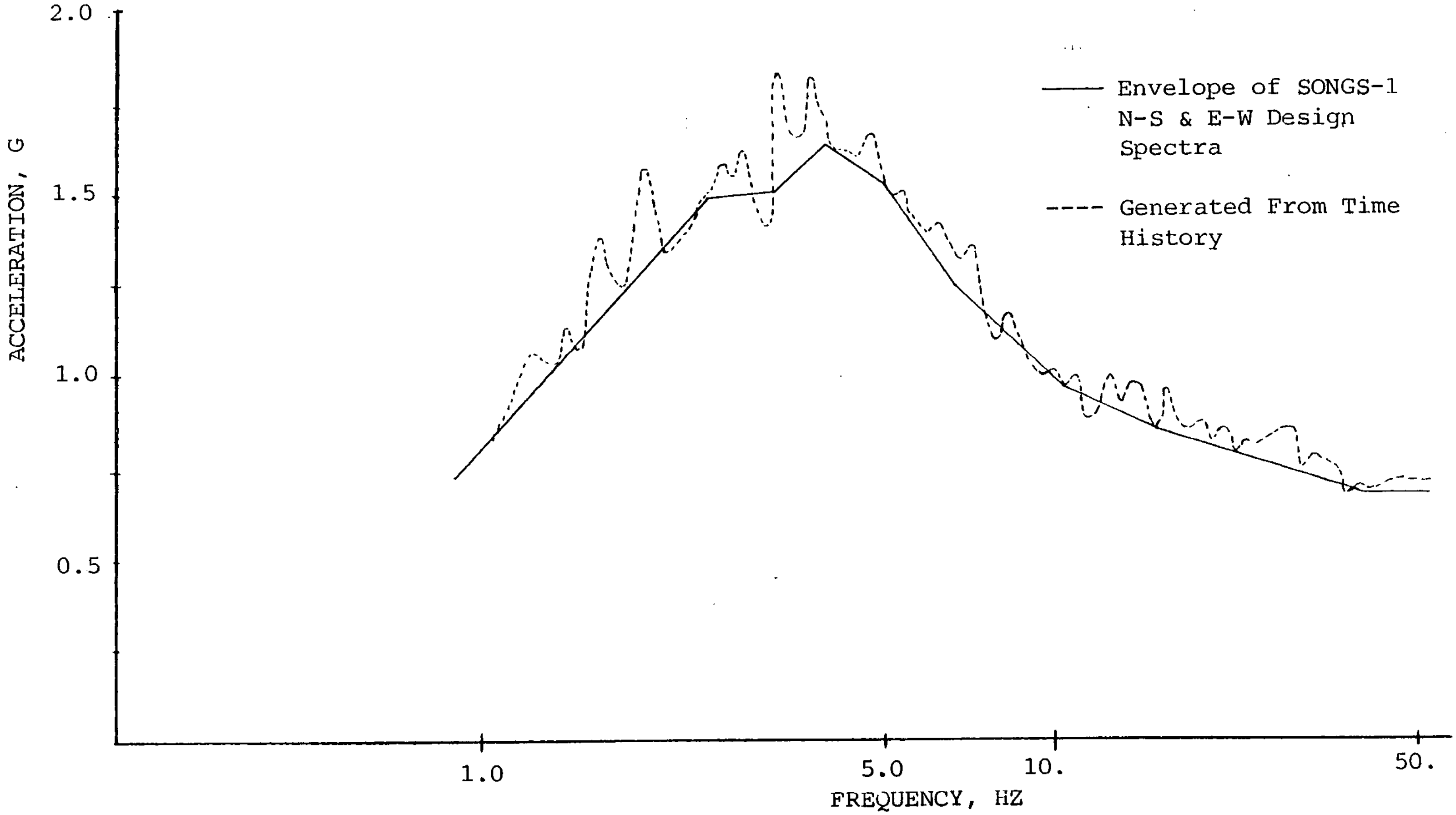

FIGURE 5.6 MW-01 Envelope Response Spectra

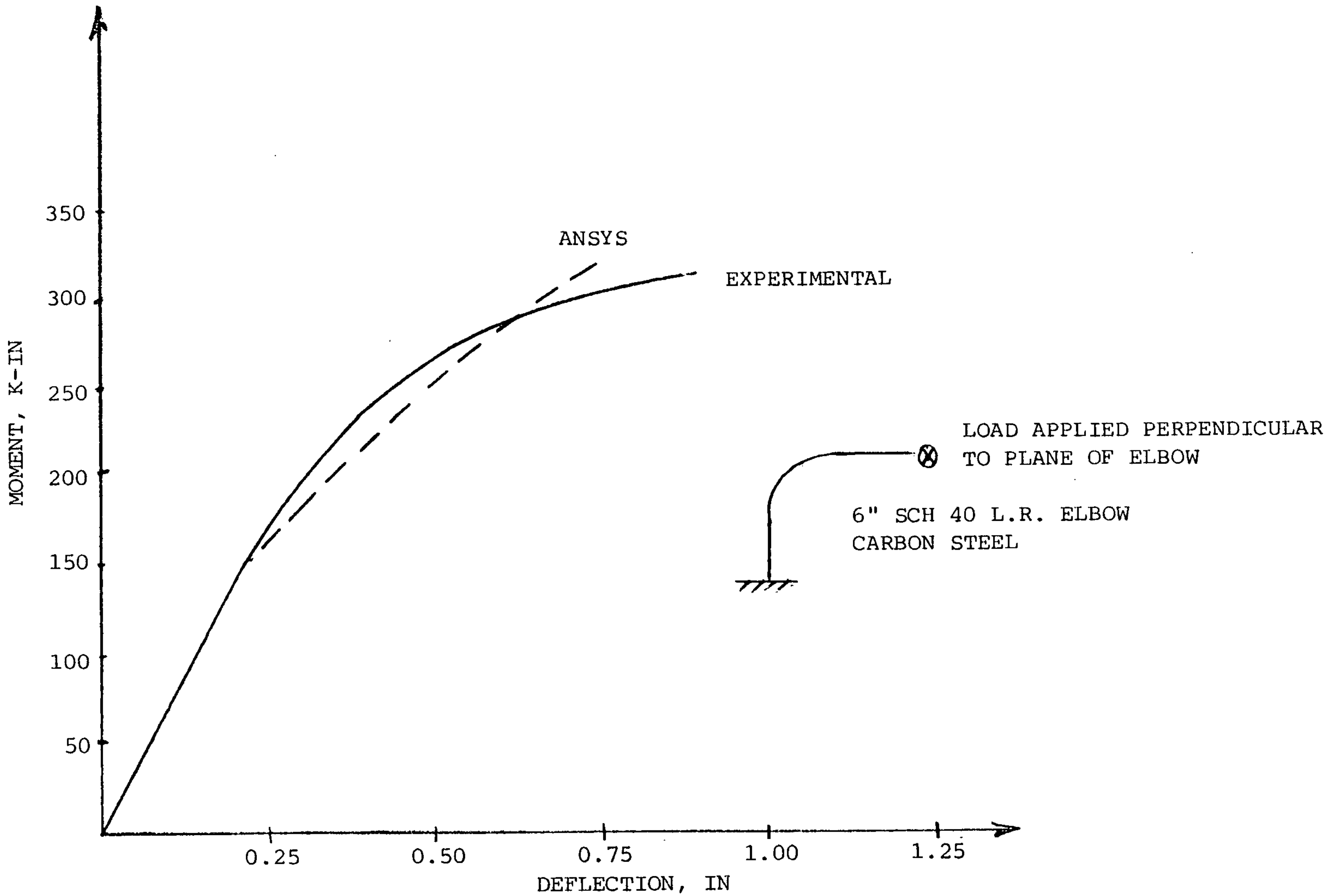

FIGURE 5.7 6-Inch Schedule 40 Carbon Steel Elbow Moment-Strain Curve

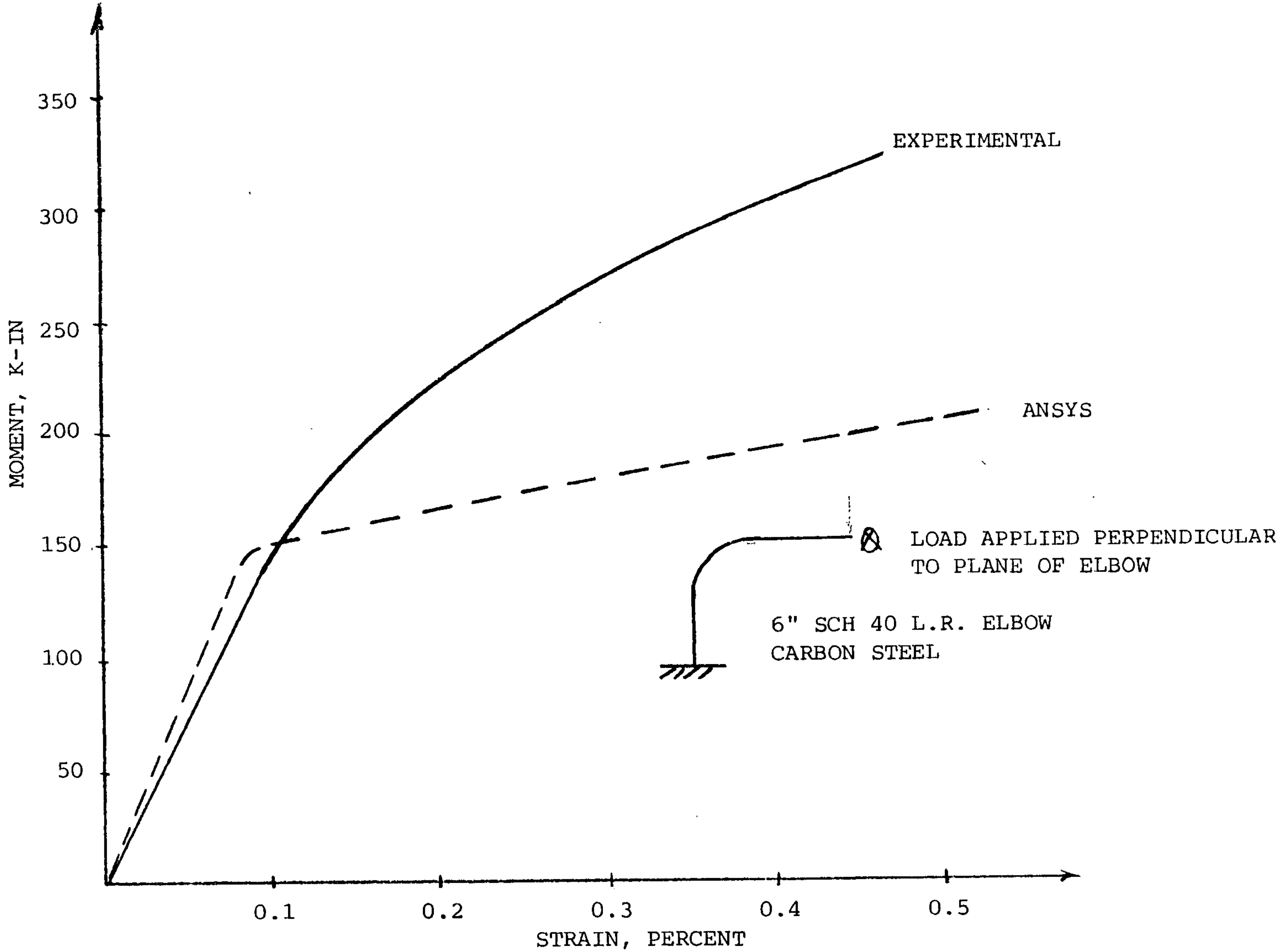

FIGURE 5.8 6-Inch Schedule 40 Carbon Steel Elbow Moment-Strain Curve

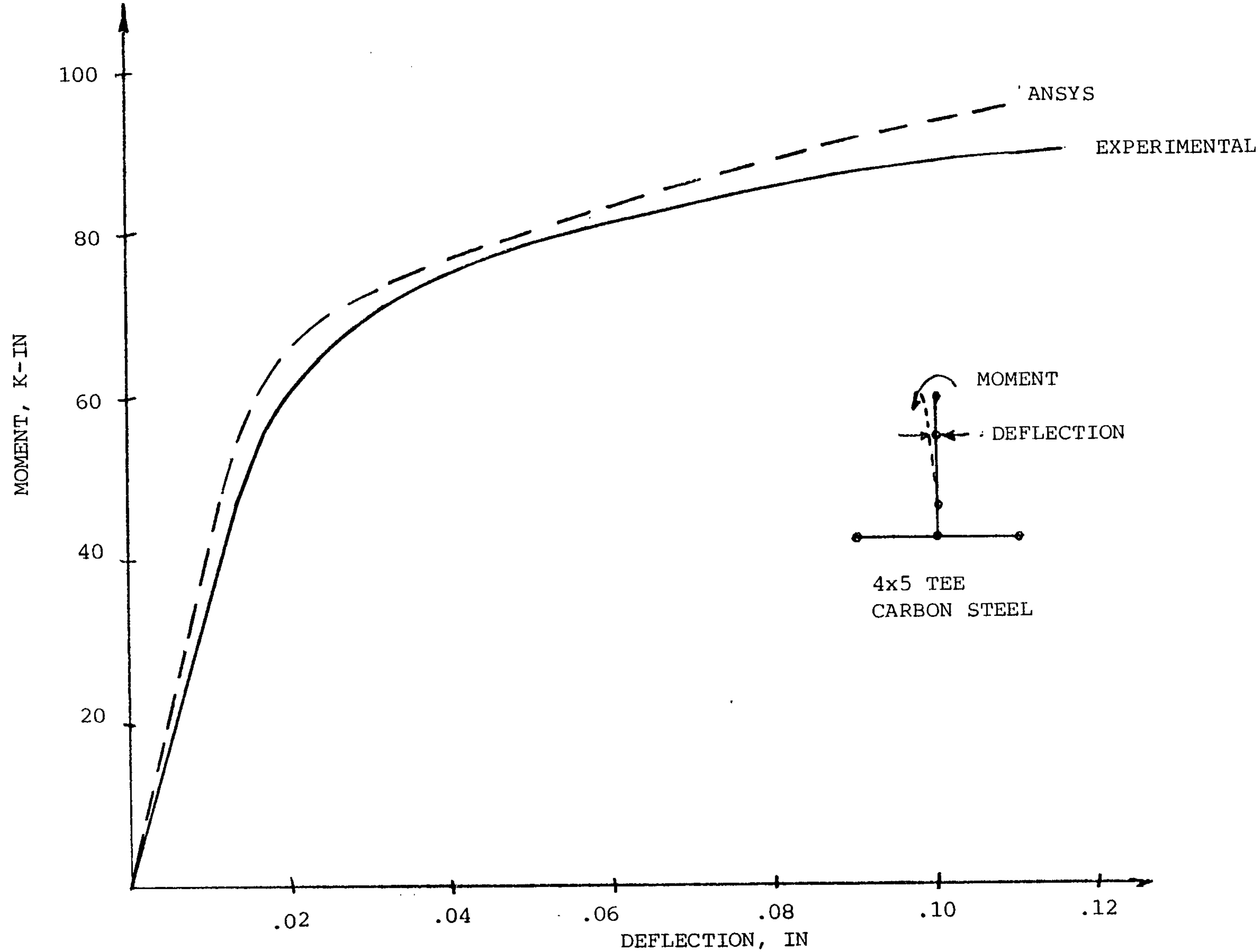

FIGURE 5.9 5x4 Tee Moment-Deflection Curve

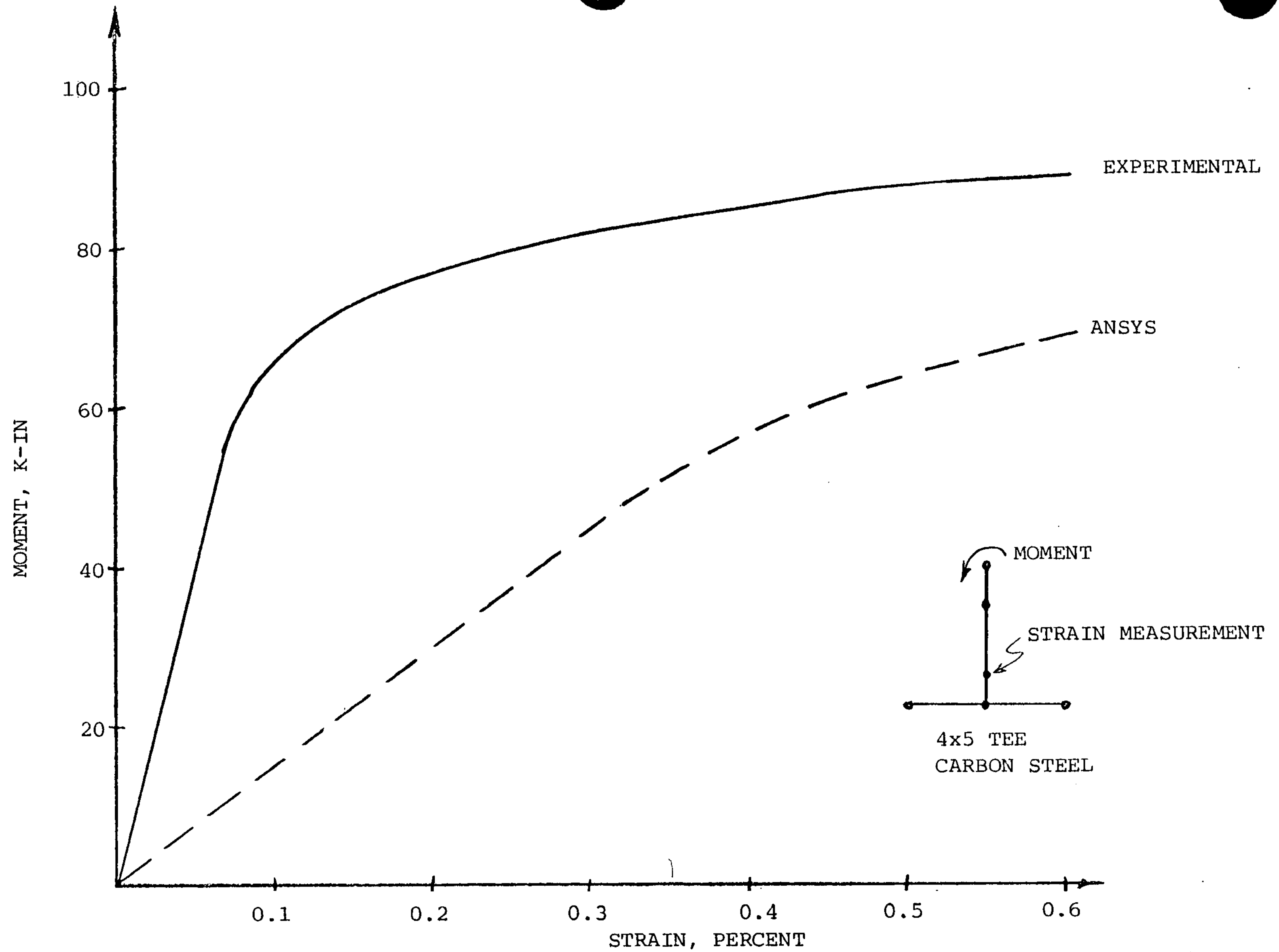

FIGURE 5.10 5x4 Tee Moment-Strain Curve

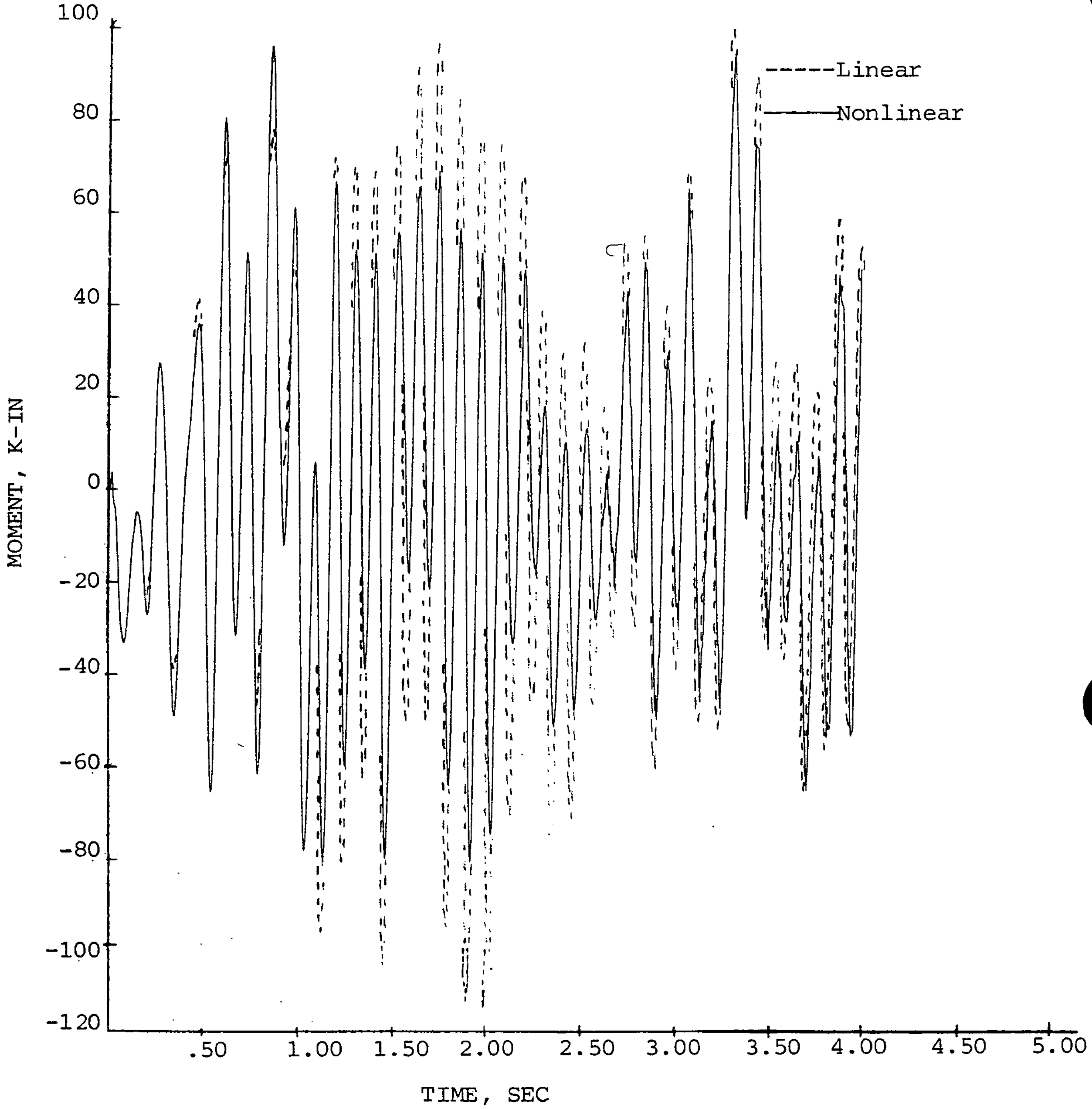

FIGURE 5.11 MW-01 Elbow 4 In-Plane Bending Moment Response History

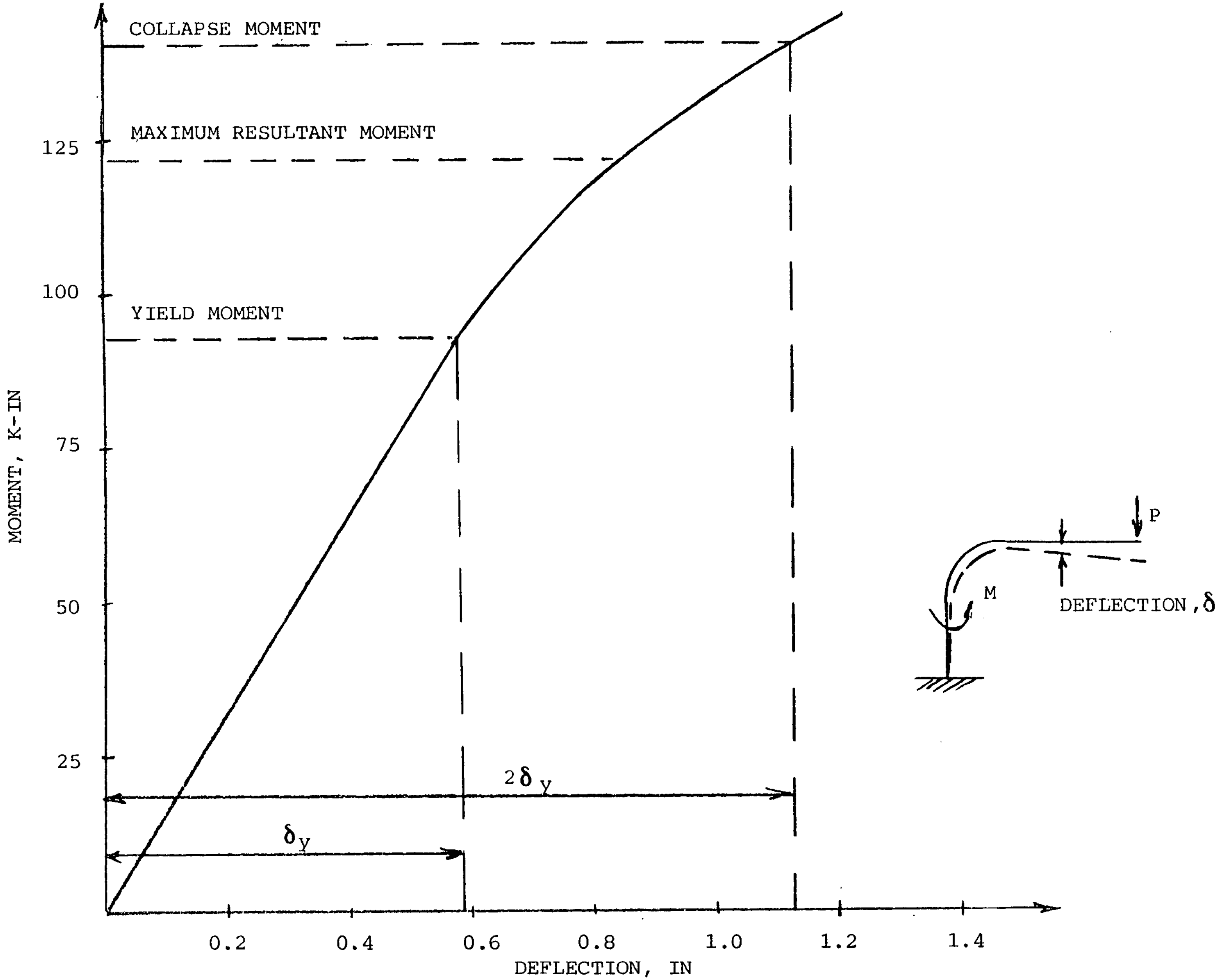

FIGURE 5.12 MW-01 8-Inch Elbow Collapse Moment

APPENDIX E

METHODOLOGY AND CRITERIA

FOR THE

QUALIFICATION OF SMALL BORE PIPING AND TUBING

December 23, 1983

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METHODOLOGY AND CRITERIA
FOR THE
QUALIFICATION OF SMALL BORE PIPING AND TUBING

1.0 Introduction

This document describes the methodology and criteria to be applied for the qualification of small bore piping as part of the program to return San Onofre Unit 1 to service. The intent of the approach outlined in the following sections is to take cognizance of the observed behavior of small piping systems in actual earthquakes and dynamic testing in developing a specific set of acceptance criteria applicable to SONGS 1.

The original design of piping systems at San Onofre Unit 1 was based on the 1955 version of the ANSI (formerly USAS) B31.1 Code for Power Piping. The fundamental basis of the 1955 version of the B31.1 Code is to develop a piping system that has a balance of flexibility and control. It is this concept of controlled flexibility that is in use today in the design of power plant piping. An inherent property of piping systems designed with controlled flexibility is the ability to absorb large amounts of energy such as is created by seismic ground motion.

Historically, piping systems designed similar to San Onofre Unit 1 have performed well when subjected to severe shaking from earthquakes of significant magnitudes. Several surveys have been made which document the satisfactory performance of welded carbon steel pipe. Two of the more authoritative works on this subject were published by Cloud (reference 1) and by Murray Nelson, et. al., (reference 2). Both of these studies concluded that for the particular earthquakes studied the performance of piping systems considerably exceeded the design basis.

In addition to the observed performance of piping systems in actual earthquakes, the performance of piping systems during dynamic tests has provided strong evidence of the substantial margins in the current design practice. It should be noted that these tests have not been limited to piping systems supported to the stringent requirements of current regulatory practice. In the case of the recent tests by ANCO engineers for KWU (reference 4) very flexible piping systems similar to San Onofre Unit 1 piping systems were subjected to seismic inputs that exceed the spectra for San Onofre Unit 1. These tests were generic and formed the basis for the acceptance without backfit to the "as-built" configuration of KWU small bore piping by the German regulatory authority of nuclear safety. In addition to the KWU tests, a number of other tests have been performed on small bore piping. These have generally supported the conclusion of excess capacity substantially beyond the design limits and even substantially beyond yield. The results of these tests are described in Section 3.0.

The proposed program for qualification of SONGS piping includes, in addition to the demonstration of design similarity to the KWU testing, the specific review of all SONGS 1 safe shutdown small bore piping. This review will apply criteria based upon good industry practice to ensure adequate lateral restraint, sufficient flexibility to provide for thermal growth, support for in-line concentrated masses, and adequate spacing of vertical supports to minimize dead weight and operating stresses. An approach to small bore piping that is similar to this is currently under consideration by a PVRC subcommittee. The criteria and walkdown procedure to satisfy the above conditions are described in Section 2.0.

2.0 Return-to-Service Criteria and Walkdown Program

To assure that the "as-is" supported configuration of the existing small bore piping and tubing at SONGS 1 (i.e., partially upgraded and partially original supports) meets the requirements implied in references 4 and 5, a field walkdown will be conducted to document the following:

1. Dead weight spans meet industry practice.

2. Valves with eccentric masses (and other in-line large concentrated masses) have supports adjacent to them.

3. Horizontal supports are placed at intervals approximately 3 times the dead weight spans.

4. U-bolt nuts and pipe clamp nuts are properly tightened and have lock nuts where appropriate.

3.0 Dynamic Testing of Small Bore Piping

Dynamic testing of nuclear related piping and equipment was first undertaken in the late 1960's at San Onofre by UCLA. These tests were performed with small shakers on the operating deck of the reactor building and were very low amplitude. The nature of the tests and the state of the art at the time were a limitation on the usefulness of the results. One of the early dynamic tests that generated significant data regarding the performance of major equipment and piping was performed by Westinghouse at Indian Point and reported by Bohm (reference 3).

Recently, a number of test programs intended to investigate the performance of typical nuclear piping systems have been undertaken. These include testing performed by EPRI, Hanford Engineering Development Laboratory, the Earthquake Engineering Research Center at UC Berkeley, and ANCO Engineers. There are similarities in all of the results reported. These may be described as follows:

(1) piping systems have withstand capabilities well beyond the limits of the piping system designs

(2) damping in piping systems tends to be higher than used in current design practice

(3) flexible piping systems have substantial withstand capability that is not predicted by linear analysis and even exceeds that predicted by nonlinear analysis.

A summary of some recent piping tests is described in the following paragraphs.

3.1 ANCO Engineers for KWU

ANCO Engineers has performed a number of small bore piping tests for KWU in the Federal Republic of Germany, EPRI, and the Bechtel Power Corporation. Of particular interest are the tests performed by ANCO to qualify small bore piping for KWU. These tests were used to generically qualify flexible small bore piping without the need to perform sophisticated computer analyses of this piping. The qualification of existing small bore piping in KWU nuclear power plants to withstand low frequency loading (SSE) and high frequency loading (aircraft impact) was successfully demonstrated by ANCO Engineers for Kraftwerk Union. A series of full-scale tests, using small bore piping systems typical of those installed in KWU nuclear power plants, were conducted on a shake table. These tests clearly showed that small bore piping is capable of surviving low and high frequency loads where large displacements, accelerations, and even plastic strains occurred.

3.1.1 Description of Inputs and Configurations for KWU Tests

A film of the testing described in the report was shown to the NRC staff December 14, 1983. In addition, data regarding the input spectra and test configurations are provided in this report and are extracted from references 4 and 5.

The test shake table consists of an open steel frame supported on four pivot areas. The linkages have spherical bearings on both ends and are set at 45 degrees from the horizontal. This constrains the motion of any point on the table to a plane perpendicular to the linkages. The pivot arms are oriented transversely, thus coupling transverse and vertical motion. With the pivot arms in this orientation, longitudinal, horizontal motion is independent of transverse and vertical motions. The pivot arms can also be oriented longitudinally so that transverse horizontal motion is independent of longitudinal and vertical motions. Both pivot arm orientations were used during the course of testing each configuration.

All test configurations were instrumented with accelerometers, strain gages, and displacement transducers. Some were coated with Tens-Lac brittle lacquer. In addition, test configurations were monitored using an 8mm movie camera, a video cassette recorder, photographs, and a contact indicating compound. All conditioned signals were filtered (eight-pole, low-pass, Butterworth) by STI-AA32 filter amplifiers to prevent aliasing during digitization. The cutoff frequencies were at 100 Hz and 42.6 Hz for aircraft impact and earthquake, respectively.

Shake table response spectra and peak acceleration, displacement, and stress tabulations were obtained with a minicomputer-based vibration analysis system. This ANCO developed system is named CVTAS (Computerized Vibration Test and Analysis System).

In order to obtain preliminary information on the dynamic characteristics of each different piping configuration, low-level tap tests were performed. These tests simply involved the use of a low-mass accelerometer and a spectrum analyzer. Excitation was applied either by tapping the pipe with a rubber mallet or pulling the pipe back by hand and releasing it (snapback). Since tap testing imparts an impulse to the system, all modes are excited uniformly, whereas snapback techniques emphasize excitation of the lower-order modes. The accelerometer was used to measure accelerations at various points for each configuration. The spectrum analyzer performs a Fast Fourier Transform (FFT) on the accelerometer time history signal and produces a plot of frequency versus amplitude. This plot can be used to determine fundamental and higher-order eigenvalues, mode shapes, and damping values.

The piping configurations were subjected to low-frequency and high-frequency excitations (corresponding to seismic and aircraft impact loads, respectively) by three different excitation levels which are shown in Figures 1 and 2 as horizontal acceleration spectra. Most of the energy generated during the low-frequency excitation was concentrated between 2 and 15 Hz and rises up to 12g (see Figure 1). In the high-frequency excitation, most of the energy was concentrated between 8 and 80 Hz with a maximum peak value for acceleration of 24g for the most severe excitation level (test spectra 3). The low-frequency excitation lasted more than ten seconds each, and the high-frequency excitation lasted about one second each. These excitations are much higher than the worst theoretical case predicted for a variety of structures and load cases, including the effect of spectral amplification by secondary structures.

Typical earthquake table motion was provided to the shake table by selective filtering of random noise or synthesized earthquake time histories. Each earthquake signal, once modified, was recorded on analog tape for playback. In order to achieve the specific level of excitation required, additional signal conditioning and filtering were done by using various instrumentation. In general, the excitation level was controlled by the actuator amplitude settings and the drive signal control box settings. The bandpass filtering was changed from one run to another in order to provide energy needed at lower frequencies for earthquakes and to provide energy needed at higher frequencies for aircraft impacts.

For each configuration, the pipes involved were subjected to, on the average, three earthquakes (low-level, medium-level, and high-level) in each of the two shake table independent horizontal excitation directions.

All table motion involving aircraft impact tests was provided by selective filtering of one-second bursts of random noise generated by a Hewlett-Packard dual-channel analyzer and later recorded on analog tape. In order to achieve the specific level of aircraft impact required, additional signal conditioning and filtering were performed on the recorded signal.

For each configuration, the pipes involved were subjected to, on the average, three aircraft impact events (low-level, medium-level, and high-level) in each of the two shake table independent horizontal excitation directions.

The nine small piping configurations, which were selected for testing, are representative models of the large majority of piping systems in nuclear power plants and are the most critical sections for each type. Trapeze-supported, hung, and horizontally restrained systems were included in the test program. A variety of boundary conditions, such as one-dimensional restraints, hangers, stops, pressure ranges, and added masses, were also investigated to simulate values.

The configurations tested were water-filled and pressurized between 5 bars and 120 bars (70-1700 psi). A total of three different pipe sizes (DN 15, DN 25 and DN 50; that is, 0.6 inch, 1.0 inch, and 2.0 inch) and two different steel compositions (ferretic and austenitic) were tested. Concentrated eccentric masses were included, as indicated, to represent valves located along the pipe lines in actual installations. Simulated valve eccentricities of 0.4, 0.8 and 2.4 kg-m (35, 70, 211 lb-in.) and masses of 8, 12, and 24 kg were used on the DN 15, DN 25 and DN 50 pipe.

Table 1 presents the matrix of support arrangements, pipe size and routing, and internal pressure used in each test. Figures 3 through 11 are the configurations tested and while these configurations were specifically chosen from KWU plants they are also representative of SONGS 1 small bore piping. Figure 12 is applicable to SONGS 1 small bore piping which is supported by pipe racks. However, for SONGS 1, in most cases, U-bolts were utilized in lieu of welding or clamping shoes (T sections) to the piping. Figures 10 and 11 while more representative of fuel bundles, nevertheless, are representative of any 1/2-inch pipe which has long vertical runs between supports.

Table 2 compares German designations for the piping tested to U.S. designations.

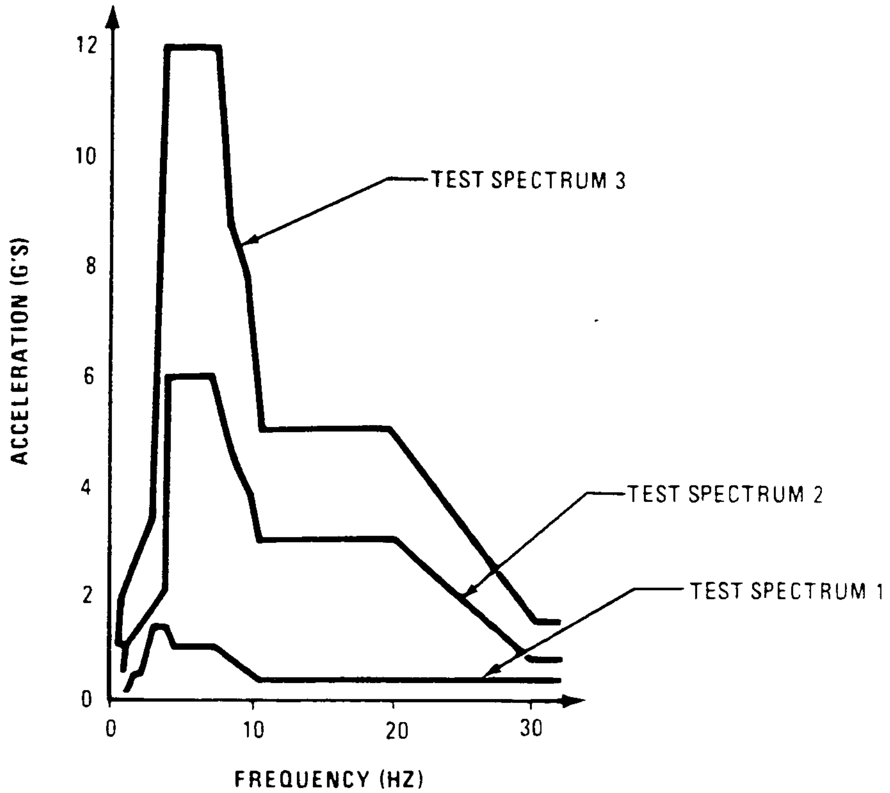


FIGURE 1
TEST SPECTRA LOW FREQUENCY LOAD CASE HORIZONTAL (2% DAMPING)

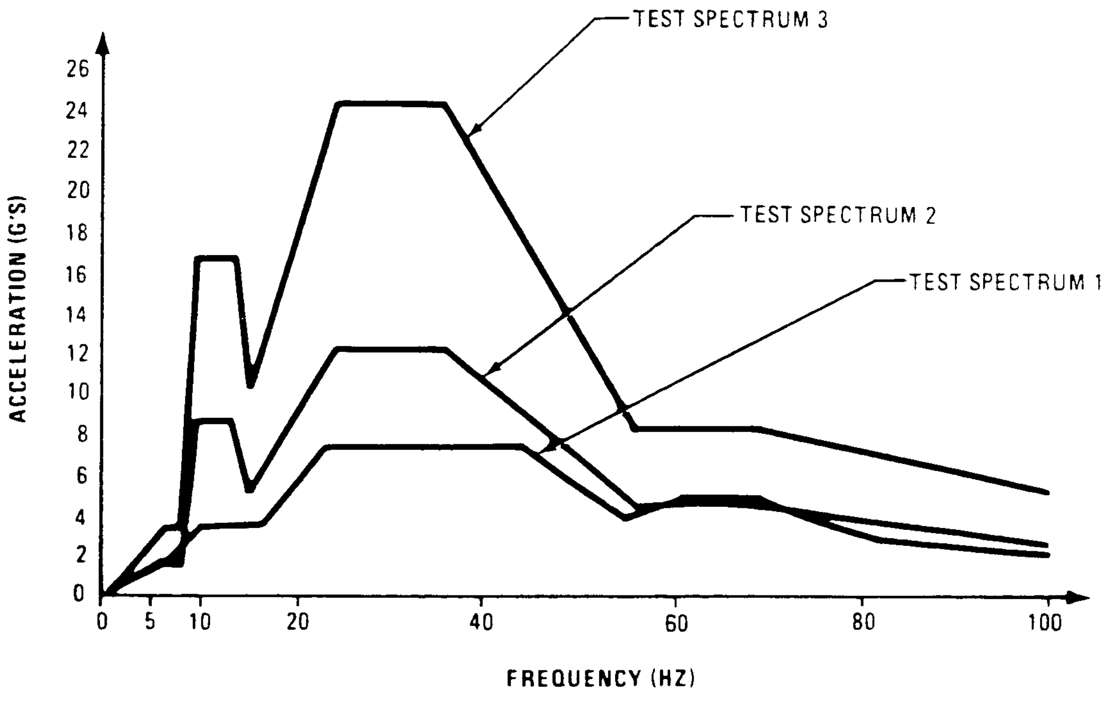
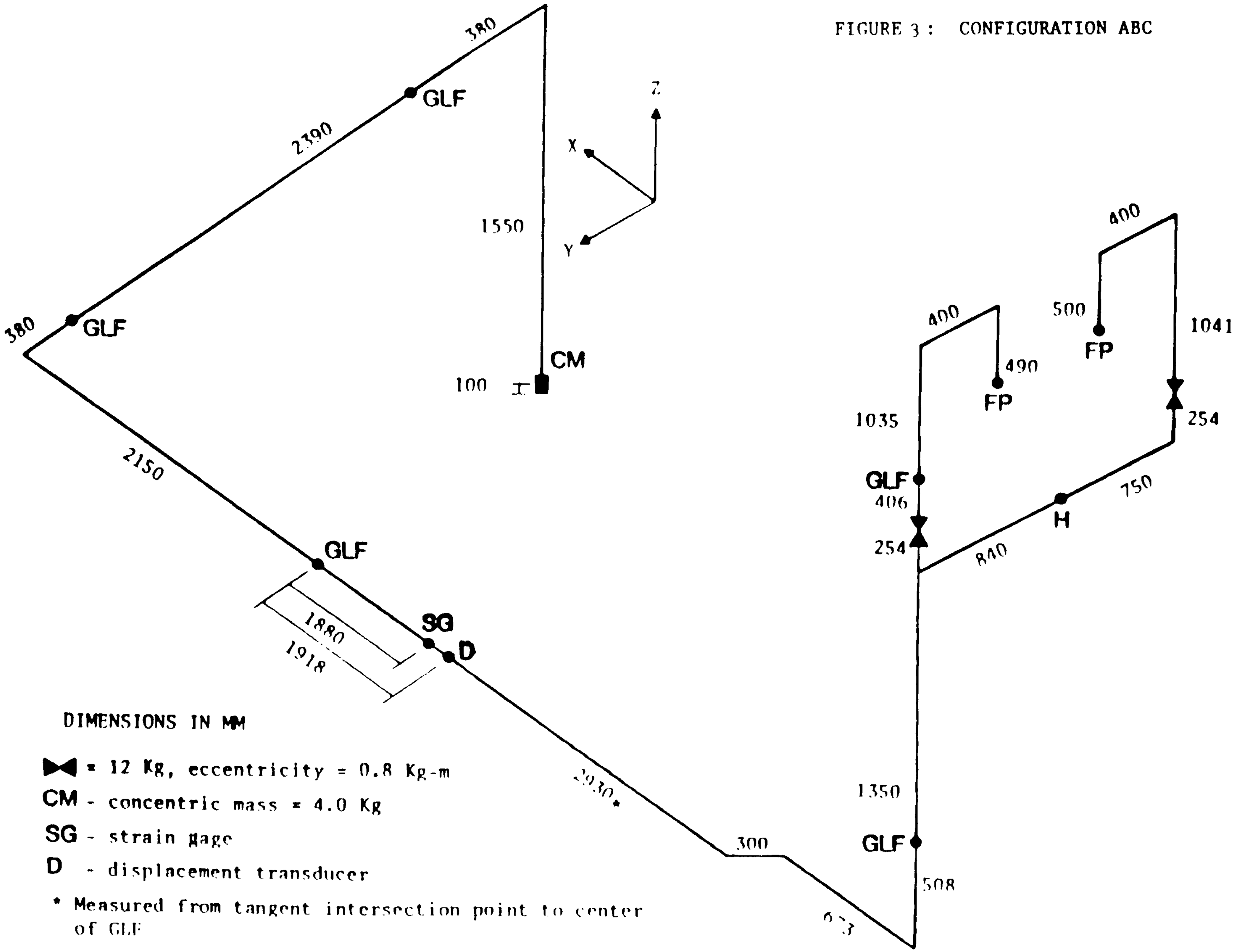


FIGURE 2
TEST SPECTRA HIGH FREQUENCY LOAD CASE HORIZONTAL (2% DAMPING)

TABLE 1: TEST CONFIGURATION MATRIX

	Pipe					Radius of Bends				Angle of Bend				Support Type						End Condition				Simulated Valve Mass				Internal Pressure			Support Spacing				
														Hung		Bottom																			
Configuration	DN 15-A	DN 15-F	DN 25-A	DN 25-F	DN 50-A	75 mm	125 mm	30 m	None	90°	135°	164°	180°	Rod	Spring	With T	With Bockschelle	Vertical	GLF	Fix-Fix	Fix-Free	2D-3D	2D-3D-2D	M = 8 kg	M = 12 kg	M = 24 kg	Water-Filled	5 bar	100 bar	175 bar	2 m	3 m	4 m	5 m	Ref. Drawing
ABC				●			●			●	●			●			●		●		●				●		●	●							●
D*1	●		●			●	●			●	●			●		●	●		●	●							●			●					●
D*2	●		●			●	●			●	●			●		●	●		●	●				●	●		●			●					●
D*3	●				●	●				●	●			●		●	●		●	●							●		●						●
D*4	●				●	●				●	●			●		●	●		●	●				●		●	●		●						●
D*5	●				●	●				●	●				●	●	●		●	●				●		●	●		●						●
E		●	●		●			●				●				●					●						●	●				●	●		
E.1		●							●				●					●				●					●	●			●	●			
E.2		●							●				●					●					●				●	●						●	

FIGURE 3 : CONFIGURATION ABC



DIMENSIONS IN MM

⧓ = 12 Kg, eccentricity = 0.8 Kg-m

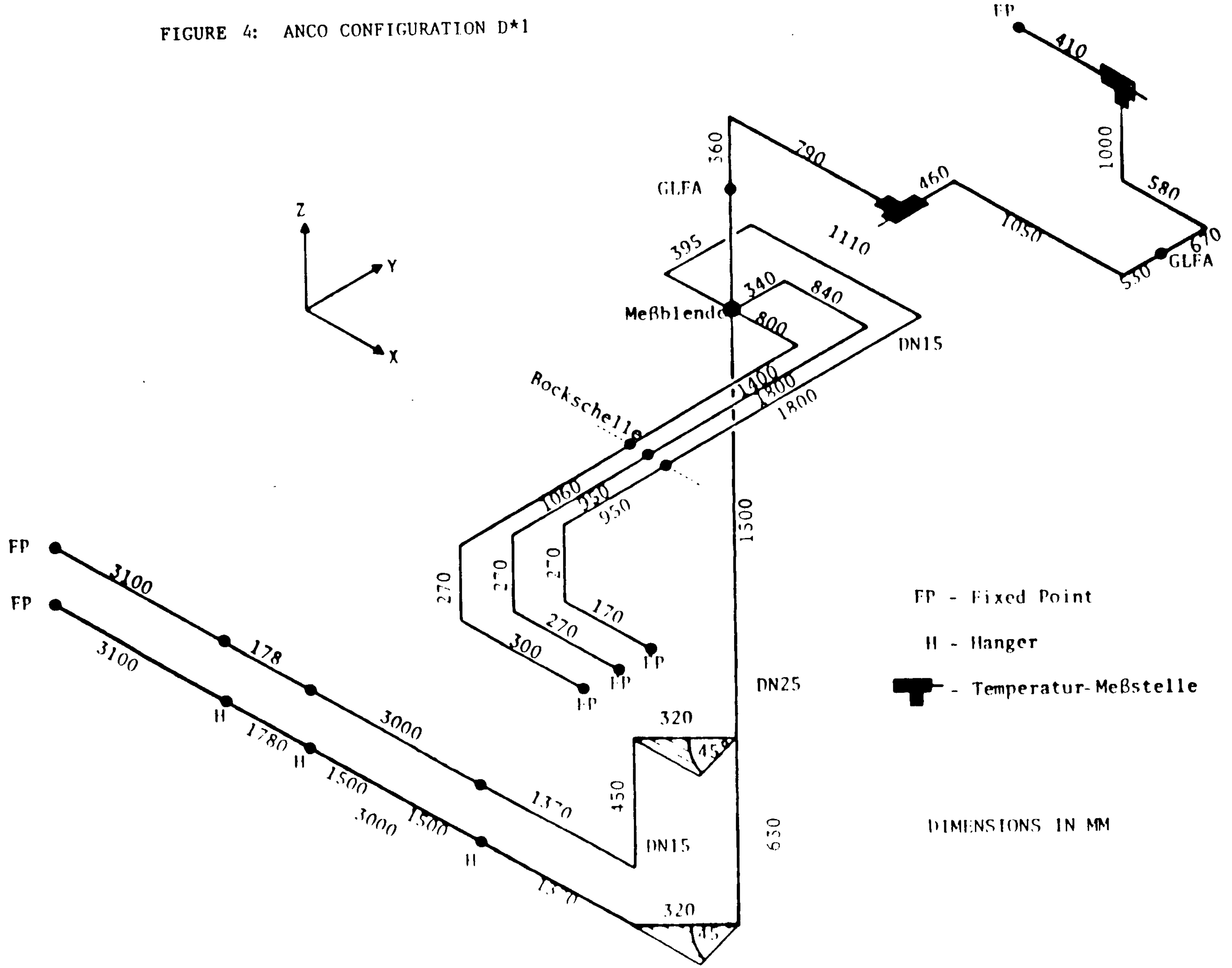
CM - concentric mass = 4.0 Kg

SG - strain gage

D - displacement transducer

* Measured from tangent intersection point to center of GLF

FIGURE 4: ANCO CONFIGURATION D*1



-11-

FIGURE 5: ANCO CONFIGURATION D*2

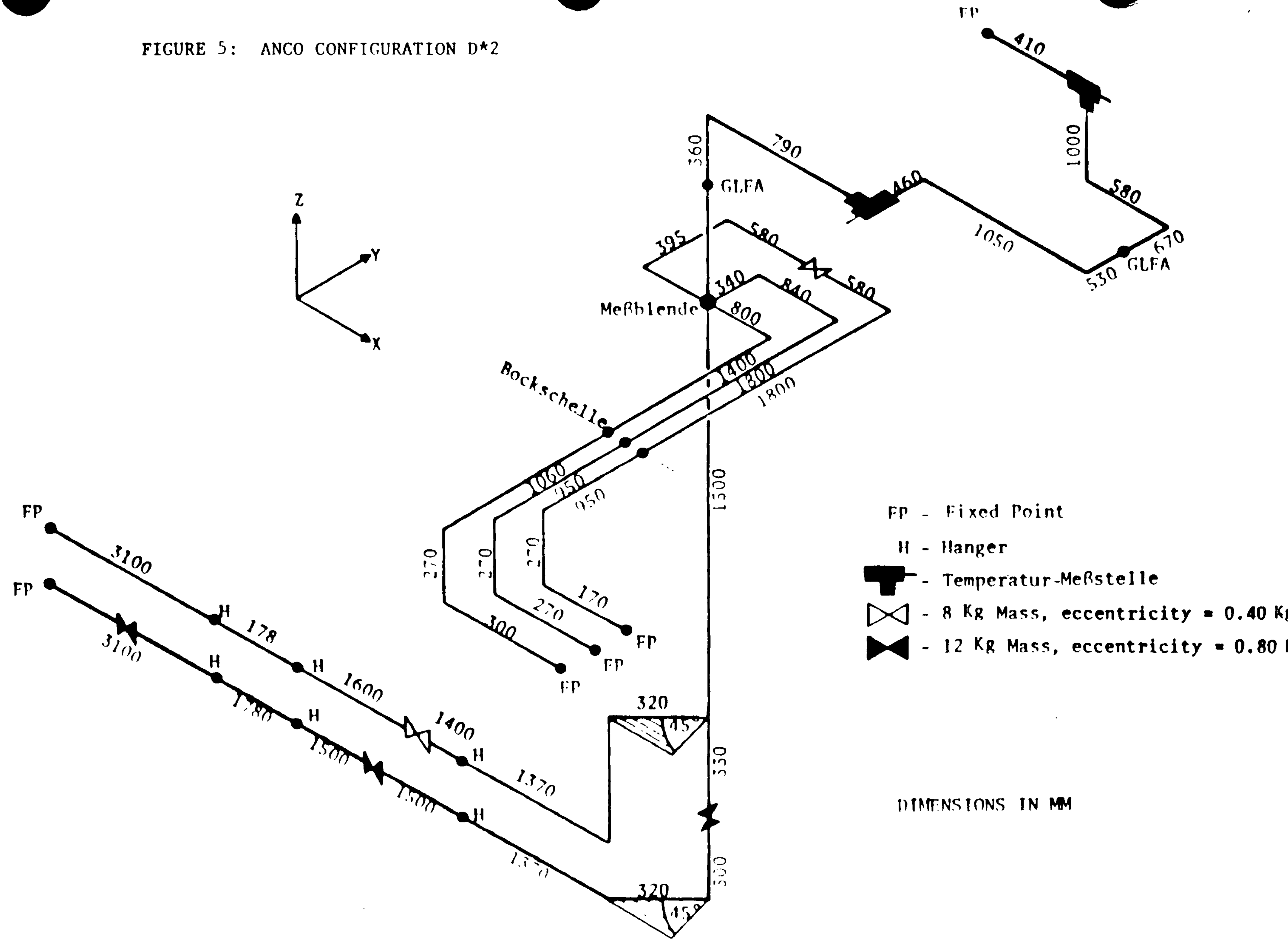


FIGURE 6: ANCO CONFIGURATION D*3

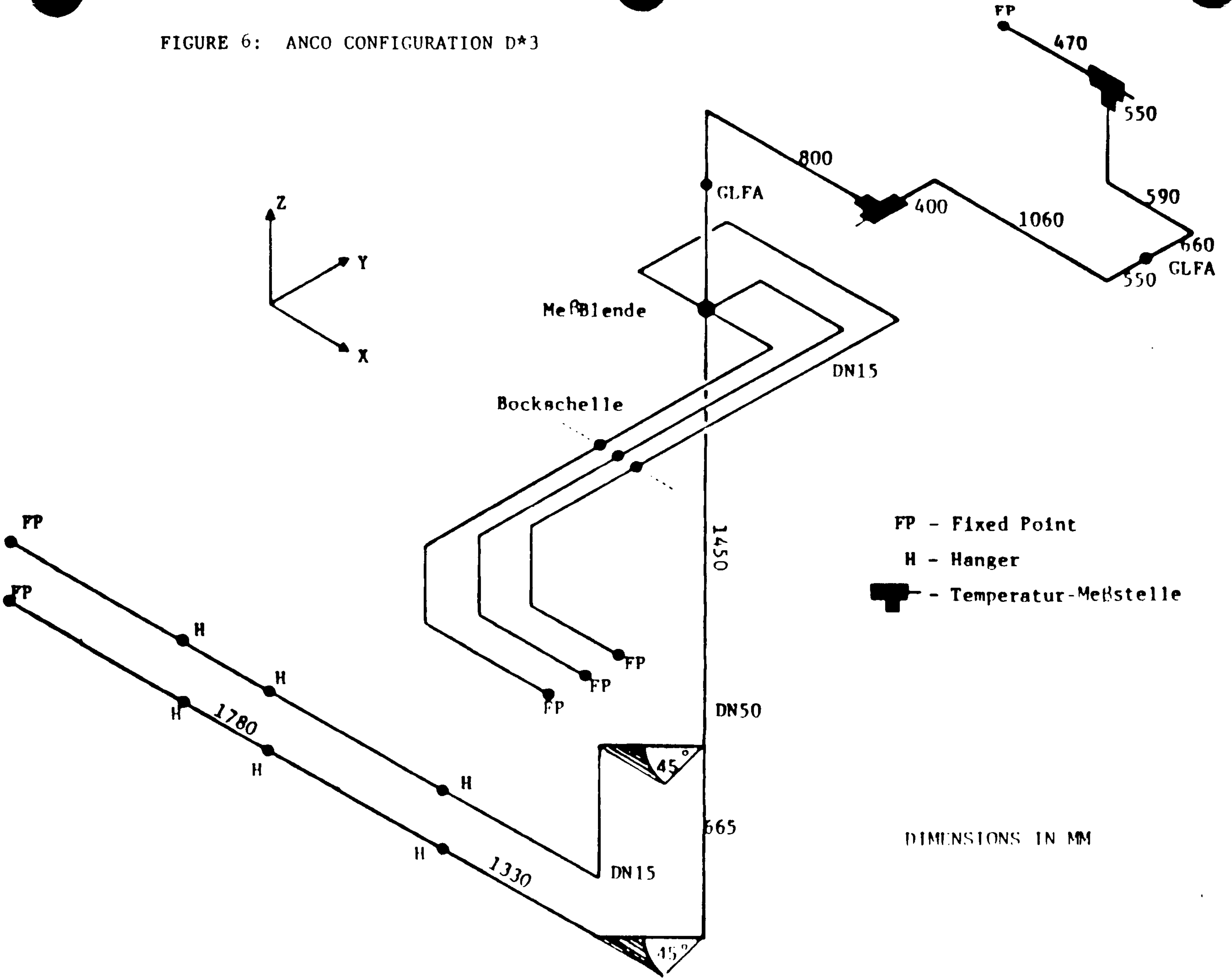


FIGURE 7: ANCO CONFIGURATION D*4

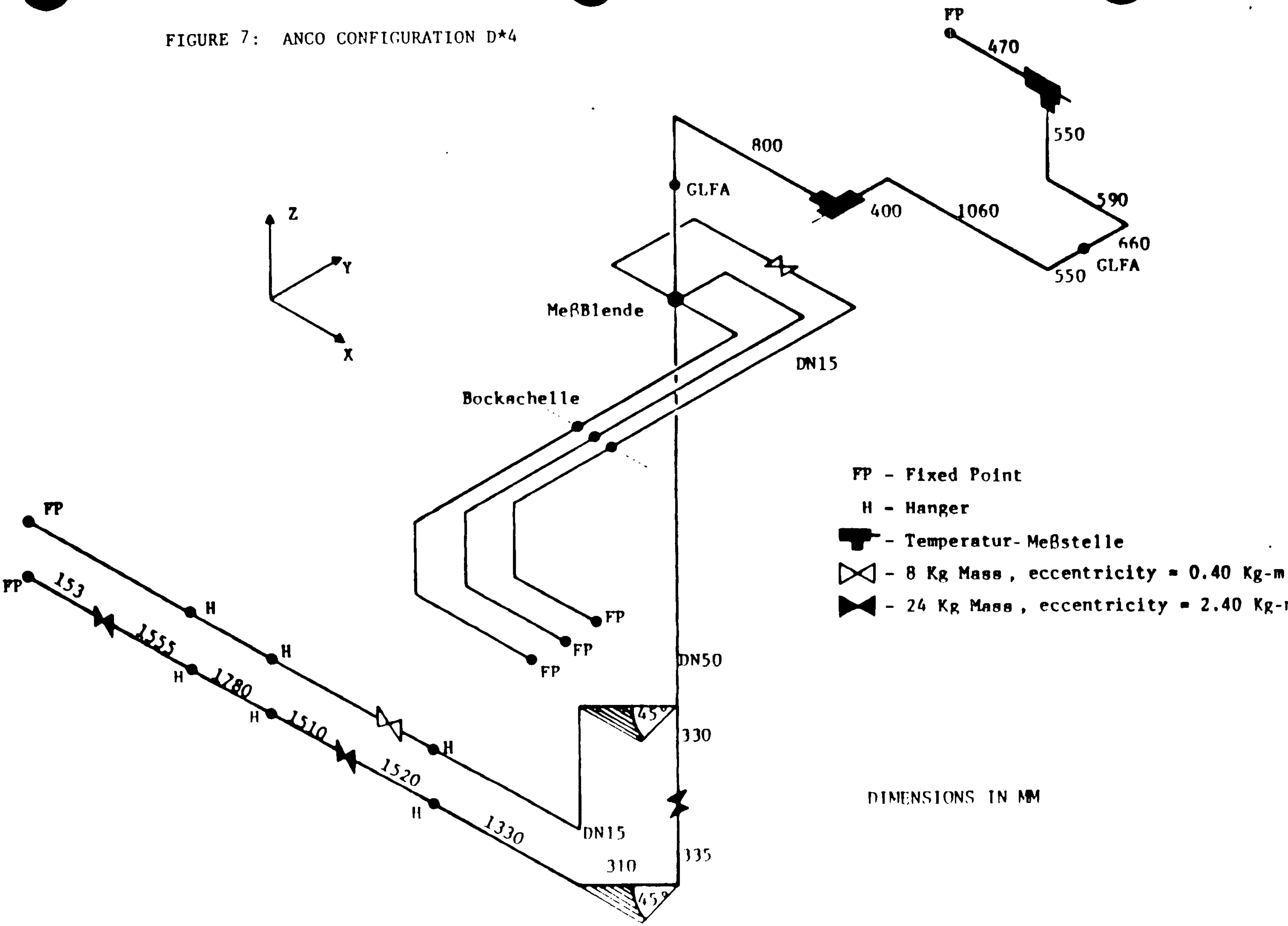
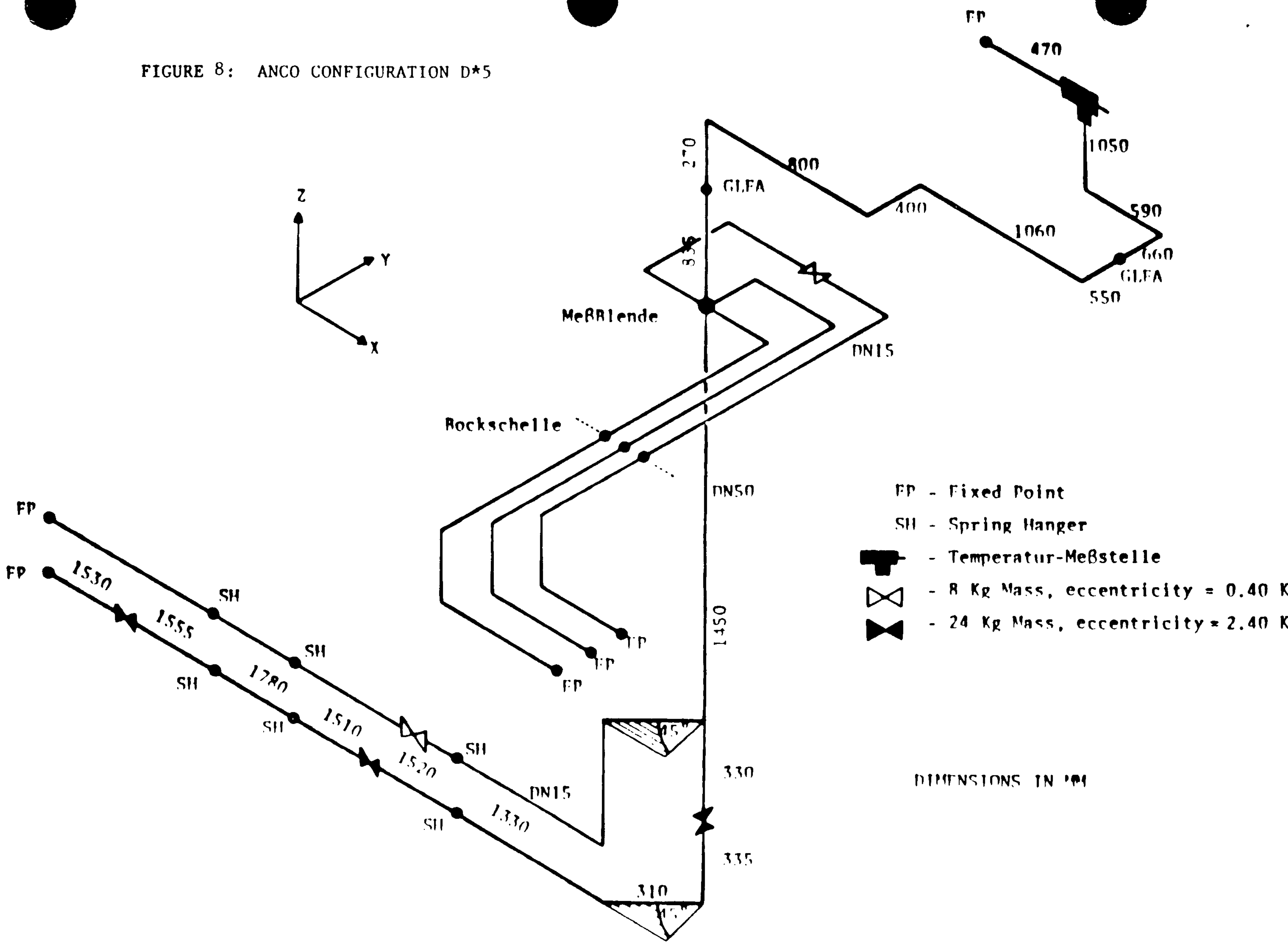


FIGURE 8: ANCO CONFIGURATION D*5



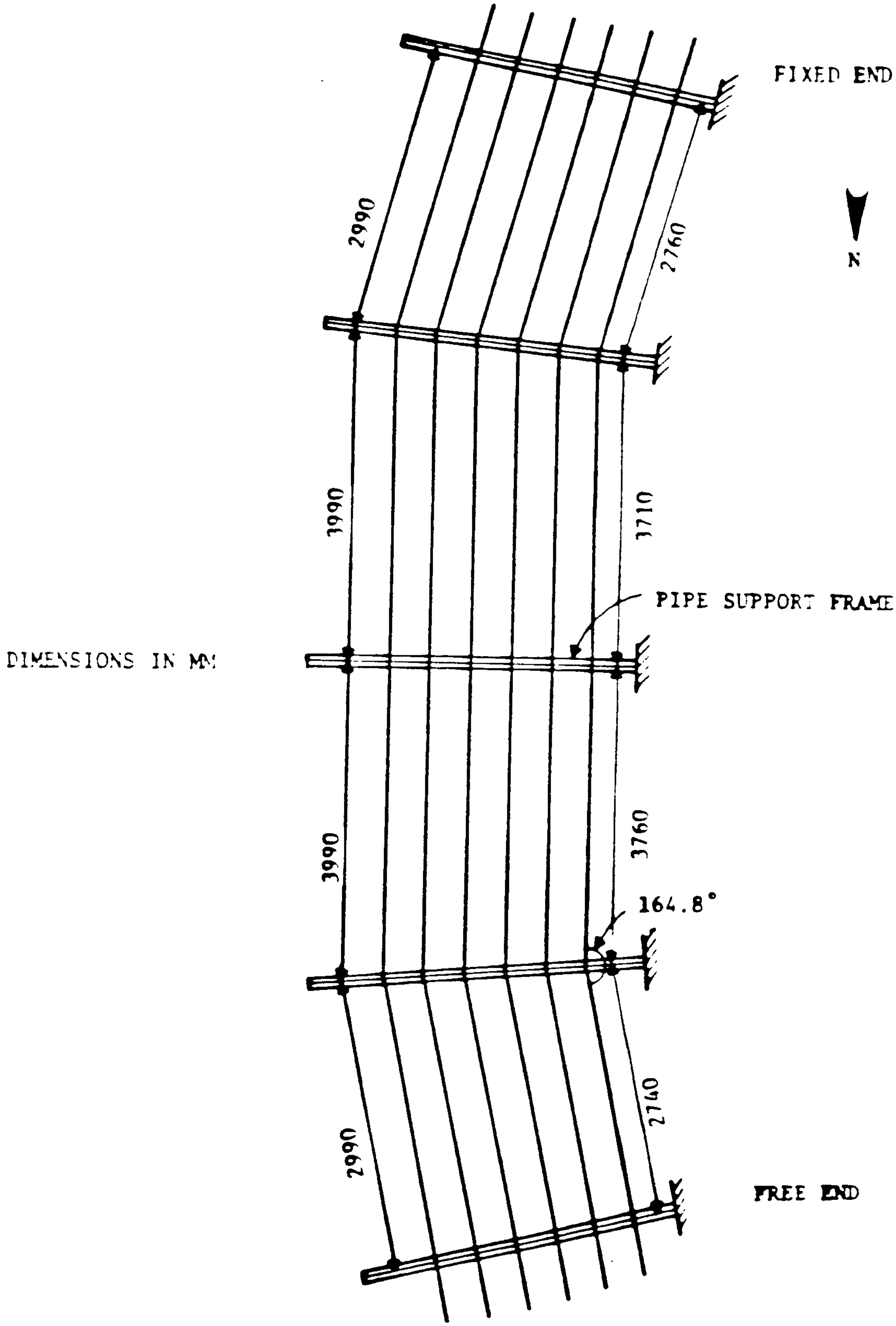


FIGURE 9 : CONFIGURATION E SUPPORT FRAME LOCATION

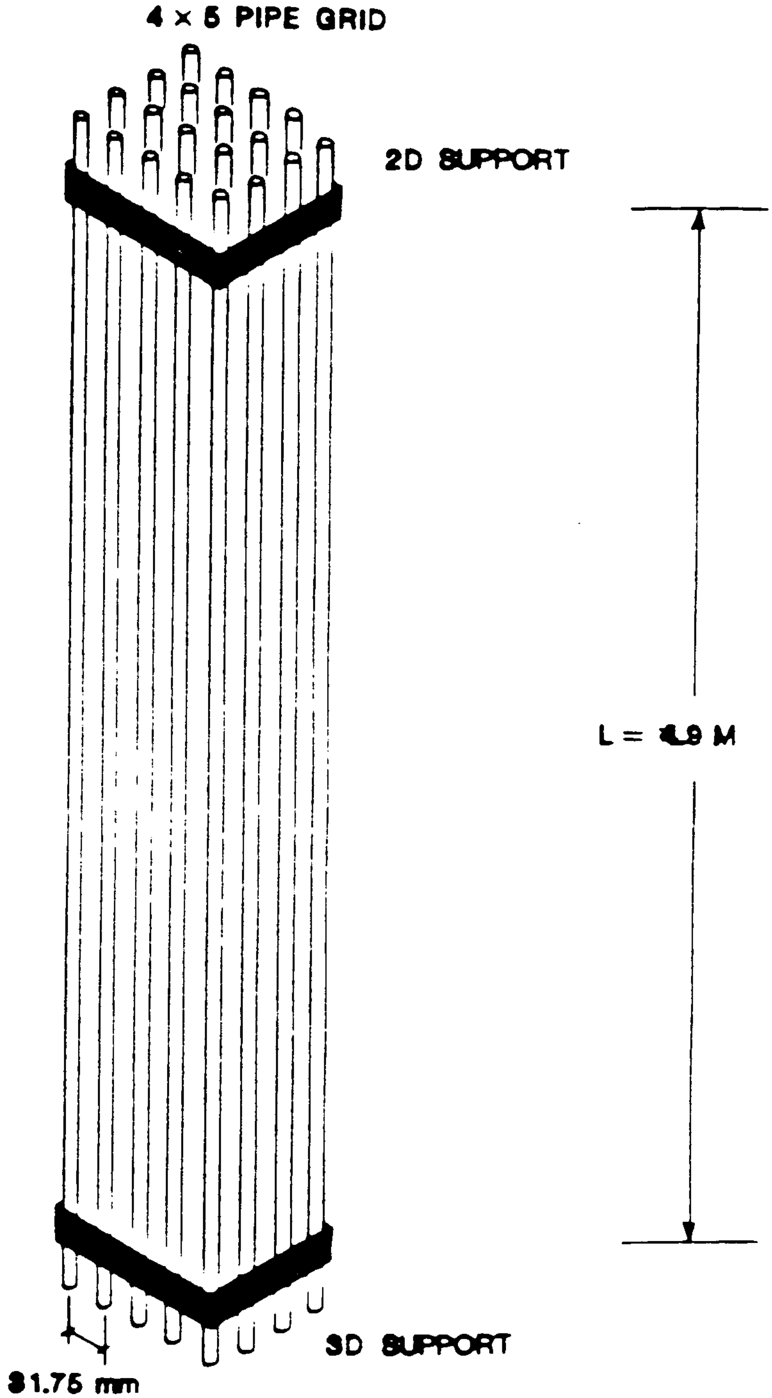


FIGURE 10: CONFIGURATION F.1

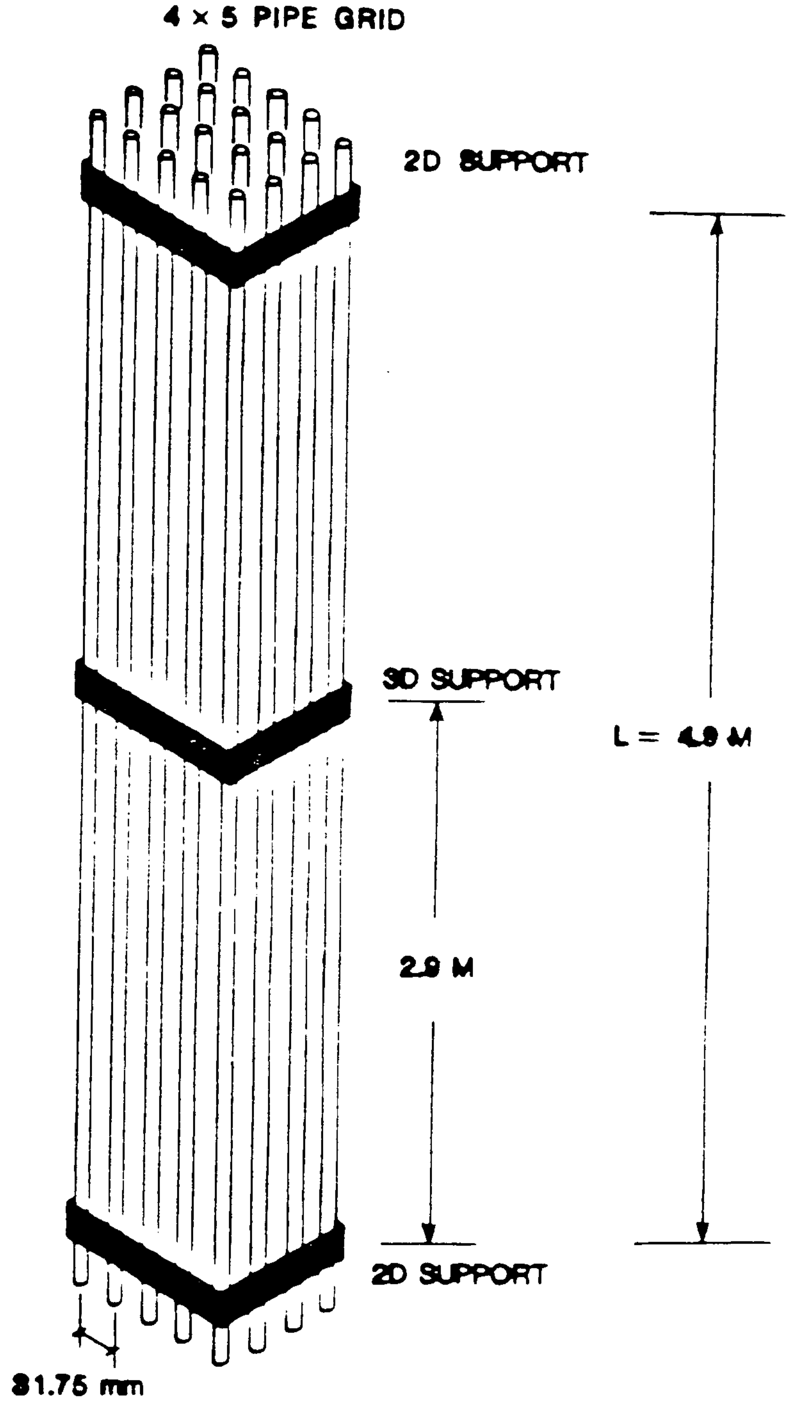


FIGURE 11: CONFIGURATION F.2

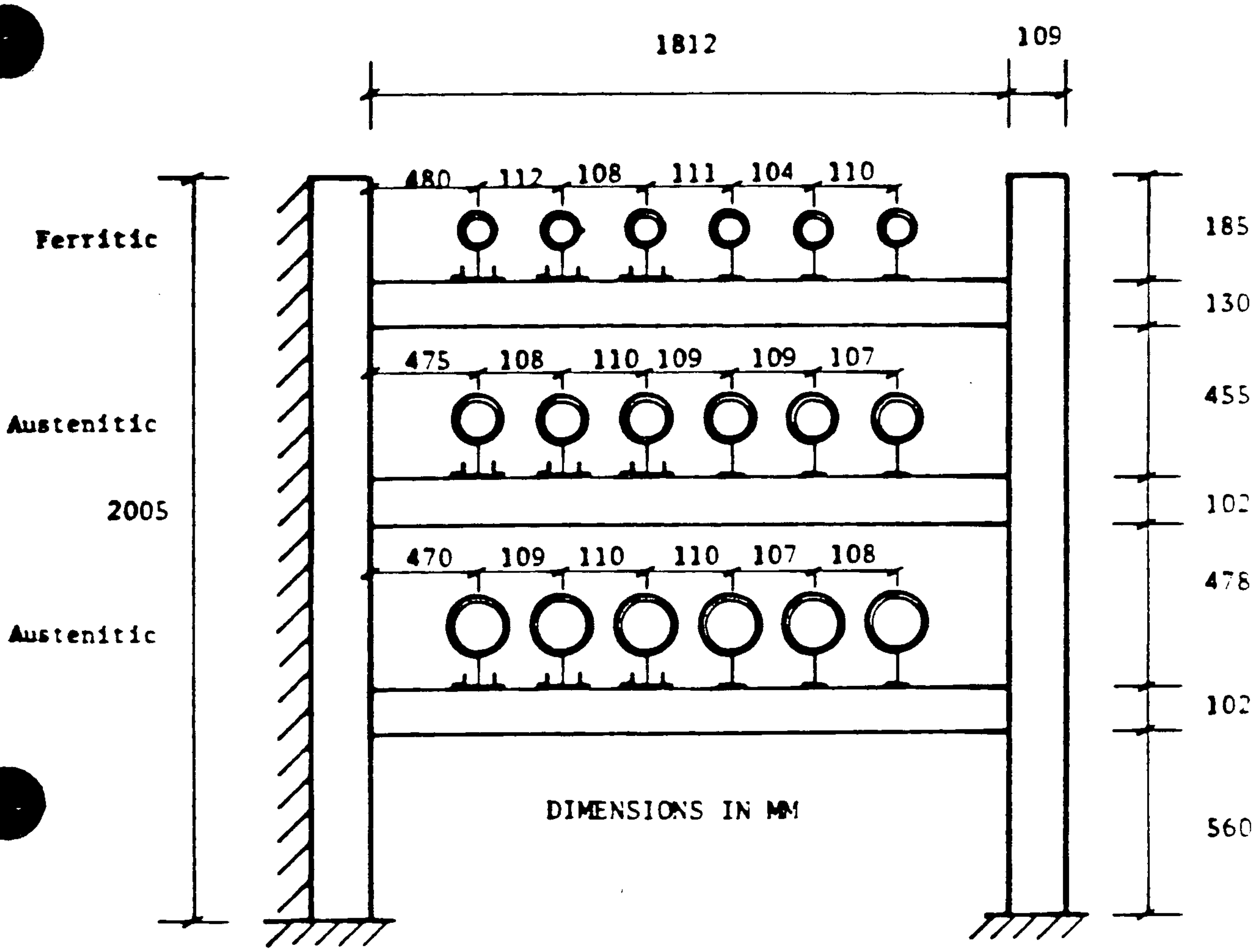


FIGURE 12 : CONFIGURATION E - SUPPORT FRAME AND PIPE LOCATIONS AT FIXED END

PIPE	US DESIGNATION	GERMAN DESIGNATION	OUTSIDE DIA., mm	INSIDE DIA., mm	CONFIG.	REF. FIG.	SUPPLIER
DN15	--	Ferrit St 35.4	21.30	17.30	E, F	4.7 - 4.10	KWU
DN15	1/2 IPS Sched. 10 Grade 304L	Austenit 1.4541	21.30	17.30	D	4.2 - 4.6	US
DN25	--	Ferrit St 35.4	33.70	28.50	A, B, C	4.1	KWU
DN25	1 IPS Sched. 10 Grade 304L	Austenit 1.4541	33.70	28.50	D, E	4.2 - 4.8	US
DN50	2 IPS Sched. 10 Grade 304L	Austenit 1.4541	60.3	54.5	D, E	4.2 - 4.8	US

TABLE 2: PIPE SPECIMENS USED FOR SEISMIC AND AIRCRAFT IMPACT TESTING

3.1.2 Applicability of KWU Tests to SONGS 1

The small bore piping at SONGS 1 is laid out and supported in a manner commonly found in power plant facilities. It has functioned as intended, to-date, as would be expected of piping systems designed and fabricated to high quality standards.

This section is intended to present piping configurations in a format that facilitates basic comparisons. Five small bore piping configurations have been included (i.e. drawn as isometric views of the respective geometries). Accompanying each configuration is a description of the operating conditions and other pertinent data. The applicable spectra curve is also provided for each example.

The first piping configuration presented herein is ANCO test configuration ABC which underwent extensive testing to justify the existing KWU installations, without backfitting. In addition, four examples are included which are typical of SONGS 1 small bore piping. They are presented in their "as-is" supported configuration.

The spans of the SONGS 1 small bore piping are typically less than those of the KWU piping tested. Also, the wall thickness of the majority of the SONGS 1 small bore piping is schedule 40 or greater which is stronger than the KWU piping tested. Lastly, the in-structure response spectra for SONGS 1 in those areas where small bore piping is supported, have peaks in the same frequency range as the KWU tests and the corresponding accelerations for 2% damping, in general, are enveloped by those used for the KWU tests.

Piping supported in the manner shown in these examples has considerable seismic withstand capability. The ANCO test configuration (designated ABC) was subjected to severe dynamic loadings comparable to but more severe than SONGS 1 seismic spectra. The test configuration included large spans, un-supported lumped masses, free ends with concentrated masses,and supports that provided restraint to the piping which in no way was more conservative than that typically found on SONGS 1 piping. It follows that small bore piping supported in a like manner to the SONGS 1 examples has inherent seismic margins of safety in excess of the ANCO test configuration ABC.

Table 3 presents a matrix which describes the size, schedules, and materials used for small bore piping at SONGS 1.

3.1.2.1 Examples of Similarities Between SONGS 1 Small Bore Piping and KWU Tests

The following four examples are representative of the small bore piping at SONGS 1 and are compared against the ANCO test configuration ABC.

TABLE 3

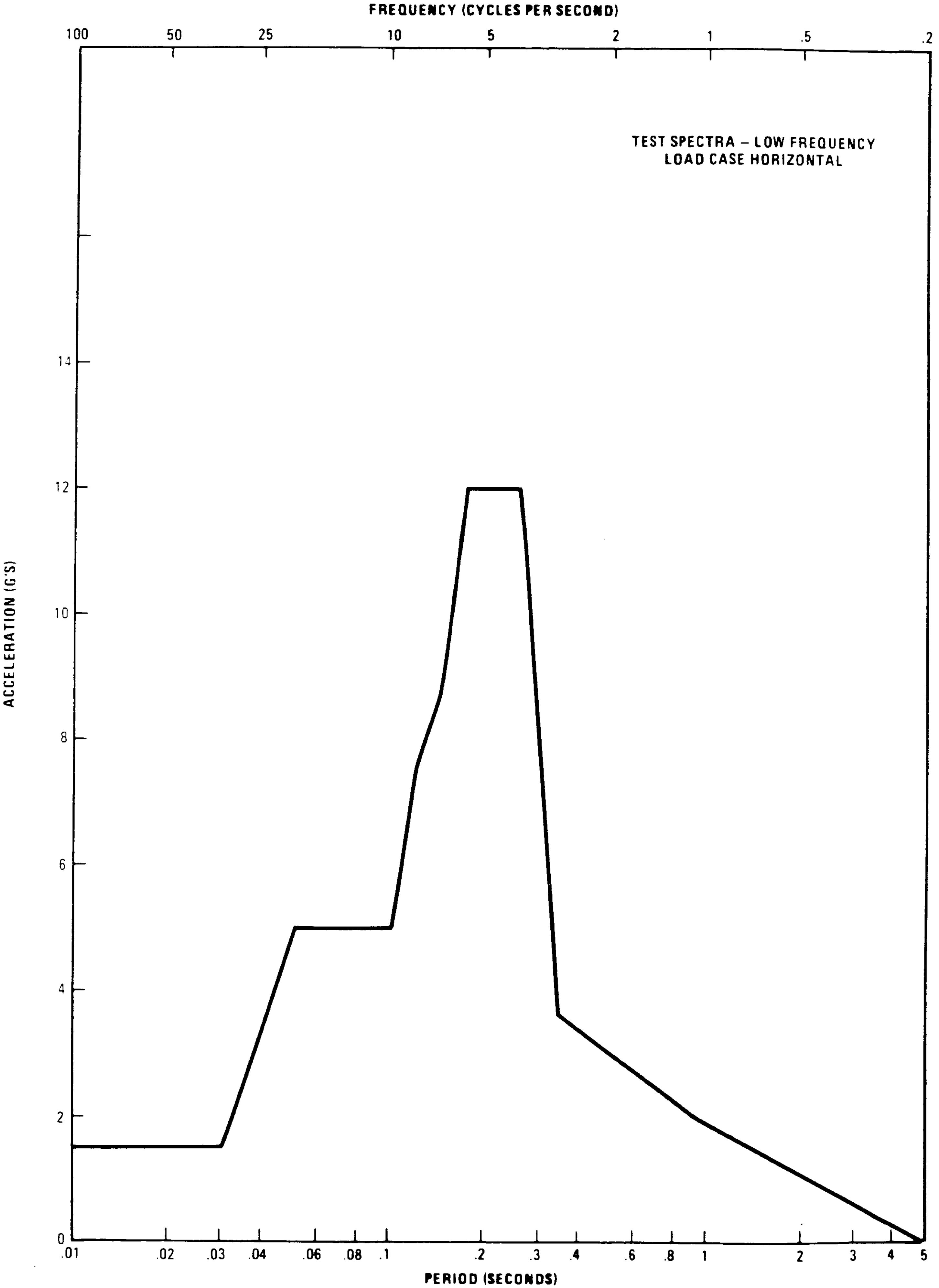
SONGS 1 Small Bore Pipe Material Specifications

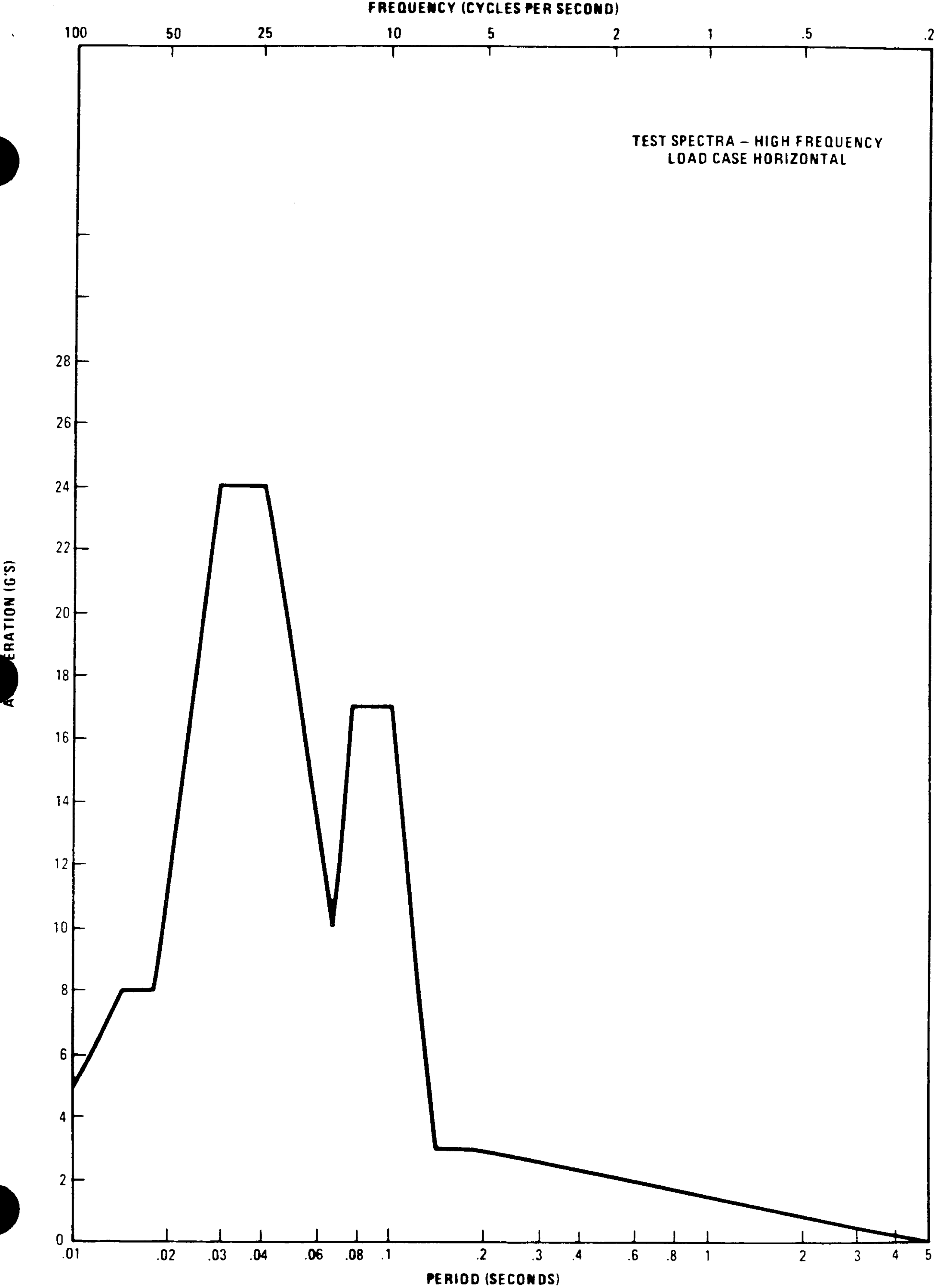
Material Class	Size	Schedule	Pressure Rating	Material	Fittings
151	1/2	10S	150	A-312, Type 304	Socket Weld
	3/4	10S	150	A-312, Type 304	Socket Weld
	1	10S	150	A-312, Type 304	Socket Weld
	1-1/2	10S	150	A-312, Type 304	Socket Weld
	2	10S	150	A-312, Type 304	Socket Weld
EG	1/2	40	600	A-106, GR.B	Socket Weld
	2	40	600	A-106, GR.B	Socket Weld
EG1	1/2	40	600	A-312, Type 304	Socket Weld
	3/4	40	600	A-312, Type 304	Socket Weld
HH	3/4	40	150	A-53, GR.B	Socket Weld
HM2	3/4	40	150	SA-312, GR-304L	Socket Weld
HP	2	10S	150	A-312, Type 304	Butt Weld
HP2	1	10S	150	A-312, Type 304	Butt Weld
	2	10S	150	A-312, Type 304	Butt Weld
GG	1	40S	300	A-106, GR.B	Socket Weld
JN	1-1/2	40S	150	SA-312, TP 304L	Socket Weld
601R	2	40S	600	A-312, Type 304	Socket Weld
2501R	3/4	160	2500	A-312, Type 316	Socket Weld
2501R	1	160	2500	A-312, Type 316	Socket Weld
	2	160	2500	A-312, Type 316	Socket Weld
2502R	2	160	2500	A-312, Type 304	Socket Weld

ANCO Test Configuration ABC

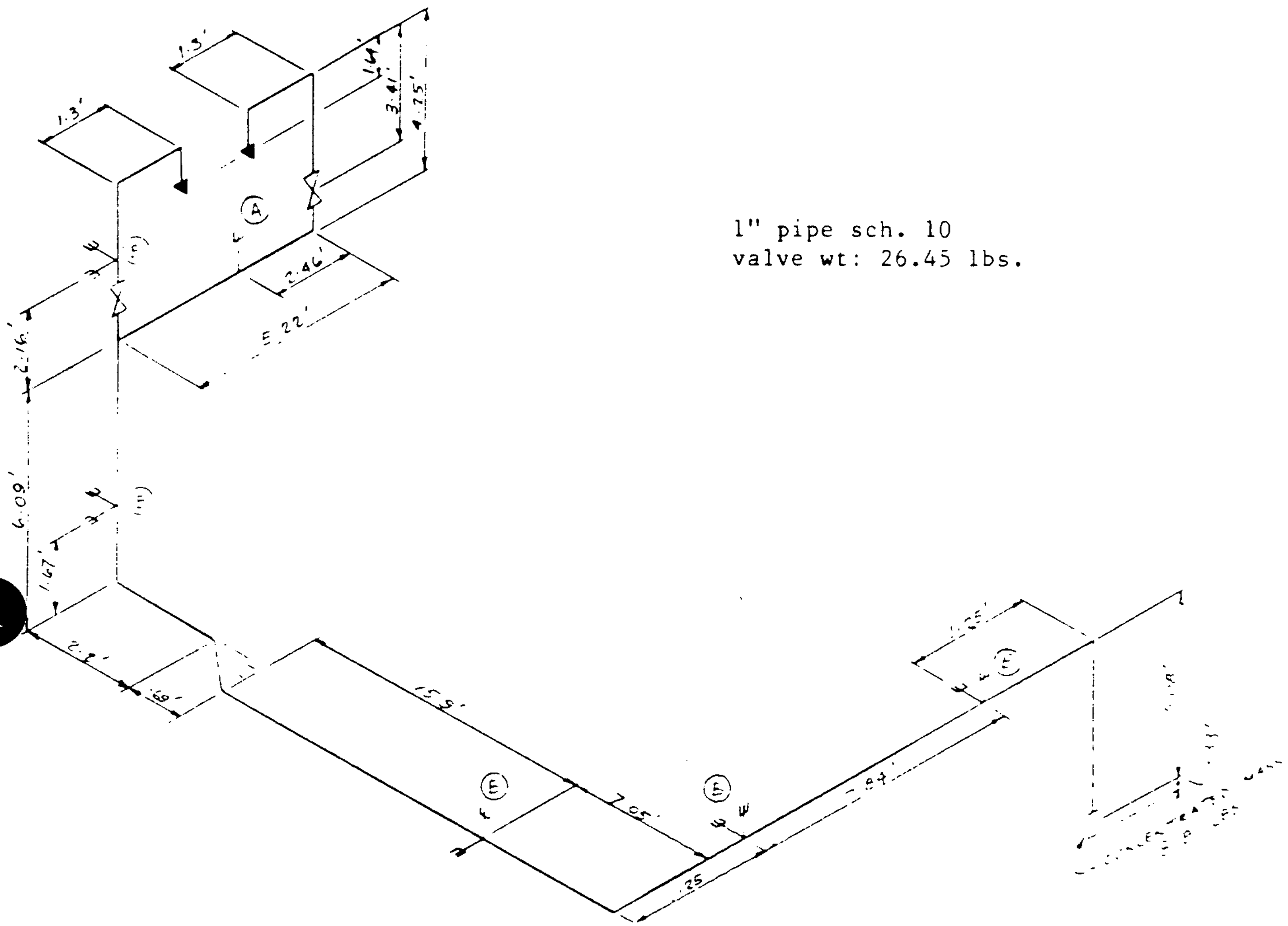
Configuration ABC represents the configuration of a system tested by ANCO Engineers for Kraftwerk Union using a shake table to simulate a seismic event and an aircraft impact event. The piping in this example is 1" O.D., schedule 10S. It is fixed at one end and restrained by one 1-directional rod hanger and five 2-directional guides. The maximum span in the example is 20'-6" with an average span of approximately 8'-6". One end of the pipe is free with a concentrated mass of 8.8 lbs at the extreme end of the pipe. The closest guide to this end is 6'-4" from the end.

High and low frequency tests were run on the pipe in configuration ABC. The maximum horizontal acceleration for the low frequency case is 12G's at 2% damping. This acceleration occurs at 4-7 cycles/sec. The maximum horizontal acceleration for the high frequency case is 24G's at 2% damping which occurs at 25-30 cycles/sec.


FREQUENCY (CYCLES PER SECOND)
100
50
25
10
5
2
1
.5
.2
TEST SPECTRA – LOW FREQUENCY
LOAD CASE HORIZONTAL
ACCELERATION (G'S)
14
12
10
8
6
4
2
0
.01
.02
.03
.04
.06
.08
.1
.2
.3
.4
.6
.8
1
2
3
4
5
PERIOD (SECONDS)


FREQUENCY (CYCLES PER SECOND)
100
50
25
10
5
2
1
.5
.2
TEST SPECTRA – HIGH FREQUENCY
LOAD CASE HORIZONTAL
ACCELERATION (G'S)
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
.01
.02
.03
.04
.06
.08
.1
.2
.3
.4
.6
.8
1
2
3
4
5
PERIOD (SECONDS)

TEST CONFIGURATION ABC



1" pipe sch. 10
valve wt: 26.45 lbs.

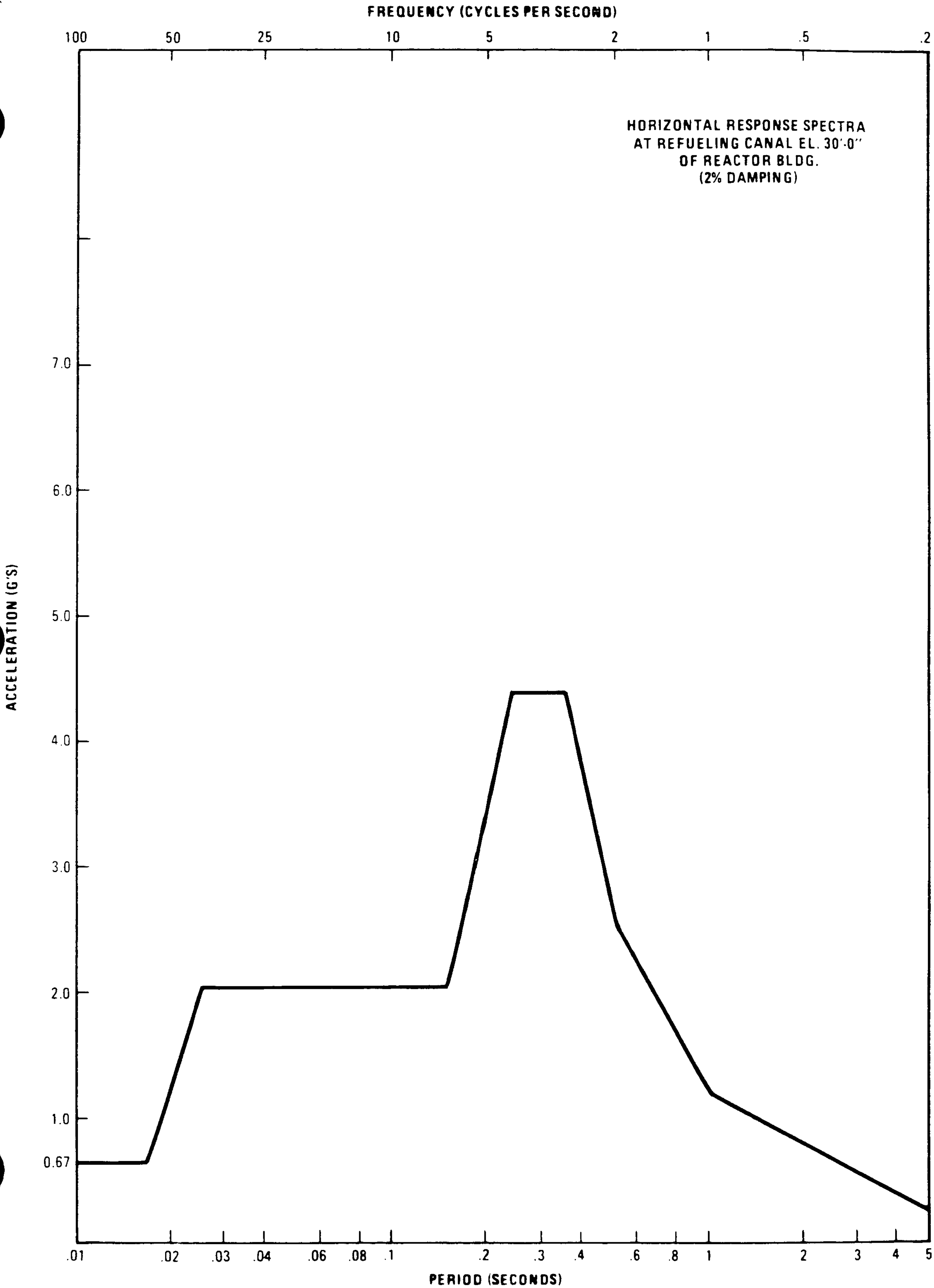
Support Type	Description
A	Rod Hanger
B	Guide

SONGS 1 Small Bore Pipe Example 1

Line 6111 is a 1" O.D., schedule 40 stainless steel drain line for a 6" O.D. miscellaneous water line off the Containment Sphere Sump Recirculation Pump. This drain line has a design temperature of 250°F and a design pressure of 155 PSIG.

Line 6111 is supported at one end by the 6" O.D. line and at the otner end, by a U-bolt 9 1/2" from the 5 lb. valve which terminates the line. The span between supports is 18'-3". Line 6111 is included in its entirety.

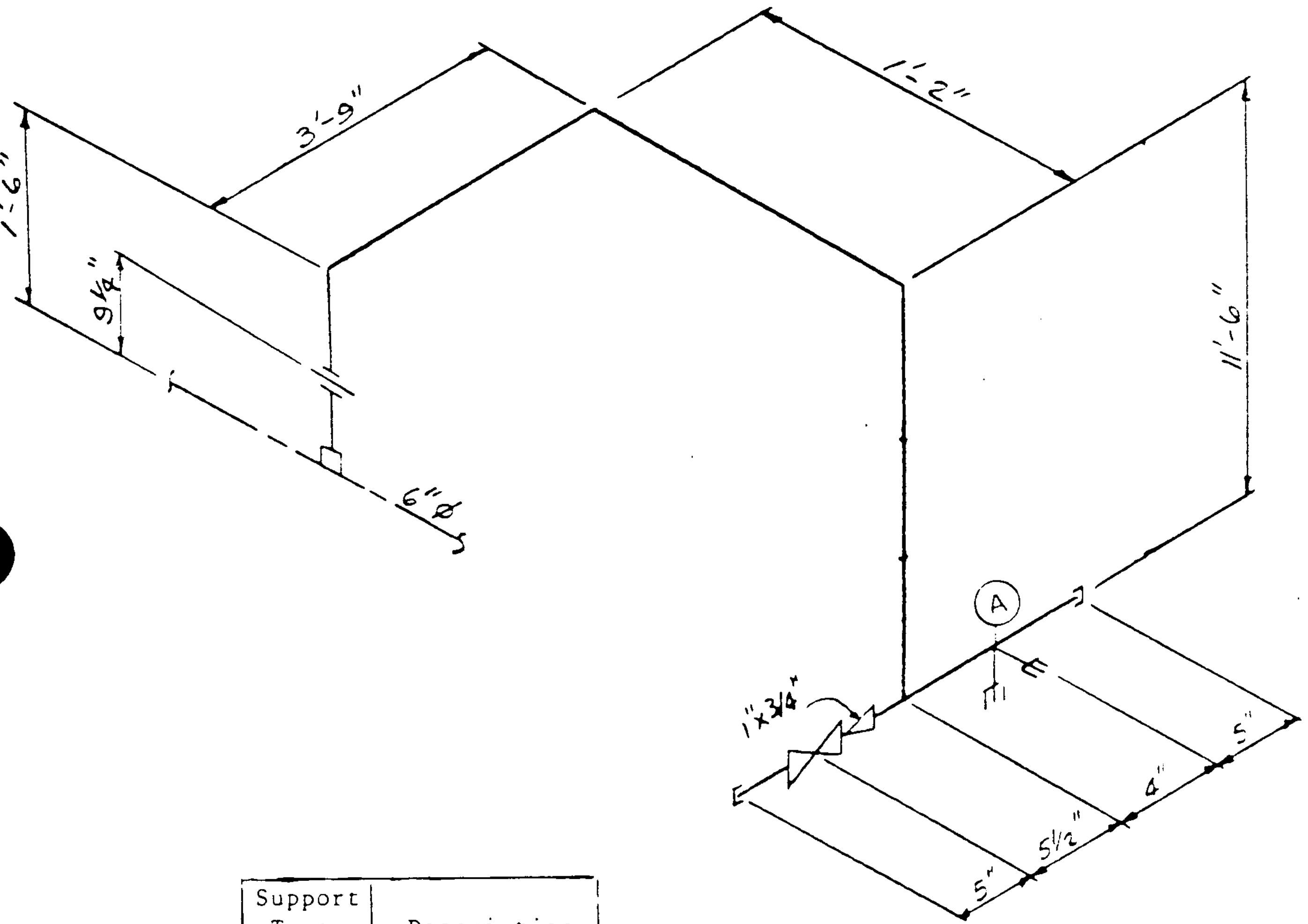
The maximum horizontal acceleration in the area of line 6111 is 4.4G's at 2-5 cycles/sec at 2% damping.


FREQUENCY (CYCLES PER SECOND)
100
50
25
10
5
2
1
.5
.2
HORIZONTAL RESPONSE SPECTRA
AT REFUELING CANAL EL. 30'-0"
OF REACTOR BLDG.
(2% DAMPING)
ACCELERATION (G'S)
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.67
.01
.02
.03
.04
.06
.08
.1
.2
.3
.4
.6
.8
1
2
3
4
5
PERIOD (SECONDS)

EXAMPLE 1

Line No: 6111-1"-HM2
Sch: 40S
Mat'l: A-312-TP3041
Welds: Socket
Insulation: None
Temp: 250° F
Press: 155 psig.
Valve wt: 5 lbs.

Ref. BPC Calc.
MW-300



Support Type	Description
A	U-Bolt

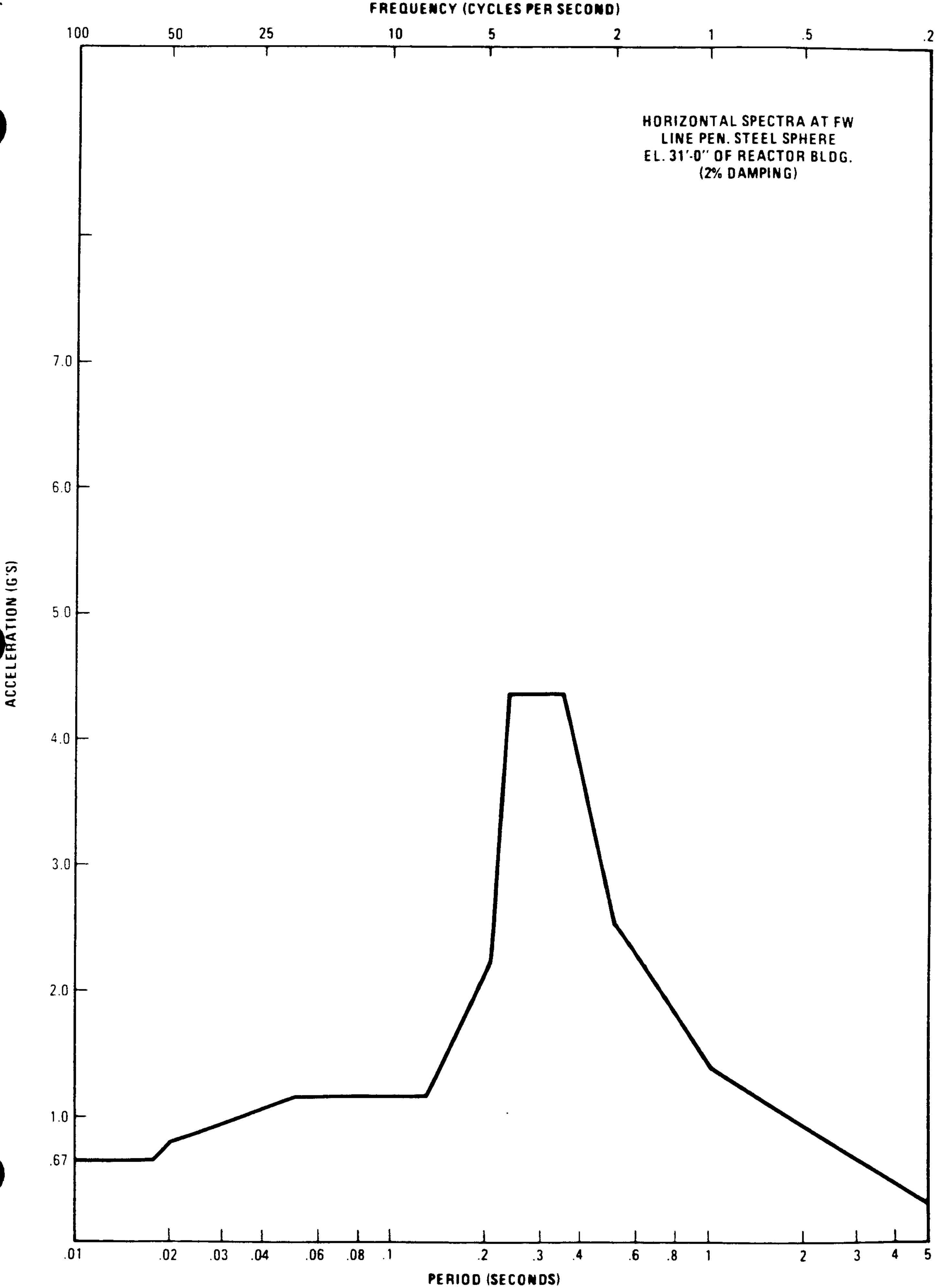
SONGS 1 Small Bore Pipe Example 2

Line 1409 is a 3/4" O.D., schedule 10 stainless steel line carrying nitrogen to the Pressurizer Relief Tank Piping. The design temperature of this nitrogen line is ambient. The design pressure is 50 psig.

The portion of line 1409 which is included in his example is supported at one end by containment penetration A-5 and at the other by an anchor which isolates this portion of line 1409. Three 2-directional intermediate supports are installed on the line.

Spans range from 2'-3 15/16" in the vicinity of a 50# check valve to 11'-4" on a section of uniform pipe.

The maximum horizontal acceleration in the area of line 1409 is 4.35G's at 2-5 cycles/sec at 2% damping.

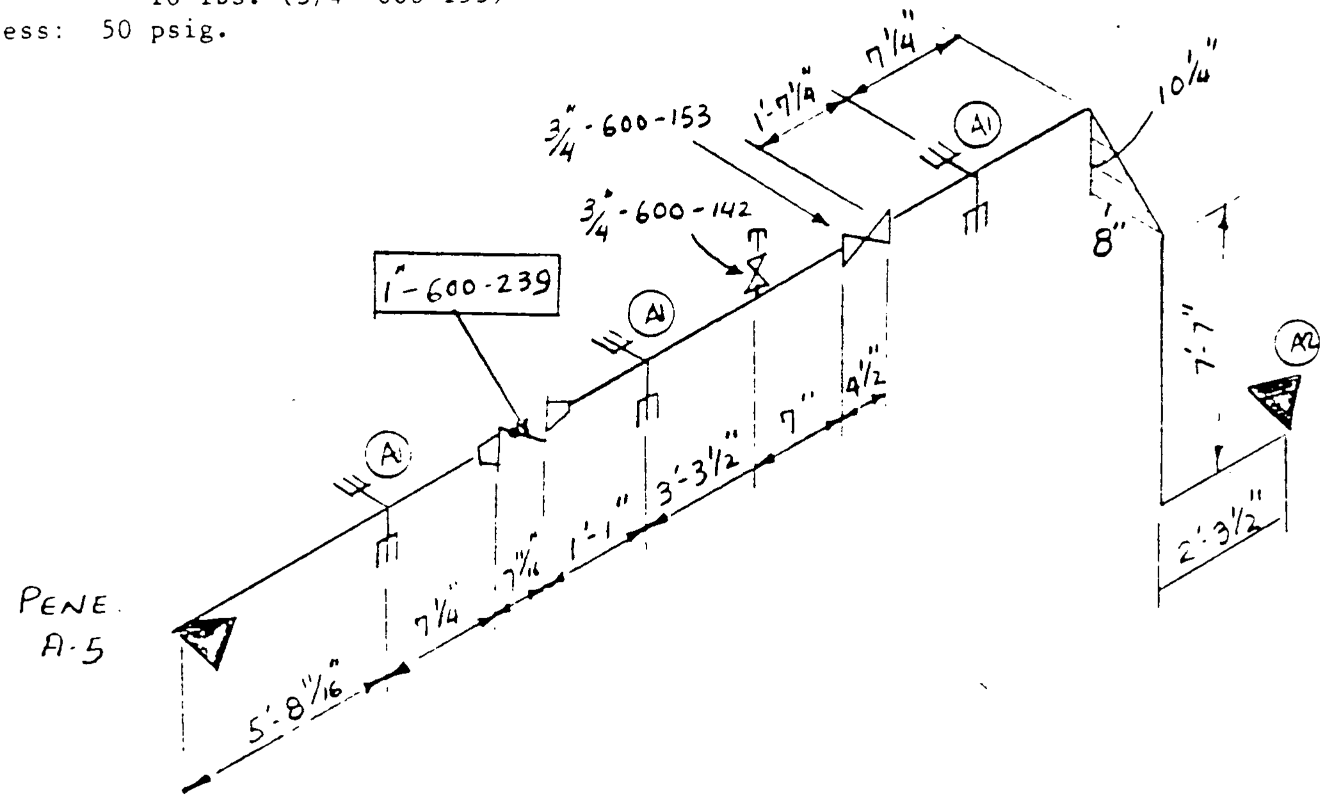


FREQUENCY (CYCLES PER SECOND)
100
50
25
10
5
2
1
.5
.2
HORIZONTAL SPECTRA AT FW
LINE PEN. STEEL SPHERE
EL. 31'-0" OF REACTOR BLDG.
(2% DAMPING)
ACCELERATION (G'S)
7.0
6.0
5.0
4.0
3.0
2.0
1.0
.67
.01
.02
.03
.04
.06
.08
.1
.2
.3
.4
.6
.8
1
2
3
4
5
PERIOD (SECONDS)

EXAMPLE 2

Ref. BPC Calc.
RC-51

Line No: 1409-3/4"-151R
Sch: 10S
Mat'l: SA-312-TP3041
Valve wt: 50 lbs. (1"-600-239)
10 lbs. (3/4"-600-153)
Press: 50 psig.



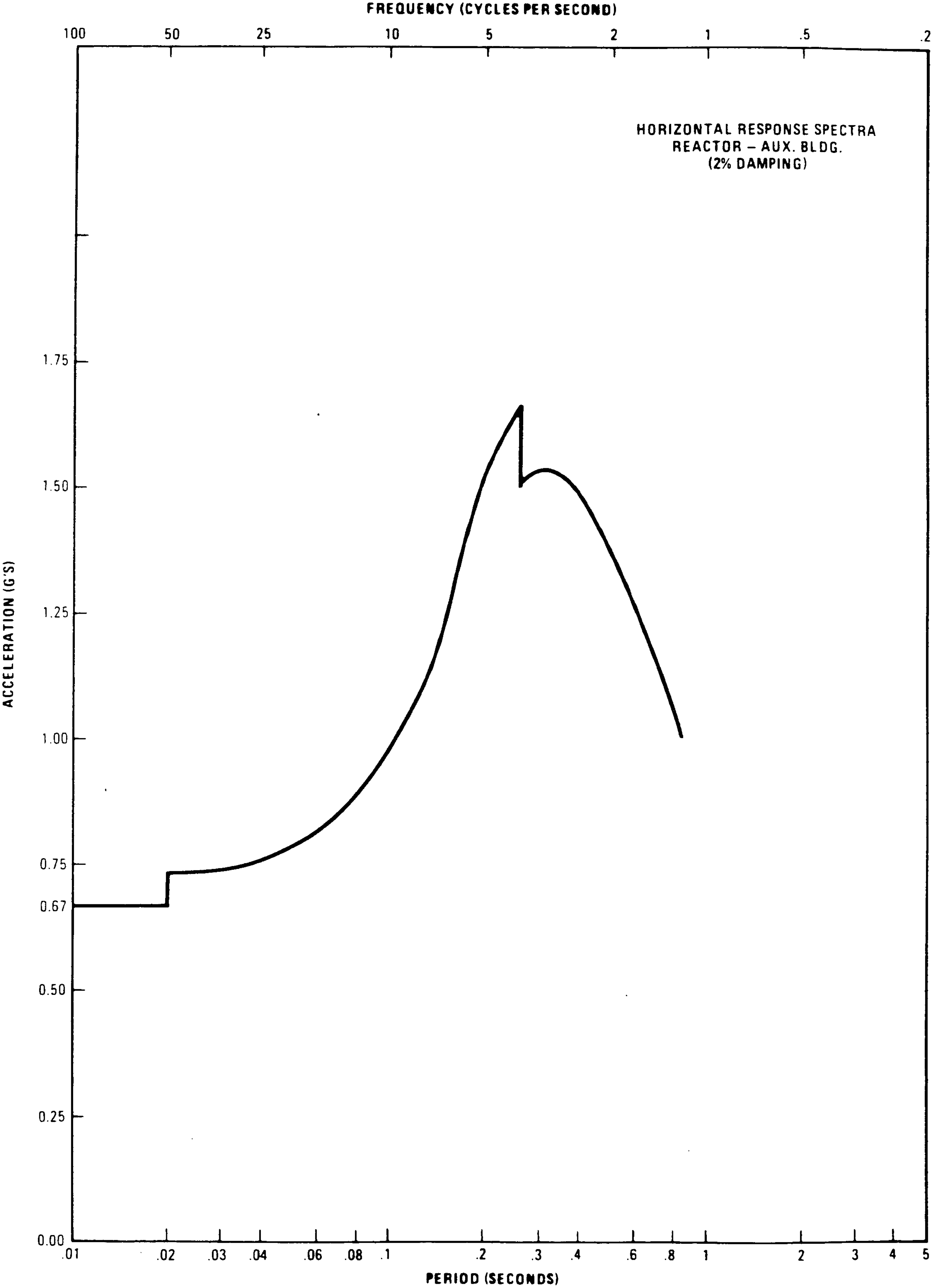
Support Type	Description
A1 & A2	Modified Support (.67g) BOPMEP Criteria
A1	Two Directional Strap
A2	Anchor

SONGS 1 Small Bore Pipe Example 3

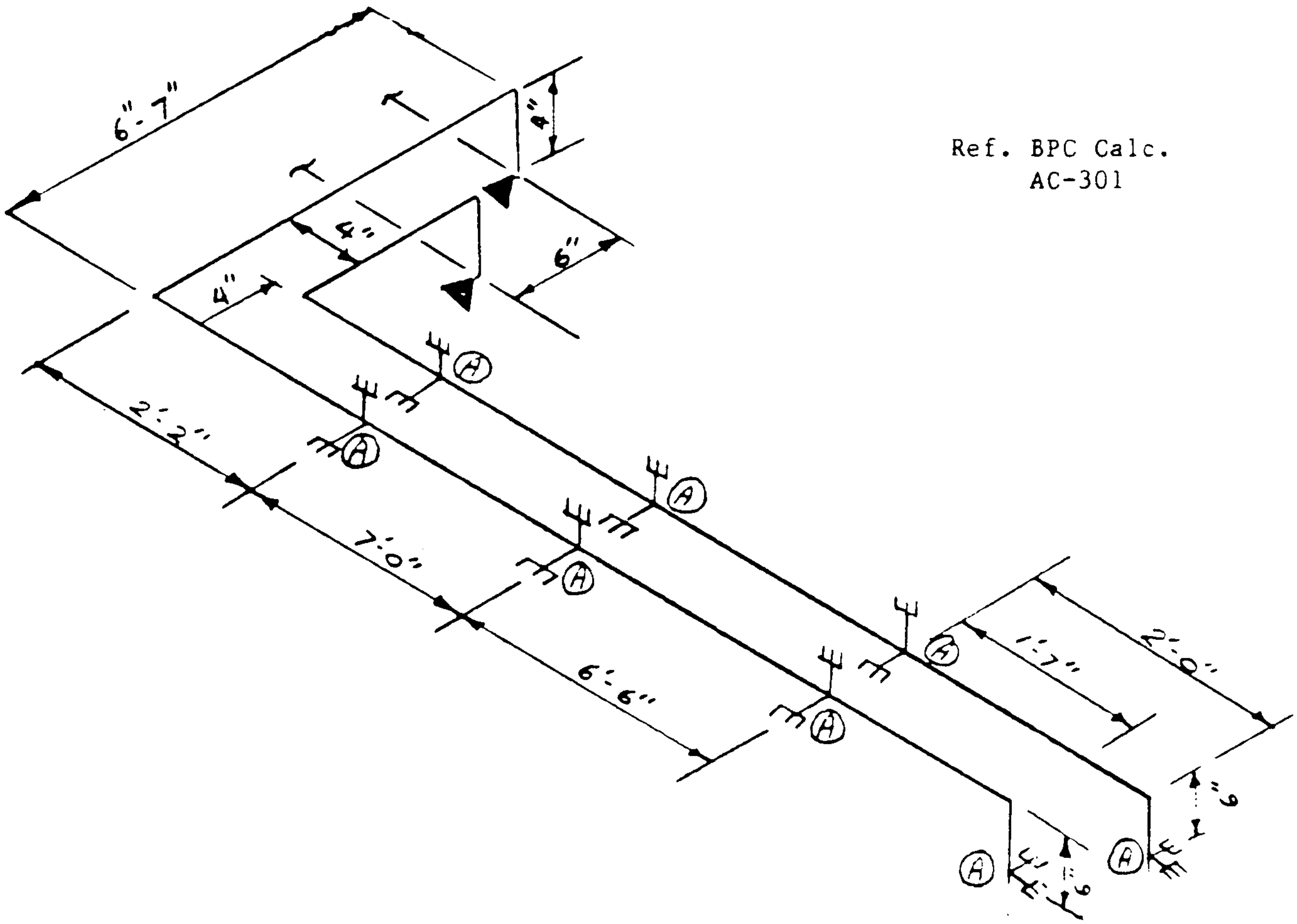
Lines 3120 and 3121 are 3/4" O.D., schedule 40 carbon steel lines carrying cooling water from auxiliary building header lines 3091-2" and 3105-2" to CV1 skid. Both lines have design temperatures of 150° F and design pressures of 80 psig.

The portions of lines 3120 and 3121 included in this example are considered supported at their respective 2" O.D. headers and are restrained throughout their runs by four 2-directional straps which have been modified to meet BOPMEP criteria. The maximum span between restraints is 9'-1". No valves or other concentrated masses are included in the portion of pipe in this example.

The maximum horizontal acceleration in the area of lines 3120 and 3121 is 1.663 G's at 4 cycles/sec at 2% damping.


FREQUENCY (CYCLES PER SECOND)
100
50
25
10
5
2
1
.5
.2
HORIZONTAL RESPONSE SPECTRA
REACTOR – AUX. BLDG.
(2% DAMPING)
ACCELERATION (G'S)
1.75
1.50
1.25
1.00
0.75
0.67
0.50
0.25
0.00
.01
.02
.03
.04
.06
.08
.1
.2
.3
.4
.6
.8
1
2
3
4
5
PERIOD (SECONDS)

EXAMPLE 3



Ref. BPC Calc.
AC-301

Line No: 3120-3/4"-HH, 3121-3/4"-HH
Sch: 40
Mat'l: A-53B
Press: 80 psig

Support Type	Description
A	Strap, Modified Support (0.67g) BOPMEP Criteria

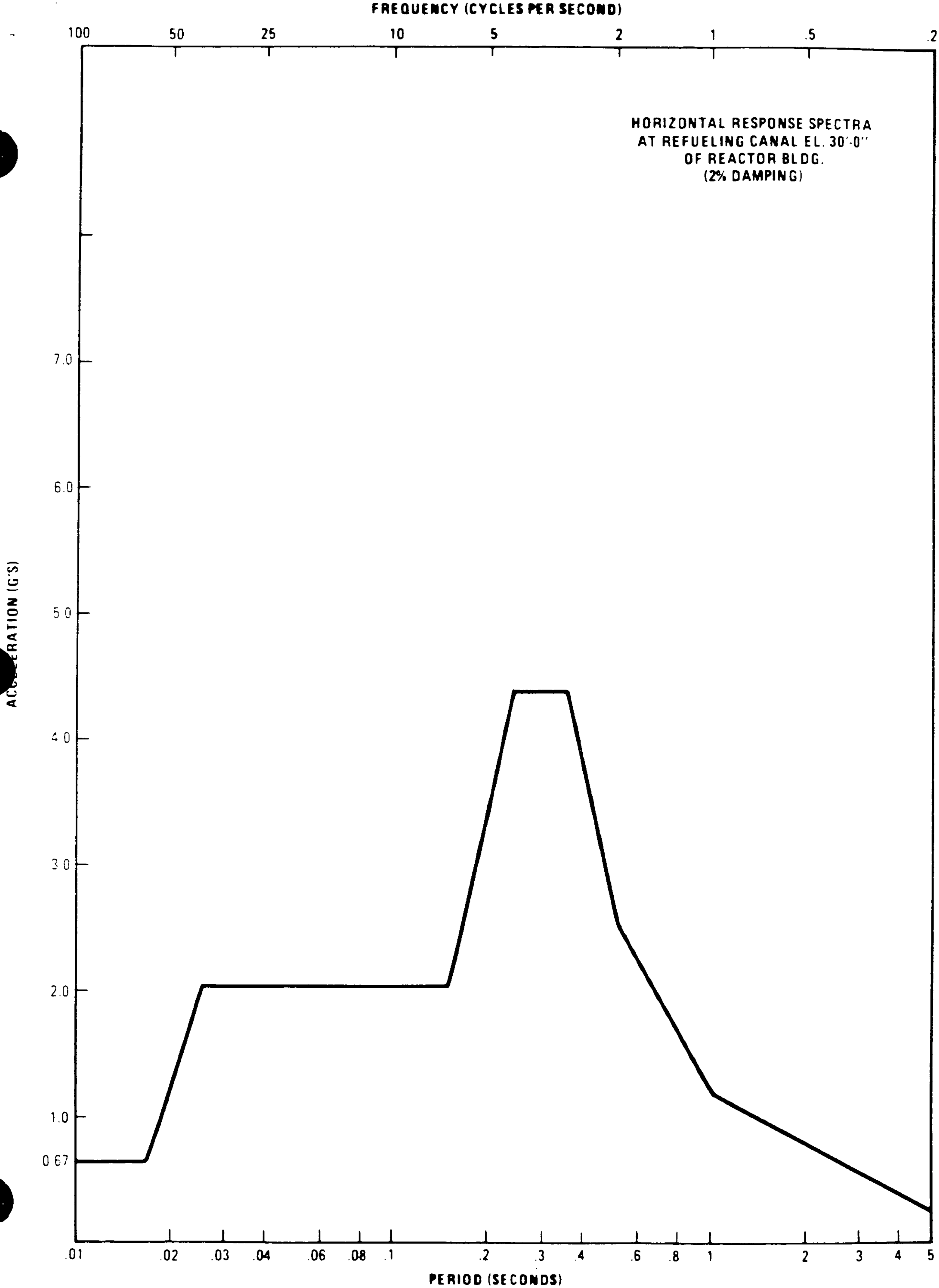
SONGS 1 Small Bore Pipe Example 4

Line 1208 is 3/4" O.D., schedule 40 stainless steel pipe carrying steam blowdown sampling. Line 1208 has a design temperature of 545° F and a design pressure of 1000 psig.

The portion of line 1208 considered in Example 4 is supported at one end by sphere penetration B-12 and along its length by two 2-directional guides, alternating with two U-bolts.

This section of line 1208 terminates with a 3-directional support which serves to isolate it from the remainder of the line. The average span between supports is approximately 12'-2".

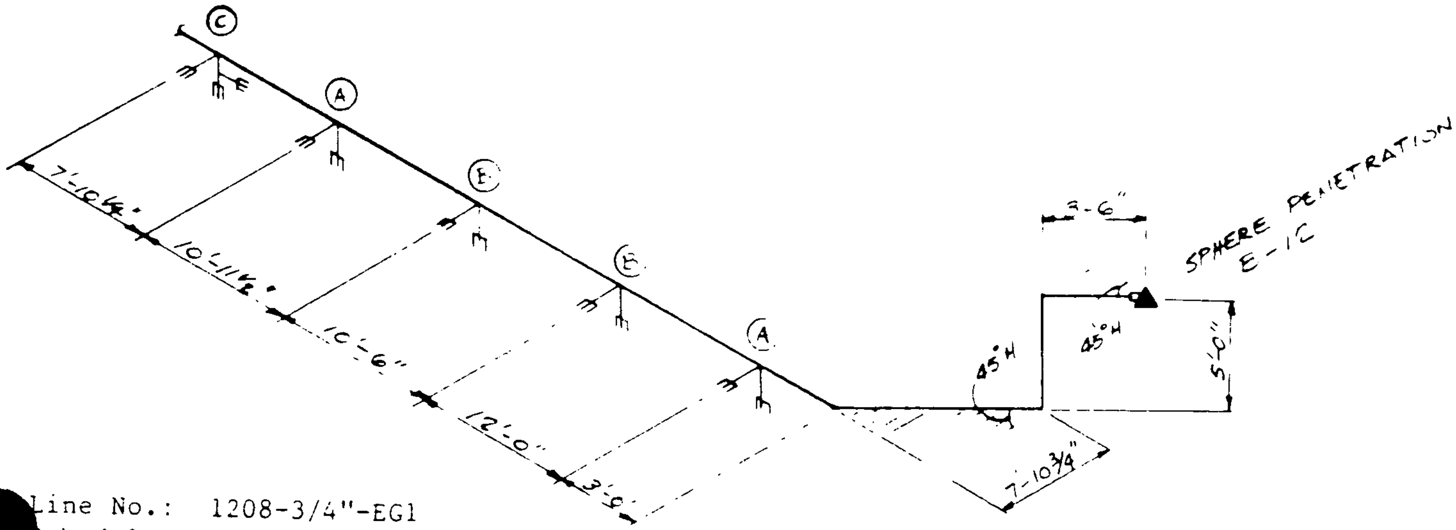
The response spectra enclosed for line 1208 envelopes the Refueling Canal Wall and Sphere Penetration. The maximum horizontal acceleration is 4.4 G's at 3-4 cps.



FREQUENCY (CYCLES PER SECOND)
100
50
25
10
5
2
1
.5
.2
HORIZONTAL RESPONSE SPECTRA
AT REFUELING CANAL EL. 30'-0"
OF REACTOR BLDG.
(2% DAMPING)
ACCELERATION (G'S)
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.67
.01
.02
.03
.04
.06
.08
.1
.2
.3
.4
.6
.8
1
2
3
4
5
PERIOD (SECONDS)

EXAMPLE 4

Reference: BPC Calc.
MS-358



Line No.: 1208-3/4"-EG1
Schedule: 40
Pressure: 1000 psig
Temp: 545°F
Mat'l: 312-TP304

Support Type	Description
A	Modified Support (0.67G) BOPMEP Criteria, Guide
B	U-Bolt
C	Modified Support (0.67G) BOPMEP Criteria, 3-way

3.2 EERC (UC Berkeley)

A series of tests were funded by EPRI to experimentally evaluate the effects of multiple support excitation. These tests were conducted on piping of 3" and 2" diameter excited by a support framing system to develop multi level inputs. A significant conclusion of this study was that "the results seem to argue for more flexible systems that connect piping systems to the structures housing them" (reference 6).

3.3 Hanford Engineering Development Laboratory

A series of tests were conducted on a 1" diameter piping system for the FFTF located in Richland, Washington. Various support configurations were tested to assess the response sensitivity to insulation and other nonlinear support characteristics. A significant conclusion of this test program was that the damping in piping systems is greatly increased if they are insulated (reference 7).

4.0 Performance of Small Bore Piping Systems in Actual Earthquakes

A formidable quantity of contemporary evidence is available, some of which has been collected in reference 1, showing that piping systems designed with controlled flexibility have the capacity to withstand forces far in excess of the forces for which they were designed. Reference 1 includes data collected from more than twenty power plants and industrial facilities which were subject to severe seismic motion. A typical example is the ESSO refinery in Managua, Nicaragua which was designed to meet provisions of the Uniform Building Code for a .2g seismic acceleration. During the 1972 Managua, Nicaragua earthquake, the peak acceleration measured at the refinery was .39g E-W and .34g N-S. Despite the fact the ground acceleration exceeded by nearly 100% the acceleration for which the systems were designed, virtually no damage was sustained by the piping systems. The plant was shut down for inspection but was operating at full capacity within 24 hours. Even more impressive evidence can be found at the ENALUF Power Plant which was subject to an estimated .6g ground motion during the same earthquake. This plant sustained no damage to its piping, despite a probably non-existant seismic design.

In addition to the survey presented in reference 1, a significant study was made of the response of the El Centro steam plant to the 1979 Imperial Valley earthquake by Murray, et. al. The results of this study were published in NUREG CR-1665. Significant conclusions of this study that relate to the piping are excerpted as follows:

(1) "No high-temperature or high-pressure piping failed during the earthquake."

(2) "General observations indicate that the piping systems are hung in a more flexible manner than that which would be required by current NRC criteria."

(3) "In most cases, the piping is supported in a similar manner to older operating nuclear power plants, and it may be inferred that the seismic response would be similar. These observations are, on the surface, encouraging since in all cases the circumstances leading to failure are dissimilar to nuclear applications in that damage occurred at weld repaired areas of past corrosive attack or at nonwelded pipe joints."

The evidence of earthquake experience clearly indicates that piping systems that are well laid out according to industry practice have an inherent resilience that permits them to withstand substantially greater seismic inputs than would be indicated by current design practice.

5.0 References

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