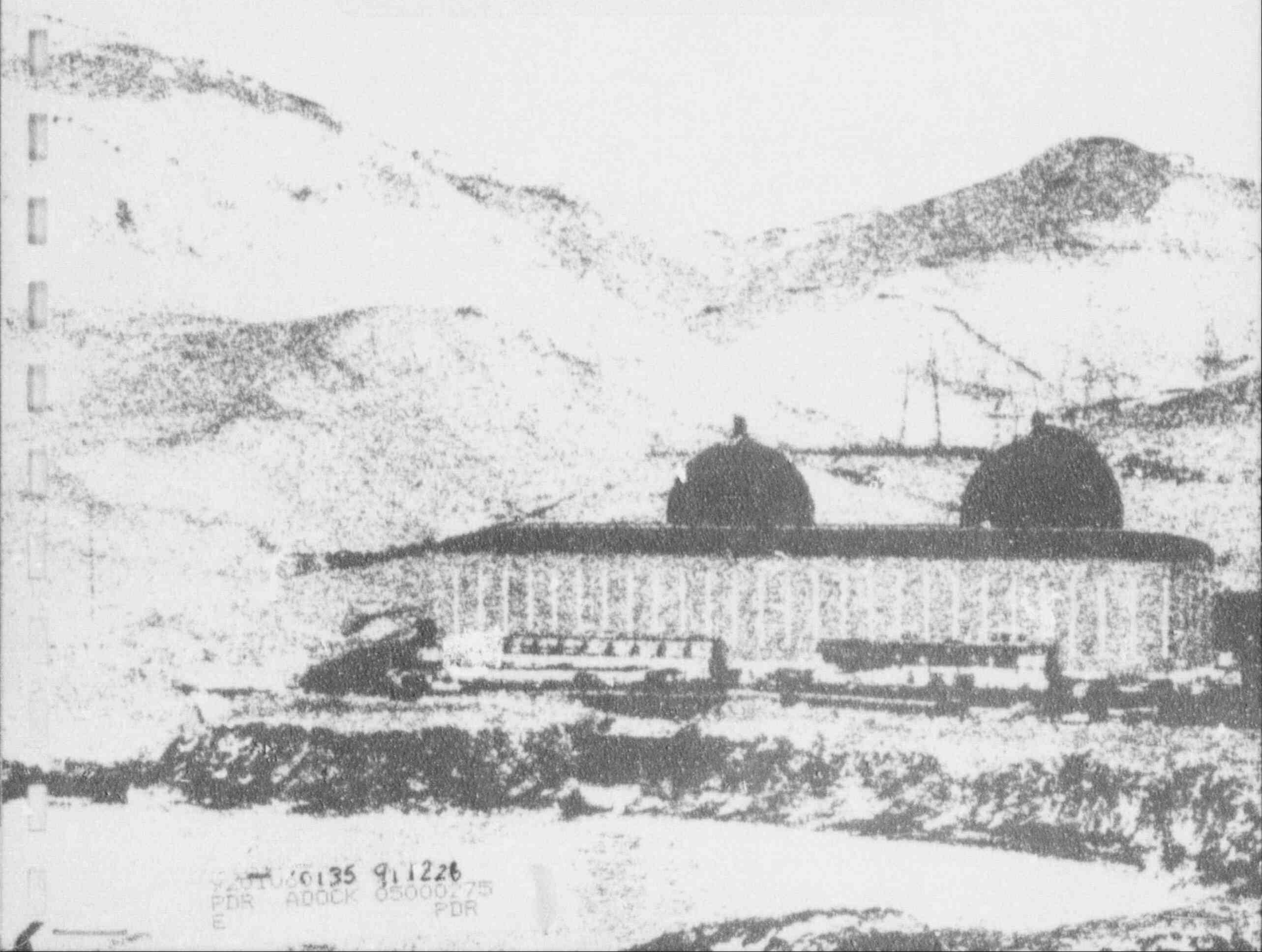
DIABLO CANYON
LONG TERM SEISMIC PROGRAM

PACIFIC GAS AND ELECTRIC COMPANY

Diablo Canyon Power Plant
Docket Nos. 50-275 and 50-323



ADEQUACY OF SEISMIC MARGINS ASSUMING AN INCREASE IN AMPLITUDE OF THE DIABLO CANYON LONG TERM SEISMIC PROGRAM HORIZONTAL AND VERTICAL GROUND MOTIONS AS DESCRIBED IN SSER 34

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This volume responds to confirmatory analysis asked of PG&E by the U.S. Nuclear Regulatory Commission (NRC) staff in their Safety Evaluation Report, Supplement No. 34, dated June 1991, to confirm PG&E's conclusions that the plant seismic margins are adequate to accommodate the horizontal and vertical spectral exceedances that result from use of the staff's estimates of horizontal and vertical spectra.

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At the request of the U. S. Nuclear Regulatory Commission (NRC) Staff, PG&E submitted discussions of the effects on the Diablo Canyon Power Plant structures and equipment of an assumed increase in amplitudes of the Diablo Canyon Long Term Seismic Program (LTSP) 84th-percentile site-specific ground motions (PG&E, 1988) in the horizontal and vertical directions at certain discrete frequency ranges (PG&E, 1991 and PG&E, 1991a). The NRC Staff reviewed these submittals and accepted PG&E's conclusions that the plant seismic margins remained adequate to accommodate the postulated ground motion exceedances. However, in the Supplemental Safety Evaluation Report (SSER) No. 34, the NRC Staff required that PG&E perform analyses to confirm that the seismic margins are adequate to accommodate the Staff's estimate of spectral acceleration levels. We have performed the analytical evaluations; a summary of the results is provided below. Note that the evaluations consider simultaneous effects of horizontal and vertical components of earthquake motions.

INCREASE IN HORIZONTAL SPECTRAL AMPLITUDE

Figure 1 shows the 5 percent damped free-field horizontal ground motions as postulated by the NRC Staff in SSER 34 and as developed in PG&E, 1988. Both the median and the 84th-percentile nonexceedance probability spectra are shown. In both the SSER 34 and the PG&E, 1988, only the 84th-percentile site-specific ground motion was considered in the evaluation of plant seismic margins.

As shown in Figure 1, the PG&E, 1988 spectrum completely envelops the SSER 34 spectrum at frequencies greater than 1 hertz. However, below 1 hertz, the SSER 34 spectrum shows an increase in spectral amplitudes of about 10 to 20 percent over the LTSP spectrum. As stated in PG&E, 1991, no essential equipment or components have fundamental frequencies lower than 1 hertz. Therefore, an increase in the ground motion spectrum in the range of 1 hertz and below would have no impact on the seismic margins of essential equipment and components.

Likewise, none of the essential building structures or structural elements are in the frequency range below 1 hertz. However, the sloshing modes of outdoor water storage tanks have low frequency responses (in the range of 0.2 to 0.4 hertz), which caused PG&E to examine what effect, if any, could be expected by the increase in spectral amplitude. The refueling water storage tank, which is a typical example of outdoor water storage tanks, was analyzed to examine the effect on seismic margin of the SSER 34 spectral amplitude increases.

Refueling Water Storage Tank

The refueling water storage tank has a relatively high median capacity of 9.92 g* (Table 6-23 of PG&E, 1988), governed by overturning moment at the concrete/bedrock interface. The base overturning moment of tank structures results from the combined effects from two modes; the impulsive mode, which includes the tank inertia and the part of the fluid that is moving in unison with the tank, and the sloshing mode. The impulsive mode typically has a much higher frequency (7.6 hertz) response and contributes significantly more to the overall demand than does the sloshing mode. A horizontal spectral amplitude increase in the range of 1 hertz and below has no effect on the dominant impulsive mode.

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* Capacities of plant structures and equipment are expressed in terms of site-specific horizontal spectral acceleration averaged over 3 to 8.5 hertz frequency range ($\bar{S}_{a\ 3\ \text{to}\ 8.5\ \text{hertz}}$)

Revised fragility estimates for the refueling water storage tank were made by considering the increased effect due to the tank sloshing mode. These revised fragilities are compared with the original estimates (PG&E, 1988) in Table 1. There is a small reduction in the High-Confidence-of-Low-Probability-of-Failure (HCLPF) value; however, the seismic margin remains high and the overall plant margin is therefore not affected.

Table 1

COMPARISON OF FRAGILITY ESTIMATES

Refueling Water Storage Tank

	Median $\bar{S}_a$	$HCLPF_{50}$	$HCLPF_{84}$	Scale Factor[1]
LTSP 1988	9.92 g	3.40 g	4.08 g	2.10
SSER 34	9.54 g	3.27 g	3.92 g	2.02

Note:

[1]Demand $\bar{S}_{a\ 3\ to\ 8.5\ hertz} = 1.94$ g

INCREASE IN VERTICAL SPECTRAL AMPLITUDE

Figure 2 shows the 5 percent damped free-field vertical ground motion developed by PG&E in PG&E, 1988. Both the median and the 84th-percentile nonexceedance probability spectra are shown. The NRC Staff postulated a vertical ground motion that, for the 84th-percentile spectrum, exceeds the PG&E, 1988 spectrum in the frequency range 1 to 10 hertz by 15 percent. The NRC Staff spectrum (SSER 34) is shown as a broken line in Figure 2 and extends between the frequencies of 1 and 10 hertz.

EQUIPMENT AND COMPONENTS

There are two factors that determine which essential equipment and components are affected by the vertical spectral increases First, components having vertical natural frequencies in the 1 to 10 hertz range, and, second, components supported at locations where the structural floor slab vertical frequency falls within the 1 to 10 hertz range. Based on this component screening criteria, the following nine components were identified for assessment:

- NSSS Piping
- Main Steam PORV
- Diesel Generator Fuel Oil Day Tank
- 4.16 Kv Switchgear
- 4.16 Kv Potential Transformer
- Safeguard Relay Panel
- Impulse Lines
- Balance of Plant Piping and Supports
- Conduits, Cable Trays, and Supports

A summary discussion of analytical evaluations of the effect of the SSER 34 spectral amplitude increase on these components is given below:

NSSS Piping

The fragility evaluation of NSSS piping established that the median spectral acceleration capacity was "high", i.e., greater than 10 g (PG&E, 1988). The critical seismic stresses in NSSS piping are due to bending moment in the pipes associated with loadings from the attached equipment components (the steam generator and the reactor coolant pump) resulting from seismic effects in the horizontal direction. The vertical natural frequencies of the steam generator and the reactor coolant pump are 23 hertz and 20.6 hertz, respectively, which are well beyond the frequency range where the spectral increases are identified. Therefore, a 15 percent increase of the PG&E, 1988 vertical ground spectrum in the 1 to 10 hertz frequency range does not impact the fragility of the NSSS piping.

Main Steam Power Operated Relief Valves

Piping attached to the main steam power operated relief valves (PORVs) has vertical frequencies in the 10 to 20 hertz range. Thus, vertical seismic loads generated from attached piping and transmitted to the PORVs are unlikely to be affected by spectral acceleration increases in SSER 34 . In addition, the main steam PORVs are mounted in a vertical position and are primarily sensitive to horizontal accelerations. Therefore, an increase in the PG&E, 1988 vertical ground spectrum in the 1 to 10 hertz frequency range does not affect the fragility of the main steam PORVs.

Diesel Generator Fuel Oil Day Tank

The fragility evaluation of the diesel generator fuel oil day tank showed the median spectral acceleration capacity greater than 10 g (PG&E, 1988). The critical failure mode is associated with bending of the tank flat bottom plate. The vertical frequency of the tank is 10.4 hertz and the critical failure mode is governed entirely by the vertical excitation. In the fragility evaluation, the seismic load was conservatively based on a vertical spectral acceleration at the peak frequency, further increased by a factor of 1.5 to allow for variations in the frequency of the tank due to the depth of the fuel oil and possible higher modes. While an increase in the vertical spectrum in the 1 to 10 hertz range does increase the seismic stress, the median spectral acceleration capacity of the tank still remains higher than 10 g. Therefore, we conclude that a 15 percent increase over the PG&E, 1988 vertical ground spectrum in the 1 to 10 hertz range has an insignificant effect on the seismic capability of the diesel generator fuel oil day tank.

4.16 Kv Switchgear

The vertical frequencies of 4.16 Kv switchgear are greater than 33 hertz for the structural failure mode and between 19 and 21 hertz for the functional failure mode. The switchgears are, however, located at turbine building elevation 119 feet, where the flexible floor slab has a fundamental vertical frequency in the 7 to 8 hertz range. Thus, an increase in the vertical ground spectrum in the 1 to 10 hertz range affects the switchgear through the local floor response. The structural failure mode of the switchgear corresponds to a bending failure of the switchgear carriage rod, which is controlled only by the horizontal response and which has a negligible contribution from the vertical excitation. Therefore, the structural failure mode is not affected by any increase in spectral accelerations of the vertical ground spectrum.

On the other hand, the functional failure mode, associated with chatter of the General Electric IAC 53 protective relay, is attributable to the vertical excitation alone. The functional failure mode fragility capacity factor was evaluated by comparing the Generic Equipment Ruggedness Spectra for the IAC 53 with the demand vertical spectral acceleration obtained at the location of the relay.

The functional failure mode fragility for the 4.16 Kv switchgear, as revised to include the effect due to the SSER 34 vertical ground spectrum, is compared below with the fragility value previously reported in PG&E, 1988.

LTSP 1988		SSER 34	
Median $\bar{S}_a$	= 3.53 g	Median $\bar{S}_a$	= 3.06 g
β_R	= 0.35	β_R	= 0.35
β_U	= 0.25	β_U	= 0.25
$HCLPF_{50}$	= 1.31 g	$HCLPF_{50}$	= 1.14 g
$HCLPF_{84}$	= 1.57 g	$HCLPF_{84}$	= 1.37 g

We note that the functional failure mode capacity is reduced as a result of an increase of 15 percent in spectral accelerations of the PG&E, 1988 vertical ground response spectrum. However, since the functional failure of the 4.16 Kv switchgear is recoverable by operator action, the plant seismic margin remains unchanged. In the discussion of the turbine building floor system at elevation 119 feet later in this document, the effect of deflection of the floor on the functionality of the switchgear is described.

4.16 Kv Potential Transformers

Similar to the 4.16 Kv switchgear, the fundamental vertical frequencies of the 4.16 Kv potential transformers are also very high. This equipment is also supported on the turbine building floor slab at elevation 119 feet, which has a vertical frequency in the range of 7 to 8 hertz. As a result, the increase in the SSER 34 vertical ground spectrum over the PG&E, 1988 spectrum at the frequency of the floor slab would increase the floor slab response, which in turn would affect the equipment fragility. In the fragility evaluation, two configurations were included: Bus F, which has three cabinets on a single stand and Buses G and H, which have two cabinets on a single stand. The vertical frequency of the three-cabinet stand is 33.7 hertz, while the vertical frequency of the two-cabinet stand is greater than 50 hertz. For both configurations, the critical failure mode corresponded to failure of the fillet weld anchoring each support leg to the floor embedment plate, for which the contribution of the vertical earthquake component is relatively small. The revised fragility evaluation showed that the fragility capacity remained very high, i.e., above 10 g. Therefore, the increased vertical ground spectral accelerations do not affect the fragility value of the 4.16 Kv potential transformers reported in PG&E, 1988.

Safeguard Relay Panel

The safeguard relay panel is also supported on the turbine building elevation 119 feet floor slab, similar to the 4.16 Kv switchgear and the potential transformers. Thus, the increased vertical ground spectral accelerations affect the panel through the local structure response. In the fragility evaluation reported in PG&E, 1988, the critical failure mode is associated with failure of the anchor welds.

By incorporating the 15 percent increase in the vertical spectral acceleration, the seismic demand load is increased. However, since the vertical response of this component makes a small contribution to the critical stress, the net effect on the median spectral acceleration and HCLPF capacities is very small. A comparison of the fragility reported in PG&E, 1988 with the values obtained using the 15 percent increase in the vertical ground spectral accelerations is shown below:

LTSP 1988		SSER 34	
Median $\bar{S}_a$	= 10.76 g	Median $\bar{S}_a$	= 10.70 g
β_R	= 0.34	β_R	= 0.34
β_U	= 0.36	β_U	= 0.36
$HCLPF_{50}$	= 3.39 g	$HCLPF_{50}$	= 3.37 g
$HCLPF_{84}$	= 4.07 g	$HCLPF_{84}$	= 4.04 g

Impulse Lines

The potential failure of impulse lines is related to their excessive deflections as a result of interactions with equipment and components to which the lines are connected. Since the lines typically have small diameters, failure is not associated with inertial loads on the lines. The components in the vicinity of the impulse lines, which were included in the fragility evaluation, have median spectral acceleration capacities in the range of 7 g to 9 g. In addition, these components have vertical frequencies that are greater than 10 hertz and failure modes that have negligible contributions for the vertical excitation. Therefore, we conclude that since the failure of impulse lines is not controlled by their inertial loads and the nearby components are not sensitive to the vertical excitation, the fragility of impulse lines is not affected by vertical spectral accelerations exceeding the LTSP 84th percentile spectrum in the 1 to 10 hertz range.

Balance of Plant Piping and Supports

As discussed in PG&E, 1988, a generic fragility was developed for the balance of plant piping based on pipe support design specifications for allowable stresses corresponding to various loading conditions and failure modes. The critical failure mode is associated with failure of fillet welds in pipe supports. As discussed in PG&E, 1988, a frequency range of 6 to 20 hertz was judged to envelope the probable piping frequencies. In addition, in the fragility evaluation, it was found that the piping systems with a frequency of 8 hertz show the least relative factor of safety. Therefore, the fragility for the balance of plant piping is affected by the increase in the vertical ground spectral accelerations in the 1 to 10 hertz range.

By incorporating the increased vertical spectral accelerations, the median spectral acceleration capacity and the HCLPF value are reduced approximately 5 to 6 percent. The results reported in PG&E, 1988 are compared with evaluations for SSER 34 below.

LTSP 1988		SSER 34	
Median $\bar{S}_a$	= 11.03 g	Median $\bar{S}_a$	= 10.34 g
β_R	= 0.40	β_R	= 0.40
β_U	= 0.39	β_U	= 0.38
$HCLPF_{50}$	= 3.00 g	$HCLPF_{50}$	= 2.85 g
$HCLPF_{84}$	= 3.60 g	$HCLPF_{84}$	= 3.42 g

Cable Trays, Conduit, and Supports

As discussed in PG&E, 1988, flexible cable tray systems will not fail. For cable trays, conduit, and supports, a generic fragility was developed based on the seismic qualification analyses for typical cable tray supports located at various elevations in the auxiliary building and the containment building.

As shown in Table D-1 of Kipp, 1989, the contribution of the vertical excitation to the critical stress ranged up to 95 percent and the vertical frequencies ranged from 6 to 35 hertz. However, for each of the typical supports, the vertical frequency ranged from 18.9 hertz to greater than 33 hertz. The low frequencies reported in PG&E, 1988 correspond to vertical modes of the cable trays themselves rather than the supports; the important frequencies for the cable tray supports fall outside of the 1 to 10 hertz range. While some vertical frequencies of the cable trays may fall into the 1 to 10 hertz range, the trays themselves are expected to perform well as demonstrated by previous tests on cable tray systems, which stayed intact and remained functional well above the design amplitude of motion (Linderman, 1981). Therefore, vertical ground spectral accelerations exceeding the LTSP 1988 spectrum in the 1 to 10 hertz range will have minor effect on the fragility values in PG&E, 1988 and Kipp, 1989, where it was reported that cable trays and supports have high capacity, i.e., greater than 10 g.

STRUCTURES AND STRUCTURAL ELEMENTS

The plant seismic margin assessment based on probabilistic methods described in PG&E, 1988, and later bench-marked by deterministic assessments in PG&E, 1990, established that shear walls are the governing elements that control seismic capacities of major civil structures. These walls are primarily horizontal earthquake resisting elements; also, the shear walls are rigid in the vertical direction having frequencies much greater than 10 hertz. We, therefore, conclude that there would be no effect of an increase in the LTSP 1988 vertical ground spectrum in the 1 to 10 hertz frequency range on the seismic capacity of major structures.

Vertical earthquake motions, however, may have some effect on parts of structures, such as flexible floors and floor beams. These vertical load carrying elements are usually controlled by ductile bending behavior, which is accompanied by a large inelastic energy absorption capability. Thus, these types of structural elements are highly unlikely to fail; however, it is necessary to examine supported equipment important to plant safety to see the effects of the attendant motions. Based upon these considerations, we have identified two vertically flexible slab systems for analysis to examine the effect of the SSER 34 spectrum. These are the control room roof slab located in the auxiliary building and the 4.16 Kv switchgear area floor slab located at elevation 119 feet in the turbine building.

Auxiliary Building Control Room Roof Slab

The auxiliary building has 11 vertically flexible slabs; however, the control room roof slab located at elevation 163 feet is the only slab that has a fundamental frequency in the vertical direction in the range of 1 to 10 hertz. Figures 3 and 4 show respectively the plan and section of the control room roof slab. As shown in Figure 3, the 3 feet-4 inch thick reinforced concrete slab is supported by embedded structural steel beams with a span of 57 feet and with end moment restraints provided by reinforced concrete shear walls. Attachments to the slab consist of light-weight lighting fixtures and ceiling tiles. These components are suspended from the underside of the slab via a grid of unistrut steel channels welded to insert plates embedded in the slab. The insert plates are anchored to the slab by steel flat bars that are welded to the bottom flange of the embedded steel beams. A few HVAC ducts are also attached to the slab by concrete expansion anchors.

The seismic margin evaluation of the slab was performed by using the conservative deterministic failure margin (CDFM) approach discussed in attachment DE-Q6A of PG&E, 1990. As described in PG&E, 1990, the seismic margin factor, which represents the amount by which the deterministic spectrum (in this case SSER 34) can be scaled to produce a demand equal to the HCLPF capacity of a structure or component, is given by the elastic scale factor times a factor F_μ. The elastic scale factor is the factor by which the deterministic spectrum can be scaled to produce a demand equal to the yield capacity of the structure or component, and F_μ is the CDFM inelastic energy absorption factor.

Progressive softening of the slab due to concrete cracking and development of plastic hinges at the end restraints of the slab together with formation of a plastic hinge near the midspan results in a fundamental frequency of the slab of about 5 hertz. The elastic scale factor was computed from the slab response just before a mechanism is formed. Capacities were based upon composite construction provisions of the Load and Resistance Factor Design Specification of the AISC.

The inelastic energy absorption factor was determined as a function of target displacement beyond yield displacement (i.e., ductility, μ) by using the Riddell-Newmark approach. For the control room roof slab, the ductility of the system is directly related to the ultimate displacement of the slab near midspan. The HCLPF value of the ultimate displacement of the slab is based upon structural capability considerations. The inelastic rotation capacity limit specified in the ACI 349 code is considered to be a HCLPF value.

For floors subjected to a significant gravity load, however, it is necessary to account for ratcheting behavior of the slab or beam when estimating the system ductility. Ratcheting refers to the progressive downward displacement that occurs following multiple seismic load reversals in the inelastic range and has the effect of reducing the available ductility of the system. To account for this behavior, a reduced effective ductility value was used in the inelastic energy absorption calculation. The process is described in Appendix A.

A sensitivity study was performed to investigate the variation of the seismic margin with displacement. The results are shown in Figure 5. We note that, at a seismic margin factor of 1.64 that corresponds to the auxiliary building seismic margin reported in PG&E, 1988, the displacement of the slab is only about 4 inches near the center of the 57-feet span. This displacement is less than about 0.6 percent of the slab span (i.e., span over 120) and corresponds to a member ductility of about 3. This can be compared to a ductility limit of about 9 determined from the rotation capacity specified in the ACI 349 code. At such small displacements, no significant degradation of the capability of anchored components is expected. We, therefore, conclude that the control room roof slab has an acceptable seismic margin over the SSER 34 vertical ground motion spectrum.

Turbine Building Floor System at Elevation 119 feet

The turbine building houses portions of three major safety-related systems: the emergency diesel generators, 4.16 kV switchgear, and the component cooling water heat exchangers. The emergency diesel generators and the component cooling heat exchangers are supported on the basemat. The diesel generator silencers at elevation 107 feet and the 4.16 kV switchgear at elevation 119 feet are located on floor systems that consist of reinforced concrete slabs supported by structural steel beams. Both floors have similar frequency responses in the vertical direction, with the fundamental mode in the 1 to 10 Hz range, and have seismic demands that are dominated by vertical ground motion. Of these two slabs, the 4.16 kV switchgear floor is located at a higher elevation, and hence has higher amplification of response. The switchgear floor also supports equipment that are more displacement sensitive, more massive, and is, therefore, subjected to greater seismic demands. Thus, the switchgear floor at elevation 119 feet was selected for evaluation.

The switchgear floor consists of a 10-inch thick reinforced concrete slab supported by compact wide flange structural steel beams and columns (Figures 6 and 7). Welded studs are used to anchor the slab to the steel beams. Thus, the concrete slab provides continuous lateral support to the compression flanges of the supporting beams. The switchgear panels are arranged in three rows that are separated by masonry walls. In addition to the building columns shown in Figure 7, the columns supporting masonry walls along column lines 3 and 4, from E.2 to G, and connected to the slabs at Elevations 119 feet and 140 feet by through bolts, also provide mechanisms for load transfer between the two floors. Note that the connections of columns along column lines 3 and 4 to the slabs at the top and the bottom, were made during the Hosgri reevaluation in order to connect the two floors, and thus, to reduce the response of the slab at elevation 119 feet.

The seismic margin evaluation was performed by using the (CDFM) approach. It was determined that the beams along column lines 2 and 3, from E.2 to G, that support masonry partition walls on each side of Bus "G" panels, were the controlling structural elements, whose failure was governed by bending (ductile behavior). Since the steel columns supporting the masonry wall along column line 2 are not connected rigidly to the floors above and below, the beam along column line 2 does not receive any support from the beam above. However, the floor beam along column line 3 acts in unison with the floor beam at elevation 140 feet; thus, the elastic scale factor for this combined system was calculated by considering the total seismic demand and their combined capacities.

The evaluation of the structural steel beams, and consequently the switchgear floor system, consisted of performing a sensitivity study wherein seismic margins (i.e., the scale factors by which the input vertical ground motion would have to be raised) corresponding to specific target displacements (or, pre-assigned ductilities) were calculated. The calculations accounted for the ratcheting effect due to dead load in accordance with the procedure described in Appendix A.

Figure 8 shows the results of the sensitivity study where the seismic margin factor is plotted against the ultimate target displacement of the floor. We observe that, for a seismic margin factor of 1.45, which is approximately the seismic margin of the turbine building reported in PG&E, 1990, the maximum displacement of the switchgear floor is about 1.7 inches. This displacement is equivalent to a member ductility of about 2 which can be compared to a value of 10, corresponding to a structural capability limit given by the AISC commentary to the Load and Resistance Factor Design specification. We have evaluated the switchgear cabinets for such a postulated displacement of the supporting floor of 1.7 inches. The evaluation is described below.

IMPACT OF TURBINE BUILDING FLOOR DEFLECTION ON 4.16 KV SWITCHGEAR

The 4.16 kV switchgear is an assembly of 4.16 kV magneblast circuit breakers, buses, insulators, wiring, control, and protecting and monitoring devices that are contained in a metal-clad enclosure. Power is made available to all load circuits through three electrically contiguous and high conductivity buses running through the entire length of the switchgear. At each cubicle a vertical tap is made for connection to the circuit breaker. The breaker is held in place by a cradle that prevents it from falling under severe conditions of vertical and horizontal seismic conditions.

The maximum displacement of 1.7 inches is reduced by the dead load displacement of 0.3 inch to obtain the differential displacement of 1.4 inches affecting the supported equipment. The 1.4 inches maximum sag of the switchgear floor, applicable to bus G, is equivalent to 0.2 inch dip of one edge of the 26-inch wide switchgear cubicle. A review and walkdown of the control and protective devices located in the switchgear cubicle, including relays, switches, fuses, terminal blocks and wiring, indicates that this deflection has no significant functional impact on these devices. Certain switchgear mounted relays could potentially trip the breaker due to chattering. Following the trip the breaker will stay open. Control room annunciation and indication of the breaker status will remain operational. After the event, realignment of the breakers can be made through operator control. Also, the copper buses have adequate flexibility so that small deflections will not affect the integrity of the bus and conductivity will be maintained.

The structural integrity of the 4.16 kV switchgear was evaluated for the combined horizontal and vertical seismic loads as well as the effect due to turbine building floor displacement. The results of the evaluation indicate that the switchgear will maintain its structural integrity with acceptable margin for loadings due to SSER 34 seismic spectra.

Note that each row of switchgear cabinets (see Figures 6 and 7) consist of several individual cabinets connected to one another in series by a line of bolts along their heights. These cabinets are thus structurally affected by the deflection of the supporting floor. In contrast, the other two equipment items, considered essential in LTSP risk assessment and margin studies, namely, the 4.16 kV potential transformer and the safeguard relay panel are individual units, and consequently, the effect of floor displacement on these equipment items is insignificant.

Fuel Handling Building Crane

In PG&E 1990, we reported results of the deterministic evaluation of the fuel handling building crane. As noted in that submittal, the crane was chosen for evaluation because its response is primarily governed by the vertical component of ground motion. The failure modes identified in PG&E 1990 lie within the frequency range from 1 to 10 hertz, and therefore, these failure modes are affected by the SSER 34 vertical spectrum.

We have reevaluated the fuel handling building crane for the SSER 34 spectra. The results are reported in Table 3, which shows that the fuel handling building crane continues to have an acceptable seismic margin over the SSER 34 vertical ground motion spectrum. Please note that, as stated in PG&E, 1990, the margins reported for the lifted case are very conservative because of the analysis methodology assumptions and low probability of a 125-ton lift concurrent with a significant seismic event.

Structural Steel Frames and Truss Systems - Pipeway Structure

The last class of structures that may potentially be affected by the vertical ground motion in the frequency range of 1 to 10 Hz are structural steel frame and truss systems. Effects of vertical seismic excitation on these types of structures are generally small compared to horizontal ground motion effects. Furthermore, steel framing systems are generally highly redundant and consist of elements that are usually controlled by ductile bending behavior, which is accompanied by large inelastic energy absorption capability.

We have selected the pipeway structure among the class of structures composed of structural steel framing and truss system for evaluation of increased vertical ground motion. In contrast to the structural steel roof truss system over the turbine building, certain safety-related systems are directly attached to the pipeway structure. For example, the pipeway structure provides support to the auxiliary feedwater pipe line as well as smaller piping and safety-related instrumentation tubing and conduits. The pipeway structure also supports the main steam and feedwater pipe lines between the containment and the auxiliary building. In addition, the pipeway holds pipe whip restraints designed to restrain pipes from whipping after a postulated pipe break.

The pipeway structure is a three-dimensional structural steel frame attached to the outside of the containment shell, the auxiliary building and the turbine building. Figure 9 shows locations of the Unit 1 and Unit 2 pipeway structures, which are basically similar to one another. The structure has five levels of plane frames with the main platform at elevation 109 feet (shown in Figure 10), and includes ten radial bents that cantilever out from the containment wall by horizontal beams and/or inclined beam columns. Connections between the pipeway structure and the auxiliary and the turbine buildings are provided with slotted holes oriented such that horizontal forces cannot be transmitted between the pipeway structure and these adjacent buil[illegible].

In a manner [illegible] [illegible]at applied to the control room roof slab and the 4.16 kV switchgear floor, the pipeway struct[illegible] seismic margin was evaluated by using the CDFM approach. Critical structural elements were identified for margin assessment on the basis of seismic capacity-to-demand ratios determined during the Hosgri reevaluation, as well as the importance of the members to the overall structural system. Capacities were determined by using the Load and Resistance Factor Design (LRFD) specification of the AISC. In some cases, CDFM capacities were taken to correspond to 1.7 times the AISC allowable stress, which is approximately at the first yield of a ductile member.

As noted previously, for ductile structural systems beyond the elastic yield limit, the seismic margin factor can be assessed by limiting the ultimate target displacement from consideration of functionality of supported systems and components. For the pipeway structure, however, we calculated the displacement corresponding to a preassigned seismic margin factor. Thus, to reach a seismic margin factor of 1.76, which is the seismic margin for balance of plant piping, beams along Bents 2B and 6.6B (see Figure 10) that become inelastic, displace vertically by up to about one inch. Long-span, flexible piping systems that are typically attached to the pipeway structure are expected to accommodate displacements of such small magnitude.

CONCLUSIONS

Based upon the confirmatory analysis described above, we conclude that the essential structures, equipment and components important to plant safety have adequate seismic margins to accommodate the NRC Staff's estimate of the horizontal and vertical ground motions shown in the SSER 34. We have also shown that the SSER 34 spectra have minor effects on the seismic fragility estimates of the structures and components. We can thus conclude that the assumed increase in amplitude of the Diablo Canyon LTSP 84th-percentile site-specific ground motions in the horizontal and vertical directions, shown in SSER 34, will not alter the conclusions of the probabilistic risk assessment (PG&E, 1988).

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Table 2

EFFECT OF INCREASE OF SPECTRAL ACCELERATIONS SHOWN IN SSER 34 FRAGILITIES OF AFFECTED EQUIPMENT AND COMPONENTS
(Based on Hazard Defined Over 3 to 8.5 Hertz Range)

EQUIPMENT/ COMPONENT	VERTICAL FREQUENCY (Hz)	SPECTRAL ACCELERATION CAPACITY							
		LTSP 1988				SSER 34			
		$\bar{S}_a$ (g)	β_R	β_U	HCLPF (g)	$\bar{S}_a$ (g)	β_R	β_U	HCLPF (g)
NSSS Piping	7-9	> 10.00				> 10.00			
Main Steam PORV	Piping 10-20	11.50	0.34	0.38	3.51	Unchanged			
Diesel Generator Fuel Oil Day Tank	10.4	> 10.00				> 10.00			
4.16 kV Switchgear*	> 33	3.53	0.35	0.25	1.31	3.06	0.35	0.25	1.14
4.16 kV Potential Transformer*	> 33	10.83	0.31	0.38	3.47	> 10.00			
Safeguard Relay Panel*	> 33	10.76	0.34	0.36	3.39	10.70	0.34	0.36	3.37
Impulse Lines	5-20	7.09	0.28	0.32	2.63	Unchanged			
Balance of Plant Piping and Supports	6-20	11.03	0.40	0.39	3.00	10.34	0.40	0.38	2.85
Conduits, Cable Trays, and Supports	6-35	> 10.00				> 10.00			

* Equipment supported on vertically flexible floor slab having vertical frequency less than 10 hertz.

Table 3

Fuel Handling Building Crane
Seismic Margin Factors

Crane Element or Failure Mode	Lifted Load (tons)	Seismic Margin Factors	
		LTSP 1988	SSER 34
Cable	125	2.90	2.52
Bridge Girder	125	1.55	1.35
Bridge Resistance to Global Uplift	0	3.2	2.7
Trolley Resistance to Uplift	0	2.4	2.0

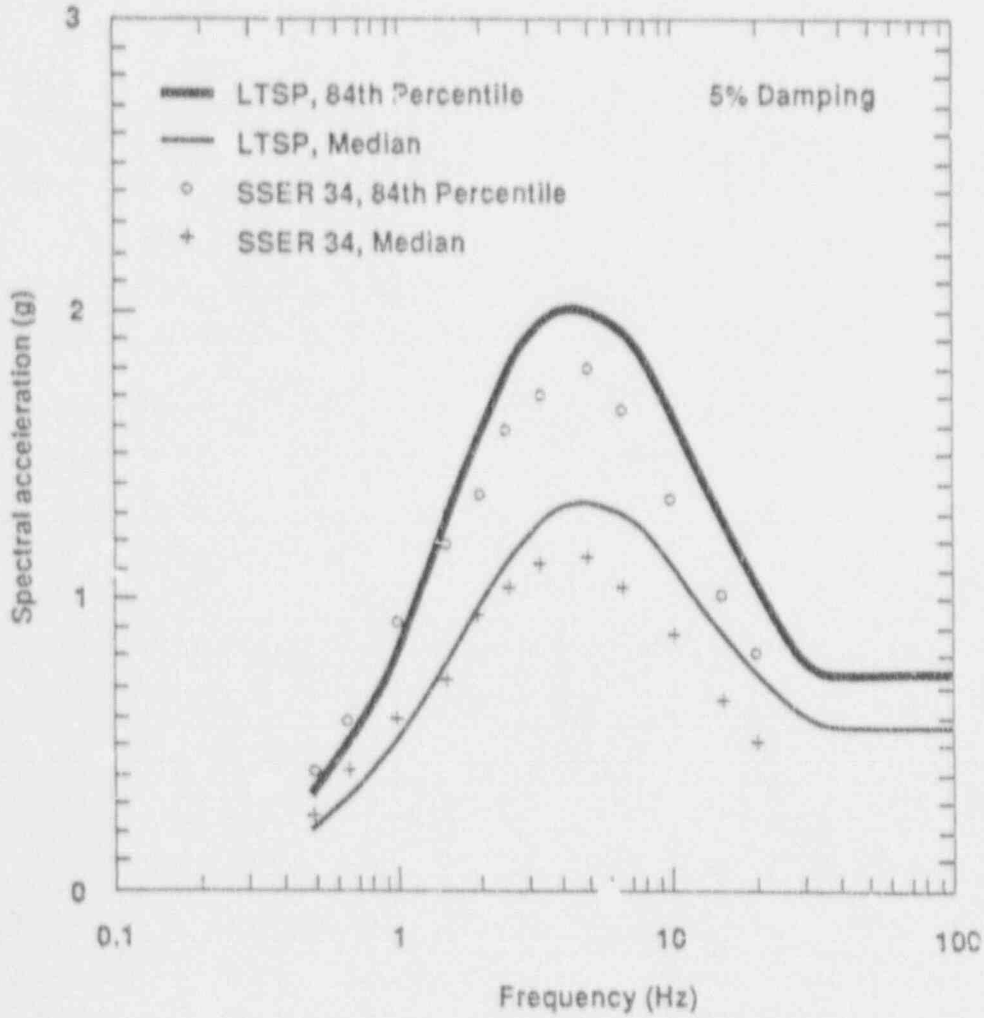

Figure 1

Site-speciifc horizontal ground motion response spectra.

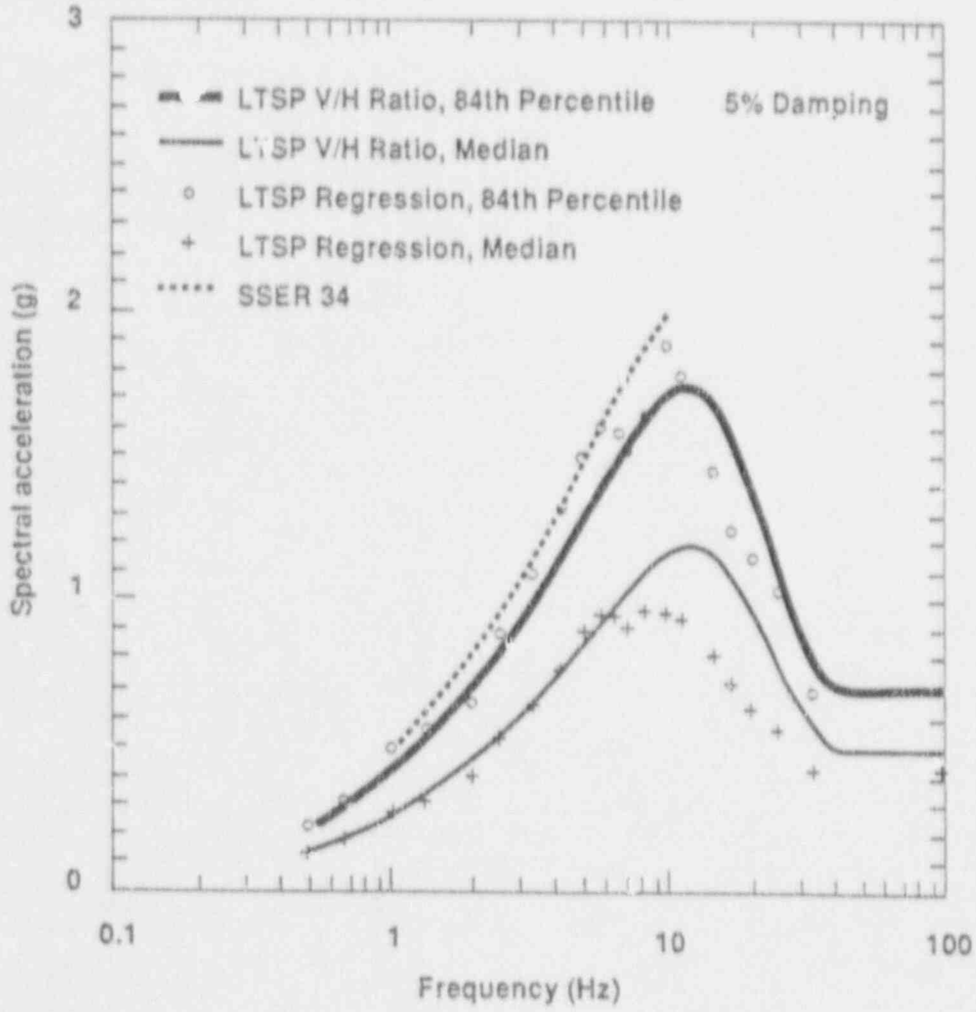

Figure 2

Site-specific vertical ground motion response spectra.

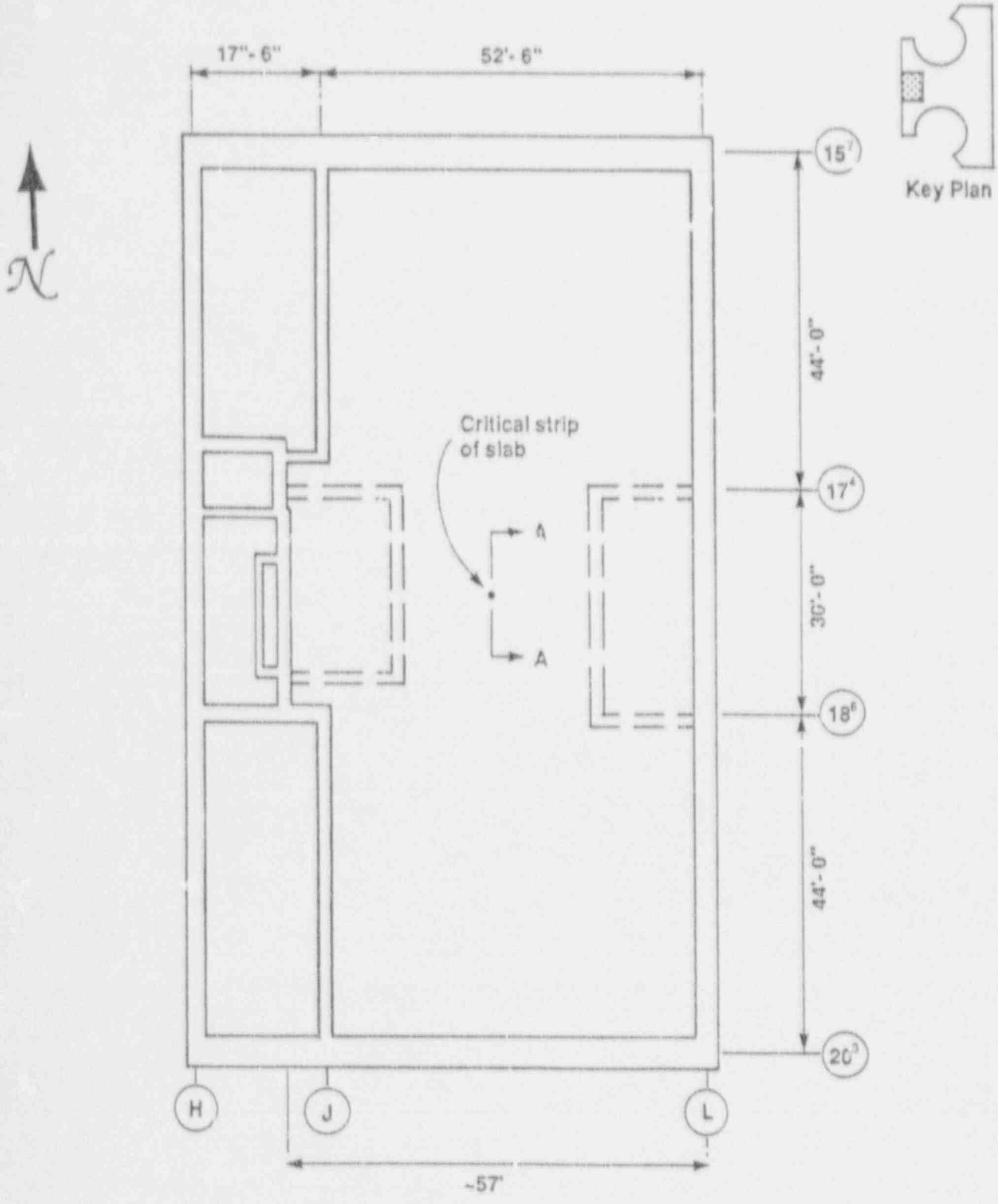

Figure 3

Auxiliary building control room roof slab.

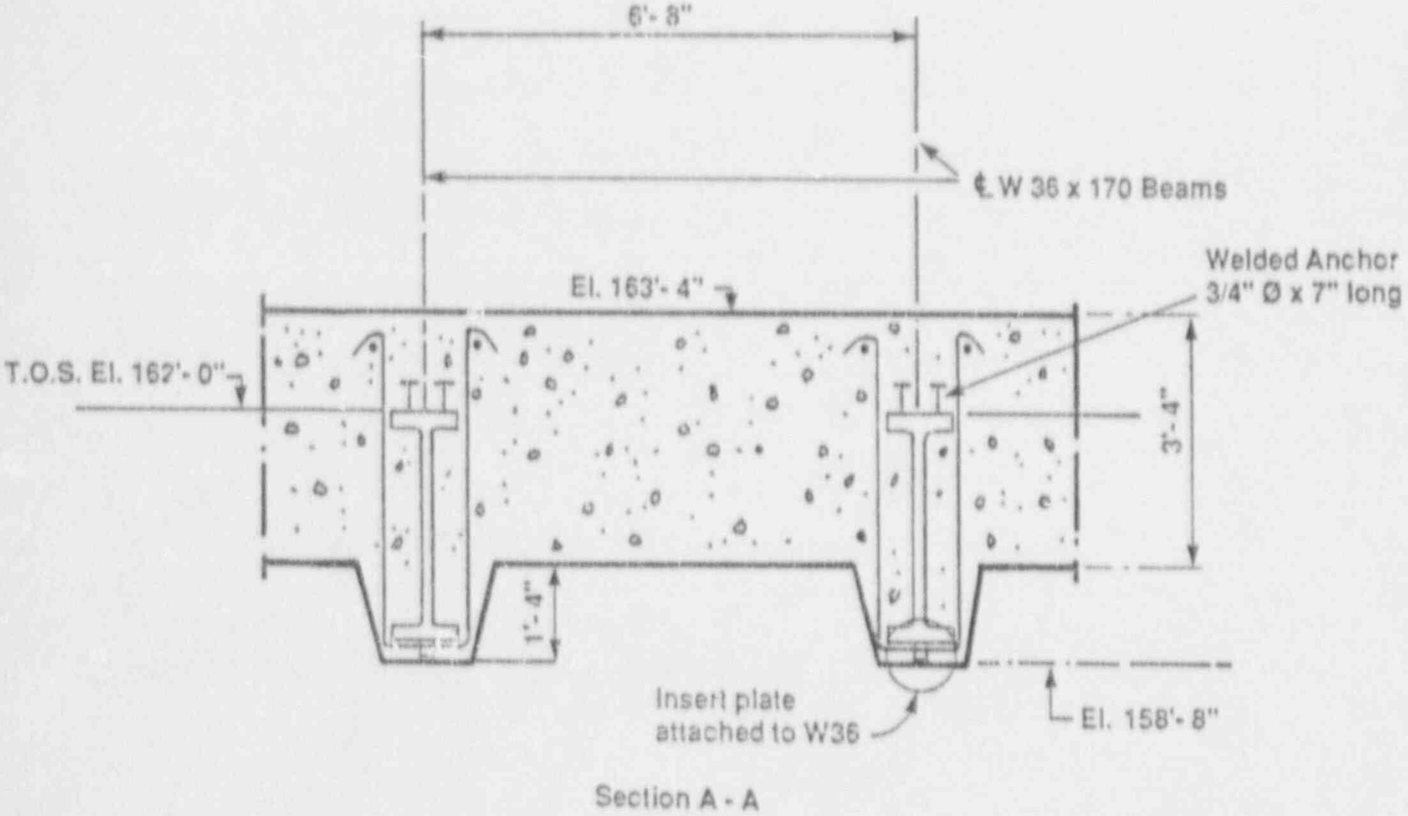

Figure 4

Auxiliary building control room roof slab. Section A - A

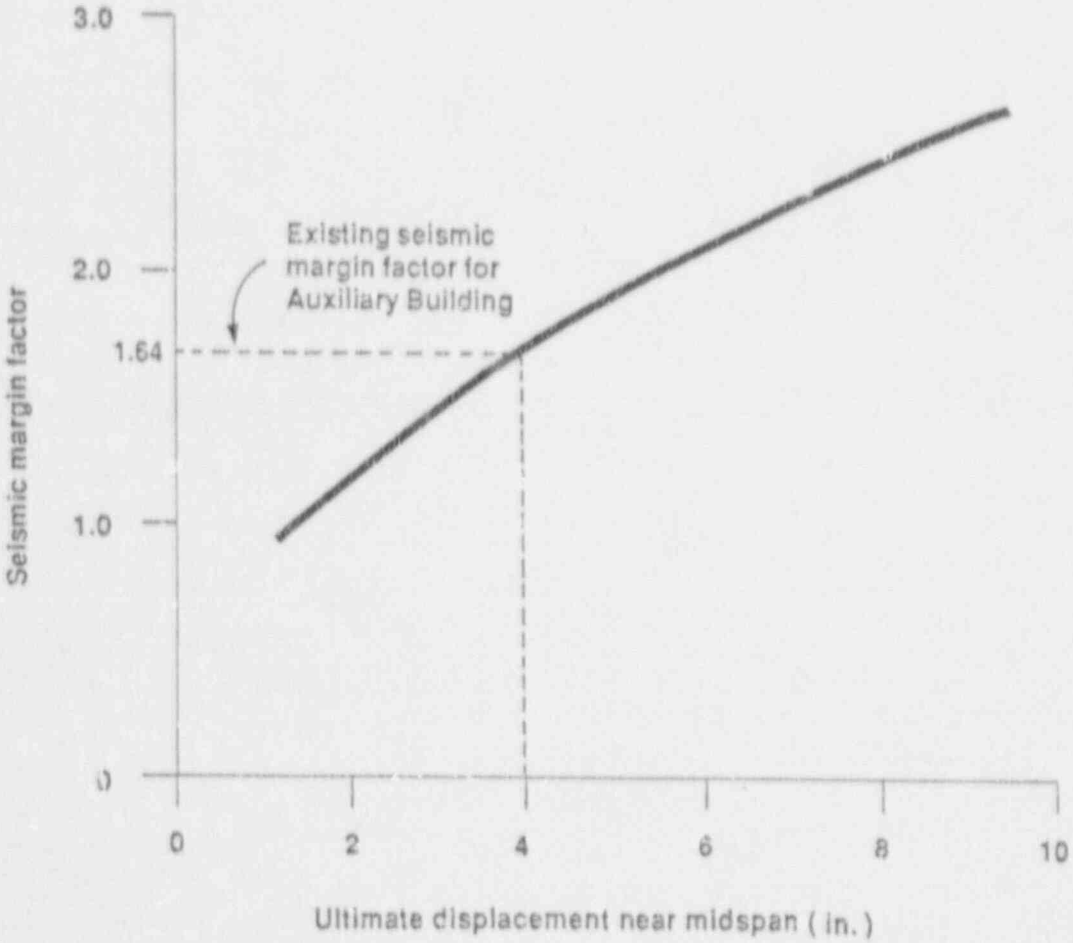

Figure 5

Ultimate displacement near midspan of control room roof slab vs. seismic margin factor.

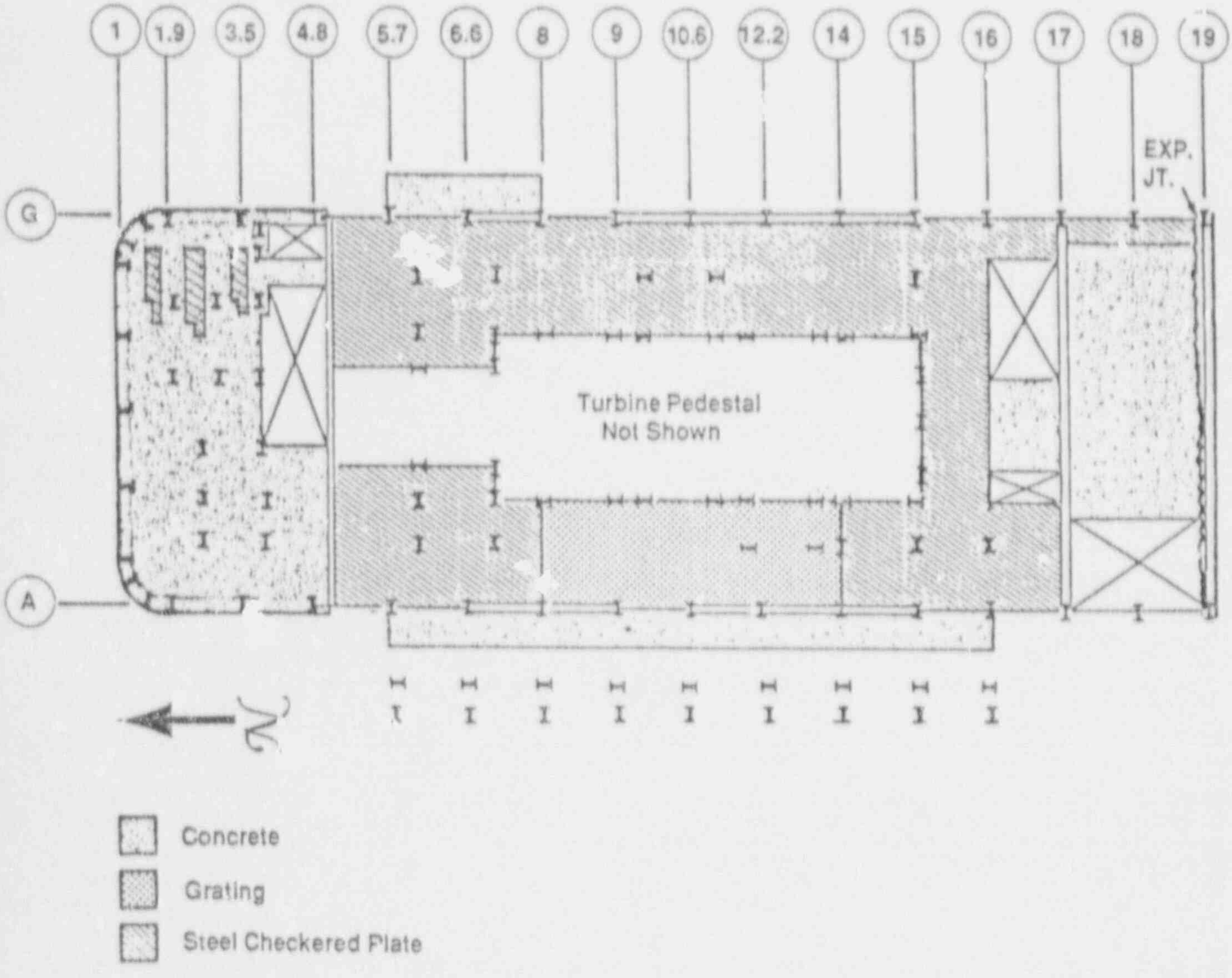

Figure 6

Unit 1 turbine biuilding plan at elevation 119 feet.

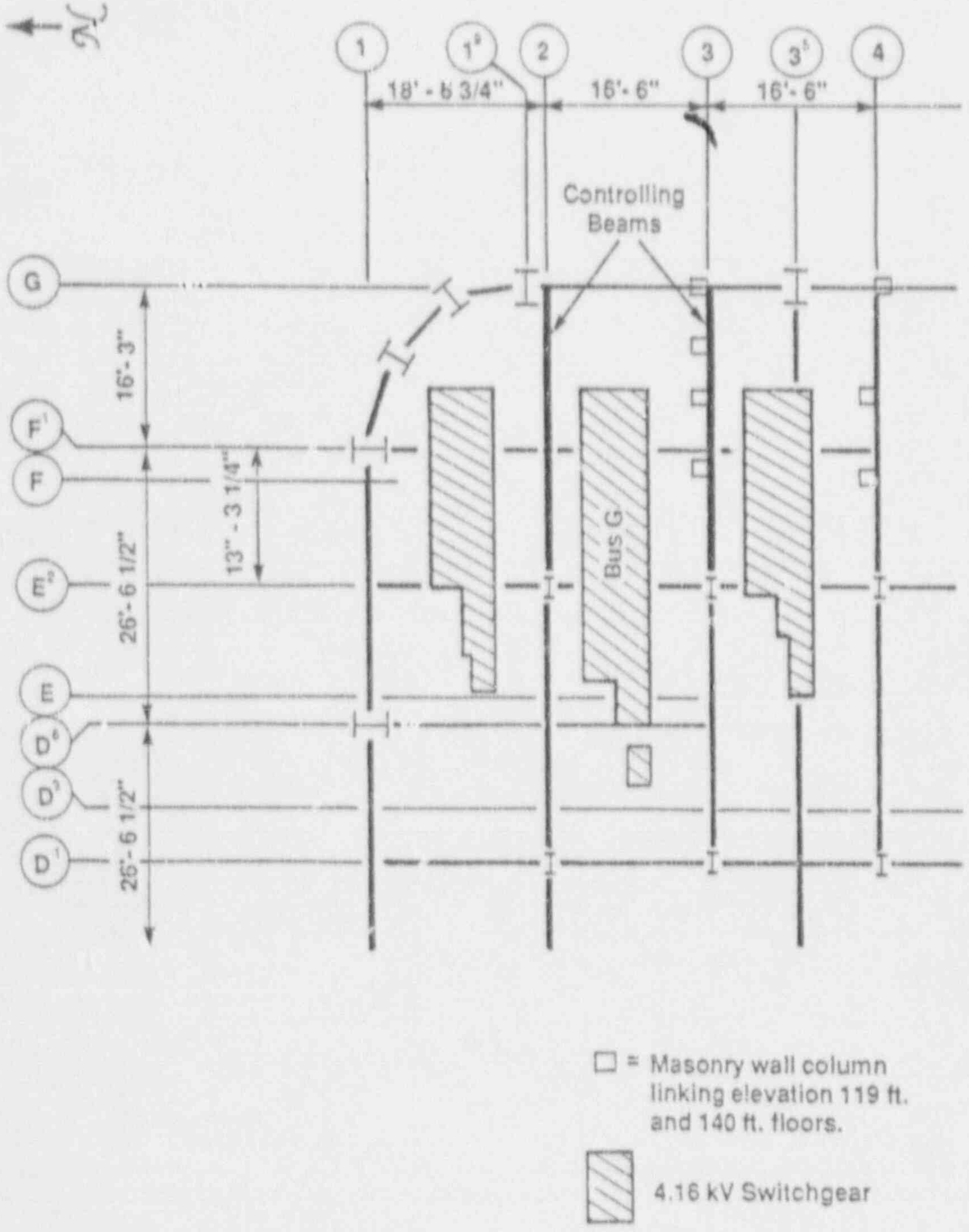

Figure 7

Turbine building 4kV switchgear floor partial plan.

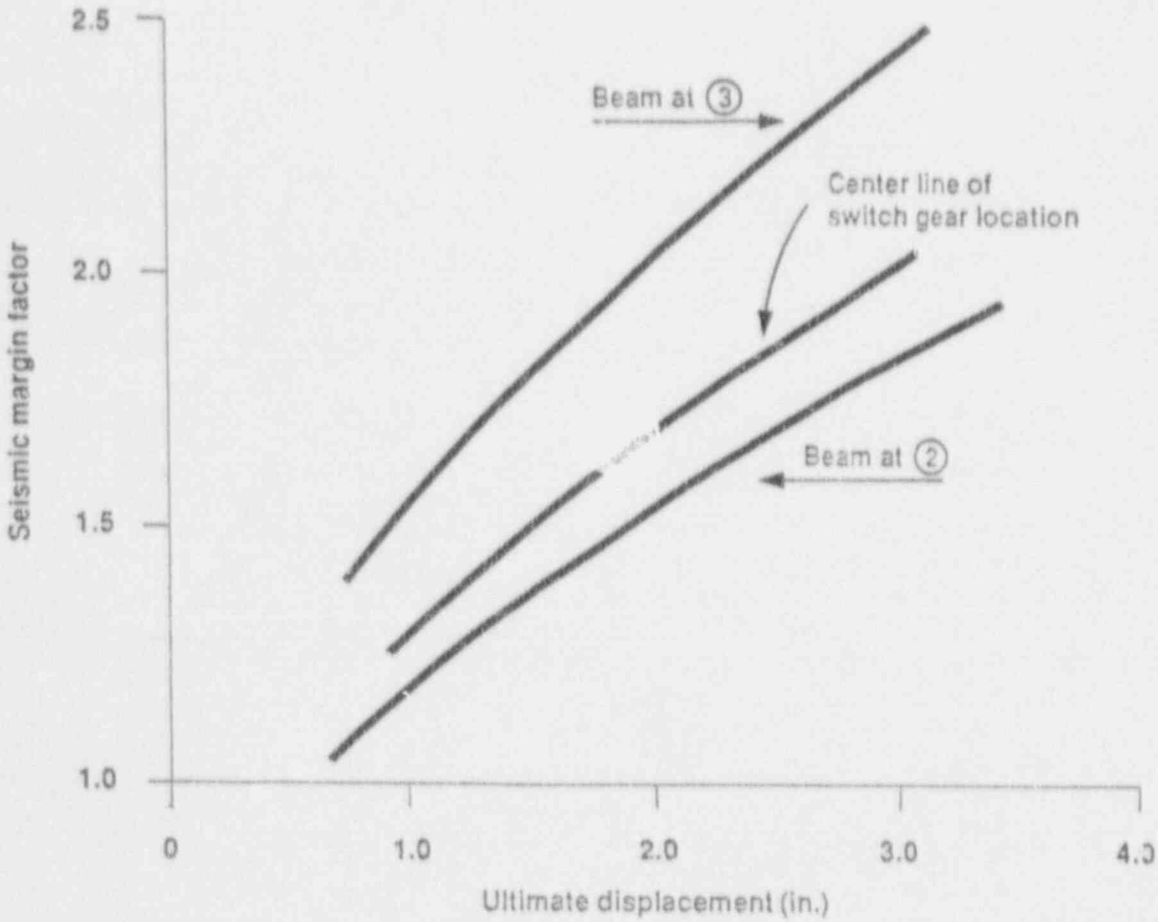

Figure 8

Ultimate displacement of floor beams vs. seismic margin factor.

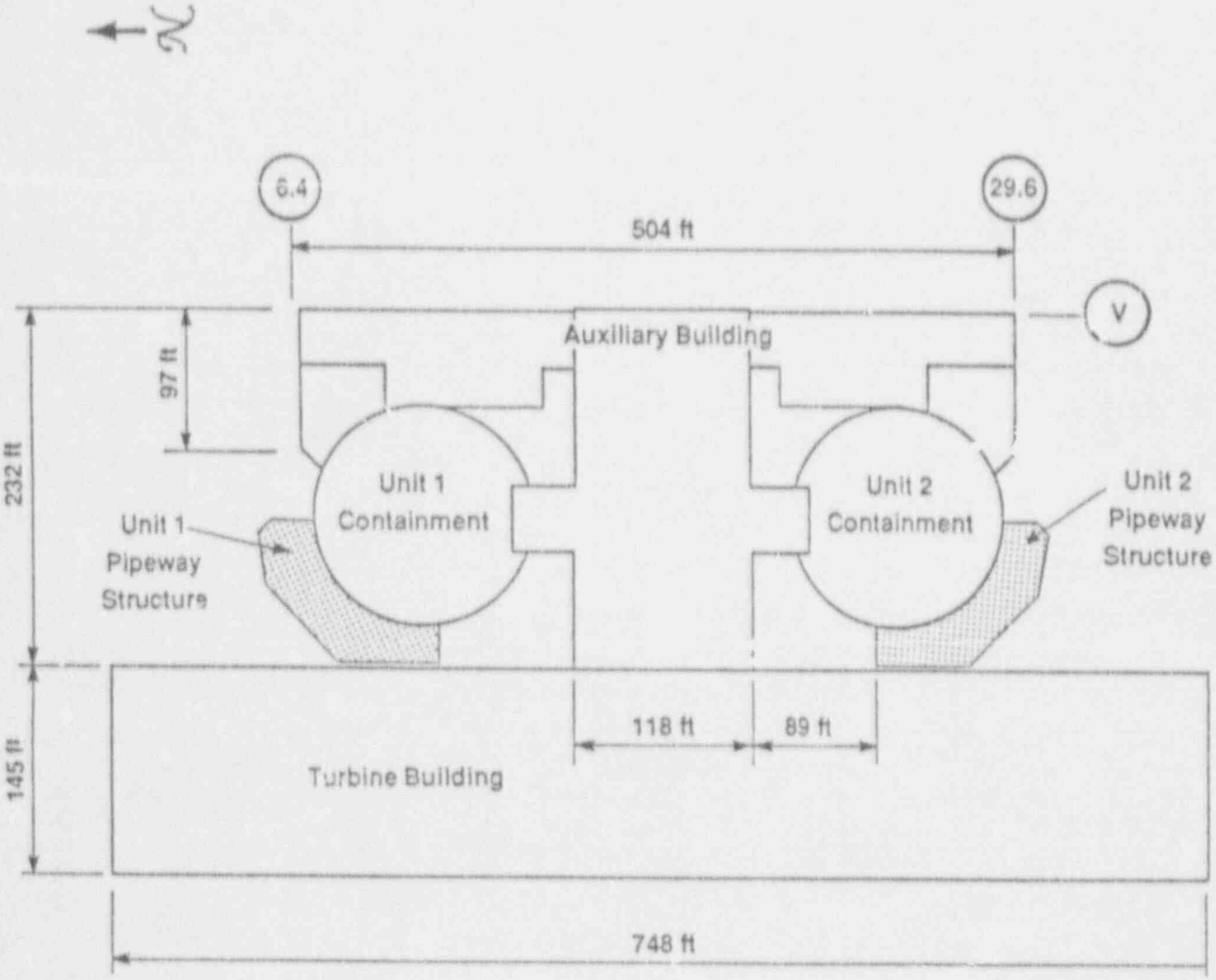

Figure 9

Power block structures with pipeway structures of units 1 and 2 highlighted.

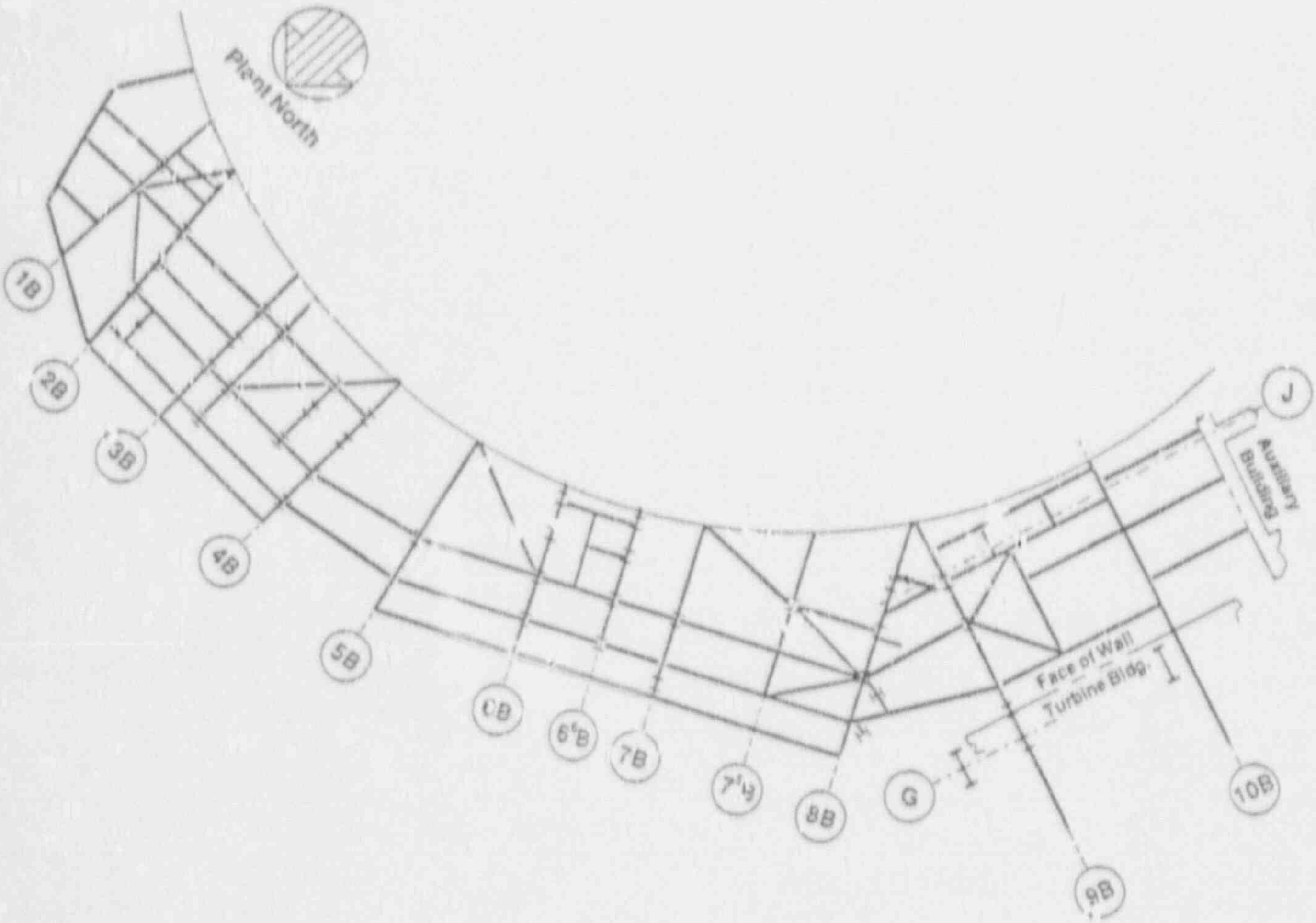

Figure 10

Unit 1 pipeway structure, platform at elevation 109 feet.

APPENDIX A

INVESTIGATION OF THE EFFECTS OF DEAD LOAD ON DUCTILITY OF FLOOR SYSTEM

December 1991

APPENDIX A

INVESTIGATION OF THE EFFECTS OF DEAD LOAD ON DUCTILITY OF FLOOR SYSTEM

Introduction

This report presents results of a study to investigate the effects of dead load on the inelastic energy absorption factor of floor systems. Previous testing and studies by others have demonstrated that both reinforced concrete and steel structures have significant capability to absorb earthquake energy inelastically at ground motion levels above the static strength. This increased capacity can be quantified by the ductility scale factor, F_μ, which scales the earthquake time history from the point at which static strength is reached to the *target level of ductility*.

The ductility scale factor was studied in Kennedy, 1984 for *vertically mounted* shear walls subjected to horizontal motions. Nonlinear analyses were performed and simplified procedures were developed and investigated in that report. Other simplified procedures for obtaining F_μ for *vertically-mounted* elements can be found in Riddell, 1979 and Reed, 1991.

Because of the constant downward pull on a horizontally-oriented structural element due to gravity, the nonlinear vertical displacement of a floor structure, when subjected to vertical motion, is biased downward. This ratcheting phenomenon reduces the ductility scale factor for a horizontal element compared to the case where the same element is mounted vertically and is subjected to horizontal motion.

The objective of this study was to develop a simple modification to the procedure for calculating the ductility scale factor for *vertically-mounted* components to calculate the F_μ factor for *horizontal elements*. This was achieved by first performing nonlinear time history analyses for *horizontally-* and *vertically-mounted* elements and then using the results to obtain a simplified procedure for including the effects of dead load.

Model and Procedure

The response of a concrete floor structure can be idealized as a one-degree-of-freedom (1-DOF) system since its response to seismic motion is predominantly due to the response of its fundamental mode. Figure A-1 shows the 1-DOF model mounted both vertically and horizontally. Also shown in Figure A-1 is the force-deformation relationship assumed for the model. This relationship is used for both orientations of the model.

The capacity of typical floor slabs in nuclear power plants, composed of both reinforced concrete and structural steel members, is due primarily to ductile flexural behavior. Since the slabs are under-reinforced, the capacity is controlled by the strength of reinforcing steel, not due to crushing of concrete. This leads to full hysteretic loops where there is very little, if any, cyclic strength or stiffness degradation. Thus, the force-deformation curve is assumed to be bilinear with a small amount of strain-hardening to reflect the characteristics of both structural steel members and reinforcing steel, as shown in Figure A-1.

The time history used in the analysis is the modified Pacoima Dam vertical record used in the Diablo

Canyon Long Term Seismic Program (PG&E, 1988). Figures A-2 and A-3 show plots of the time history and the 5-percent damped response spectrum for this time history, respectively. The absolute level of the input time history is not significant since F_μ is equal to the earthquake scale factor required to reach the capacity corresponding to the desired ductility level divided by the scale factor to reach yield. Thus, the absolute level of the input time history cancels out.

Two fundamental frequencies for the models, one at 4 hertz and the other at 7 hertz, were analyzed to reach three levels of ductility, μ. The values assigned to μ were 2, 5 and 10, μ being the ratio of the displacement at the desired ductility level to the displacement at yield (see Figure A-1). The damping was assumed to be 7 percent of critical, although additional sensitivity studies were performed to show that the ductility scale factor was only weakly influenced by the elastic damping value. Tangent stiffness-proportional damping was assumed, which implied that, beyond yield, the damping was almost equal to zero (i.e., 0.03 x 7 percent equals 0.2 percent). Finally, for all cases where the model was mounted horizontally the dead load stress was fixed at 30 percent of the yield level. This level is representative of the dead load stress level for typical floor slabs at the Diablo Canyon Plant.

Procedure and Results of Nonlinear Analysis

Nonlinear analyses were performed using the computer program DRAIN2D (Kannan, 1973, revised 1985). The bilinear truss model was used to simulate the inelastic behavior of the model, and a time step of 0.001 second was used for all computer runs.

A target displacement corresponding to an assigned value of ductility μ, was chosen and an iterative procedure was followed to determine the level of earthquake time history required to create the target displacement. Certain features in DRAIN2D ensured convergence of the iterative process in about 5 to 10 iterations.

Table A-1 shows results of the nonlinear analyses. Scale factors are shown for 4 hertz and 7 hertz models, and for both the cases of no dead load and dead load equal to 30 percent of yield. The "Yield FS" column is the factor used to scale the earthquake time history to reach yield level (i.e., the point defined by P_y and Δ_y in Figure A-1c). Similarly, the "Ultimate FS" column is the factor required to scale the time history to reach the target displacement level (i.e., the displacement equal to $\mu\Delta_y$ in Figure A-1c). Finally, the ductility scale factor, F_μ, is the ratio of the corresponding values in these two columns.

Note that the yield scale factor for the "dead load equal to 30 percent P_y" cases do not exactly equal to 70 percent of the yield scale factors for the corresponding "no load" cases. This is because peak response for the dead load is sensitive to the sign of the input motion (i.e., plus or minus). For the "no load" case, the peak of the two direction responses is always used, while for the "dead load equal to 30 percent P_y case the downward component was used. It turned out that the signs were opposite for the cases considered. The effect of reversing the input direction on the ductility scale factor is discussed below.

Table A-1 shows that the ductility scale factor is reduced for the case where dead load is present. Figures A-4 and A-5 show example response time histories for the "no dead load" case and the "dead load equal to 30 percent P_y" case, respectively. The effect of the dead load can be clearly seen by comparing these two figures.

The effect of reversing the earthquake time history sign was also studied. Table A-2 gives results of changing the earthquake direction and compares the results to the original case (i.e., from Table A-1). Although the scale factors to reach yield and ultimate are both less for the reversed sign case, which produced the greatest response elastically, the ductility scale factors are very close. This implies that the earthquake sign does not have a significant effect on F_μ.

Development of Simplified Procedure

A simplified procedure was developed for estimating F_μ by first computing an "effective" system ductility (μ_e), which takes into account the effects of ratcheting. The Riddell-Newmark method (Riddell, 1979) was then directly applied to compute F_μ.

The effective ductility μ_e was determined by adjusting the system ductility as follows:

$$\mu_e = 1 + \frac{\mu - 1}{n}$$

Where:

μ_e = Effective system ductility by considering effect of ratcheting

μ = System ductility without ratcheting effects

n = Adjustment factor to account for ratcheting (n=3 was used in this evaluation)

The value of "n" may be interpreted as the number of strong motion ratcheting cycles to which the structure is subjected, and may depend upon, among other variables, duration of ground motion, difference in the available resistances in the upward vs. downward directions before yield, and input time history of ground motion. Such an interpretation of "n" is illustrated in Figure A-6.

In the Riddell-Newmark method, the ductility scale factor is predicted from equations that are defined for the amplified acceleration, velocity and displacement regions of the input response spectrum. For a damping ratio of 7 percent of critical, the following equations predict F_μ for systems in different frequency ranges. For systems considered in this study, only the acceleration and velocity regions are of interest.

Amplified Acceleration Region: $F_\mu = (2.673\mu - 1.673)^{0.411}$ (1)

Amplified Velocity Region: $F_\mu = (2.24\mu - 1.24)^{0.611}$ (2)

Rigid Range Limit: $F_\mu = \mu^{0.11}(S_a/ZPA)$ (3)

Where:

S_a = 7 percent damped spectral acceleration

ZPA = Zero period acceleration

For the amplified acceleration region, F_μ is defined as the lesser of either equation (1) or equation (3).

Also, since both the 4 hertz and 7 hertz oscillators have frequencies that lie below the peak of the input spectrum, the rigid range limit was not applied.

Note that, for this study, the simplified procedure was established by modifying the above Riddell-Newmark equations by substituting effective ductility, μ_e, for the value of system ductility, μ.

Comparisons of Results of Simplified Procedure with Nonlinear Analyses

The simplified procedure was used to calculate values of ductility scale factor, F_μ, assuming an earthquake having 3 strong motion ratcheting cycles. These were compared with the corresponding quantities from nonlinear time history analyses and the results are shown in Table A-3.

As shown in the table, F_μ for the 4 Hz system agree well with the prediction by the Riddell-Newmark method using the velocity region equations. Since the 4 Hz frequency lies far below the peak of the input spectrum, use of velocity region equation is appropriate. For a ductility of 10, however, the Riddell-Newmark method under predicts the F_μ value. For the 7 Hz system, the results agree well when, for the Riddell-Newmark method, the geometric mean of the acceleration and the velocity region equations are used.

Conclusions

The study was performed to examine the effect of the constant downward pull on a horizontally-oriented structural element due to gravity on the nonlinear vertical displacement of a floor being subjected to vertical motion. We observe from the results of nonlinear time-history analyses that the ratcheting phenomenon created by the dead load reduces the ductility scale factor for a horizontal element compared to the case where the same element is mounted vertically and is subjected to horizontal motion i.e., with no dead load.

A simplified procedure is developed by modifying the Riddell-Newmark method. The procedure incorporates the effect of ratcheting by dead load and is achieved by using an "effective ductility", μ_e, that is a function of the system ductility, μ, and the number of strong motion cycles, n. For LTSP application using the LTSP vertical ground motion spectral shape, the simplified procedure is considered to be valid when a value of n = 3 is used, and for structures subjected to gravity loads in the range of about 30 percent of the yield strength of the system.

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Riddell, R., and Newmark, N. M., 1979, Statistical analysis of the response of nonlinear systems subjected to earthquake, SRS 468, Department of Civil Engineering, University of Illinois.

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Kannan, A. E. and Powell, G. H., DRAIN-2D: A general computer program for dynamic analysis of inelastic plane structures, 1973 (Revised 1985), distributed by NISEE/Computer Applications, EERC, University of California, Berkeley, Reports No. EERC 73-6 and EERC 73-22.

Table A-1

Scale Factor for No Dead Load and Dead Load Cases

Ductility, μ	4 Hz Model			7 Hz Model		
	Yield FS	Ultimate FS	F_μ	Yield FS	Ultimate FS	F_μ
No Dead Load						
2	4.83	8.52	1.76	-	-	-
5	4.83	17.77	3.68	3.78	8.57	2.27
10	-	-	-	-	-	-
Dead Load Equal to 30 Percent P_y						
2	3.71	5.29	1.43	3.03	4.09	1.35
5	3.71	8.27	2.23	3.03	5.55	1.83
10	3.71	18.22	4.91	3.03	9.16	3.02

Table A 2

Comparison of Scale Factor Based on Earthquake Sign

	4 Hz Model			7 Hz Model		
	Yield FS	Ultimate* FS	F_μ	Yield FS	Ultimate* FS	F_μ
Original Sign (+)	3.71	8.27	2.23	3.03	5.55	1.83
Reversed Sign (-)	3.38	7.75	2.29	2.65	5.08	1.92

* Ductility, μ, equa. to 5

Table A-3

Comparison of Nonlinear Analysis Results with Modified Riddell-Newmark Method

Frequency (Hz)	Target Ductility μ	Ductility Scale Factor, F_μ			
		Nonlinear Analysis* (see Table A-1)	Modified Riddell-Newmark Method		
			Velocity Region	Acceleration Region	Geometric Mean
4	2	1.43	1.41	1.30	1.35
	5	2.23	2.33	1.87	2.09
	5**	2.29	2.33	1.87	2.09
	10	4.91	3.49	2.47	2.94
7	2	1.35	1.41	1.30	1.35
	5	1.83	2.33	1.87	2.09
	5**	1.92	2.33	1.87	2.09
	10	3.02	3.49	2.47	2.94

* Dead load equal to 30 percent P_y

** Denotes case where the input time history is applied with a reverse sign.

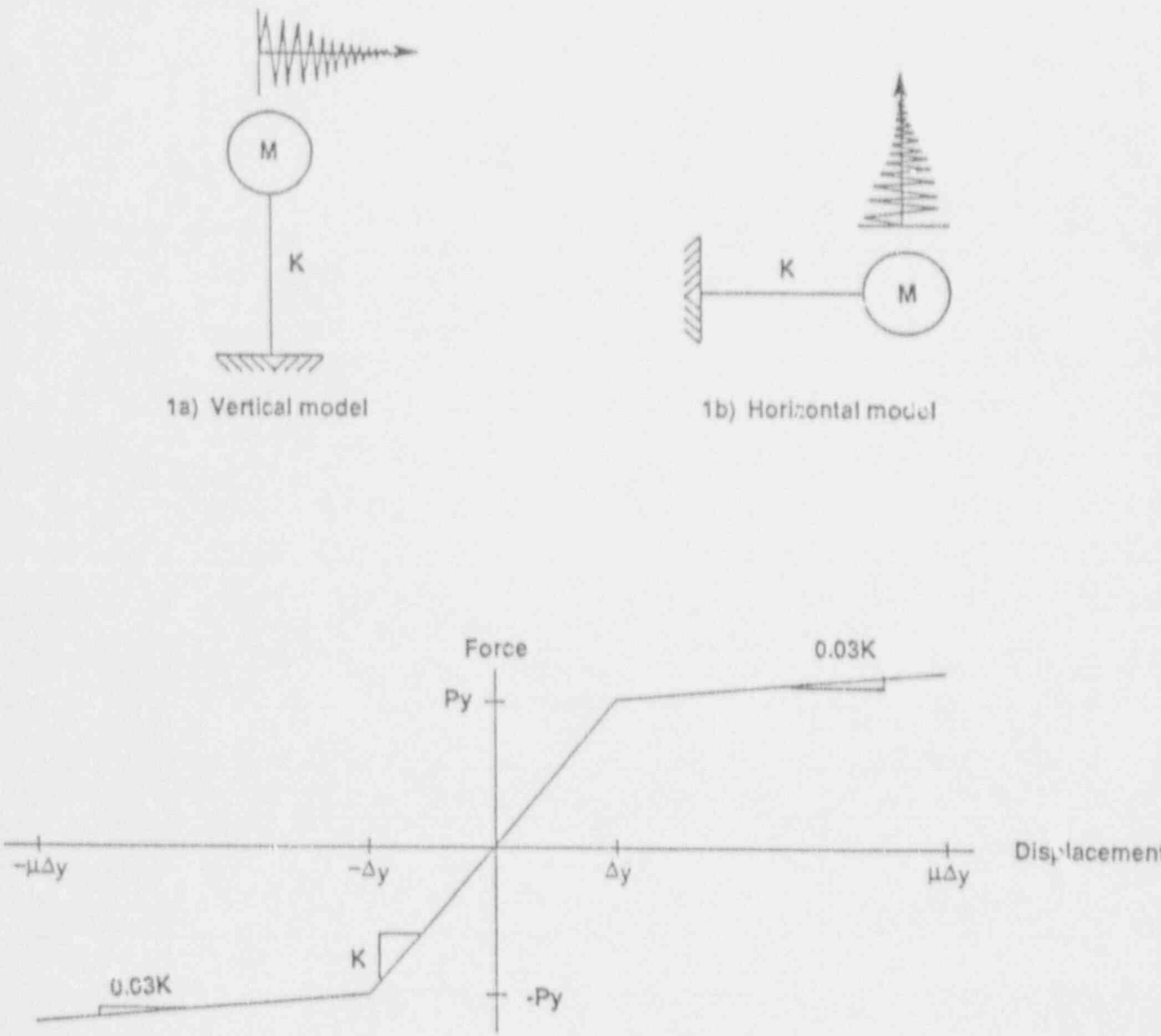


Figure A-1

Models used in the nonlinear analysis.

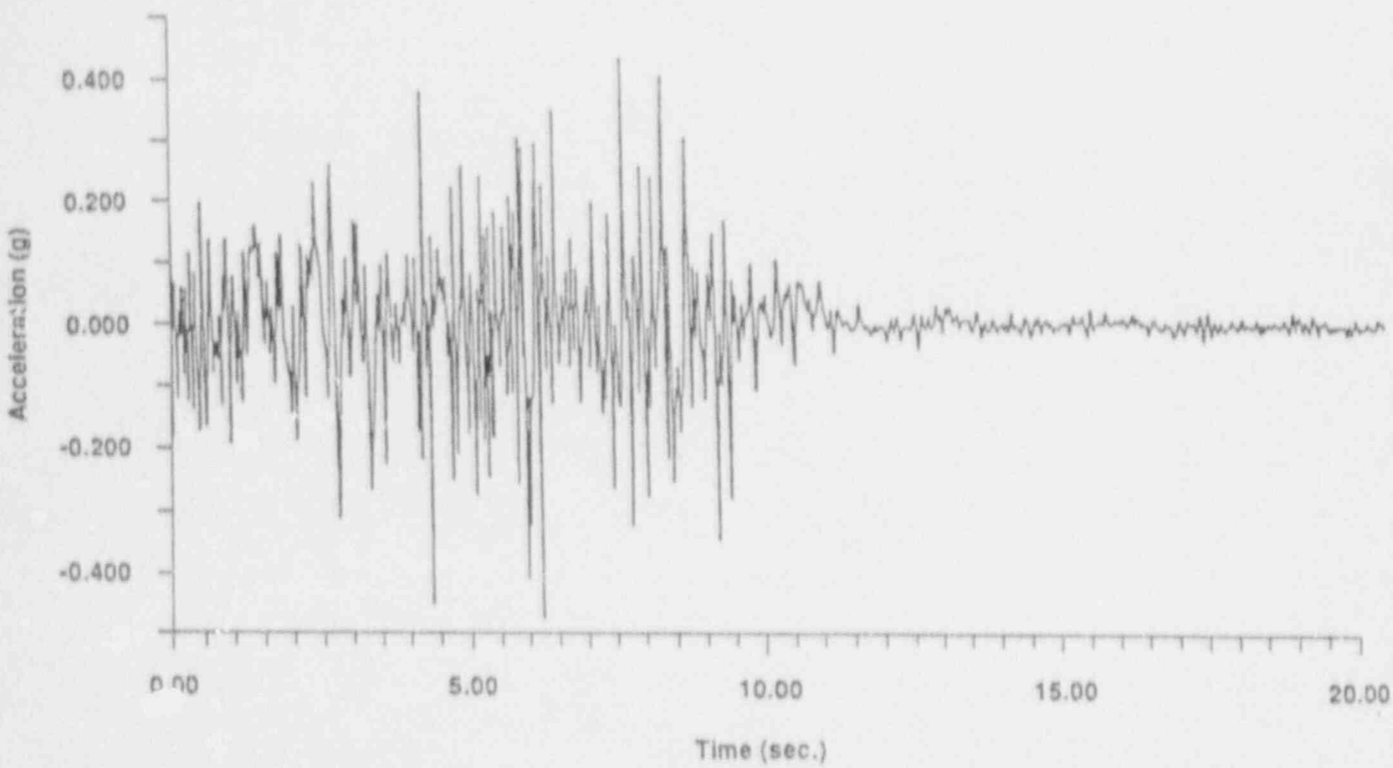


Figure A-2

Input time history used in the nonlinear analysis.

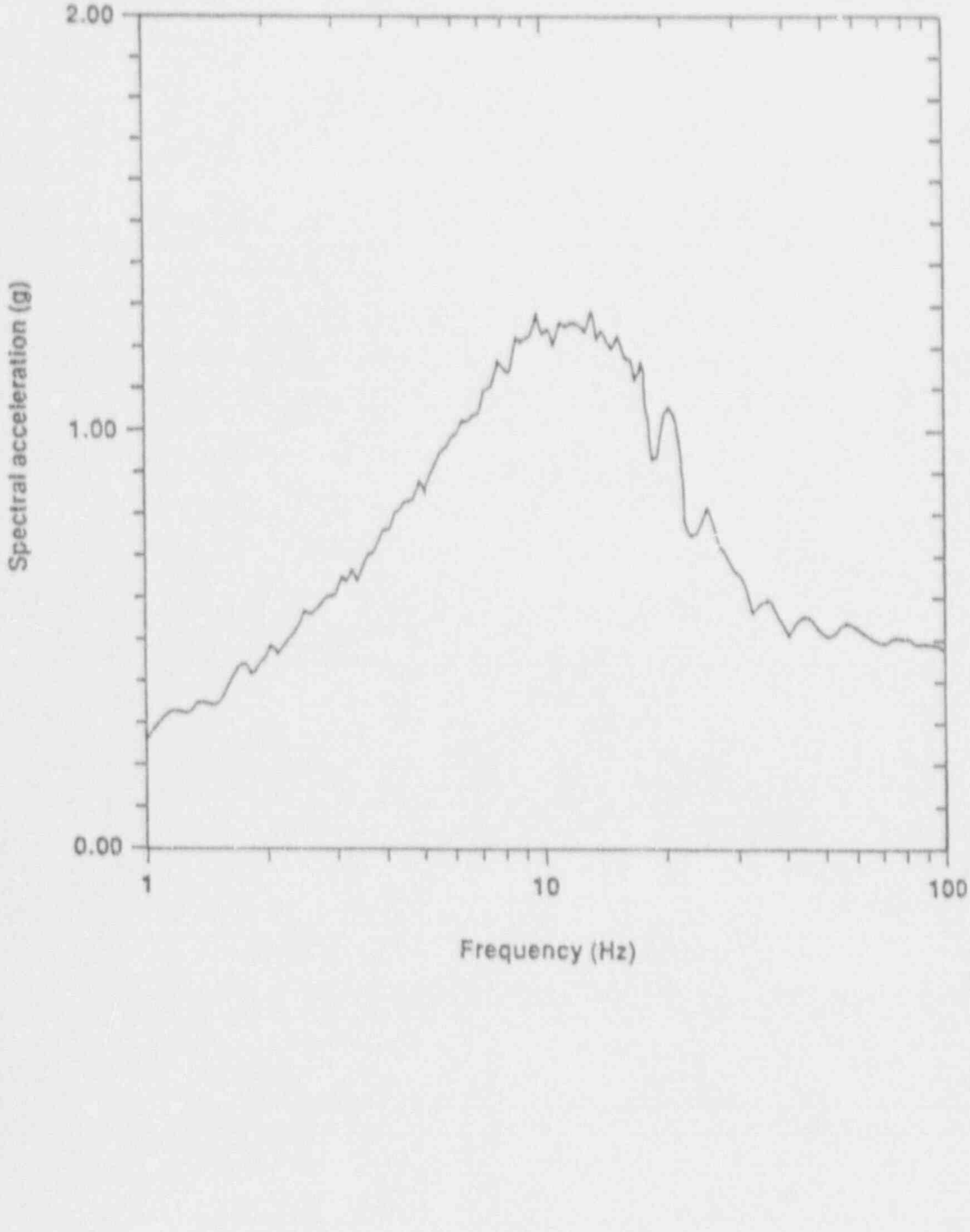


Figure A-3

Response spectrum of the input time history.

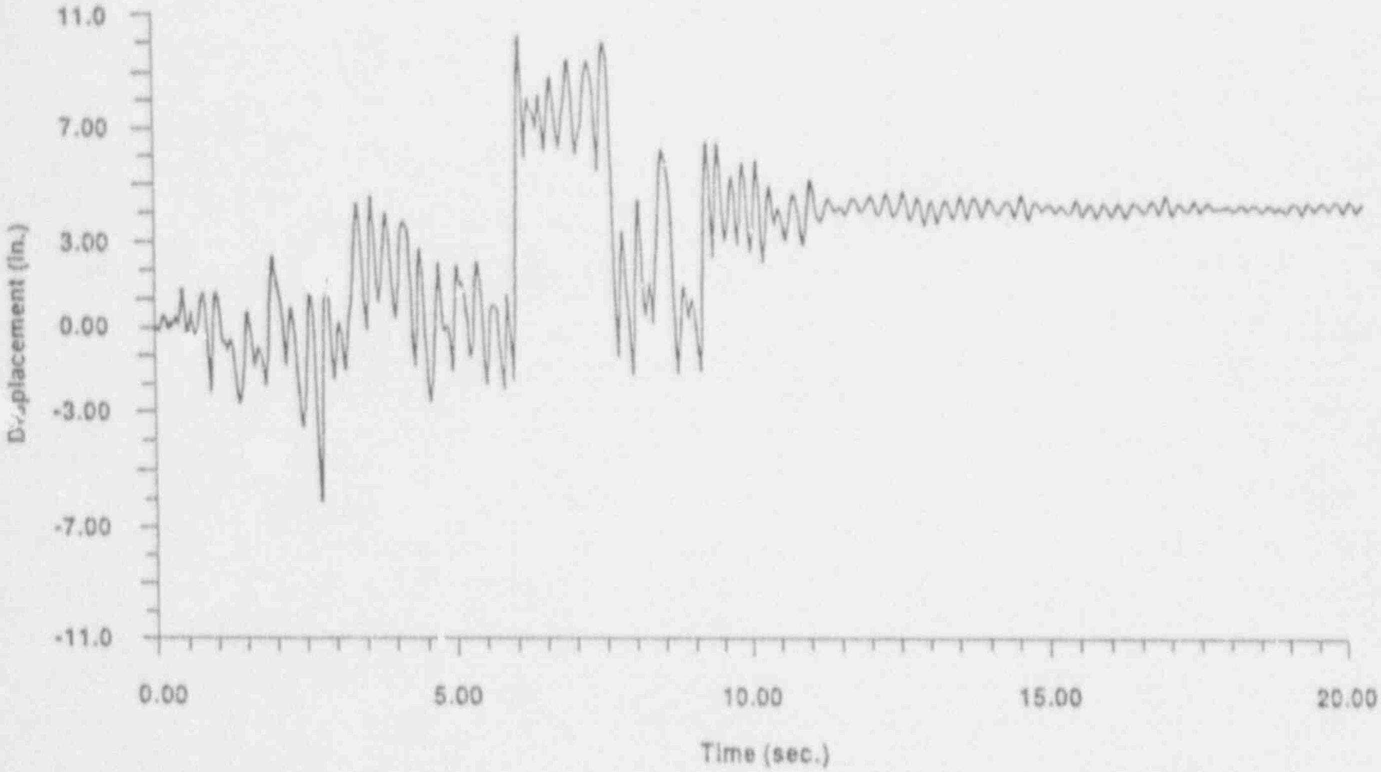


Figure A-4

Response time history for the case without dead load.

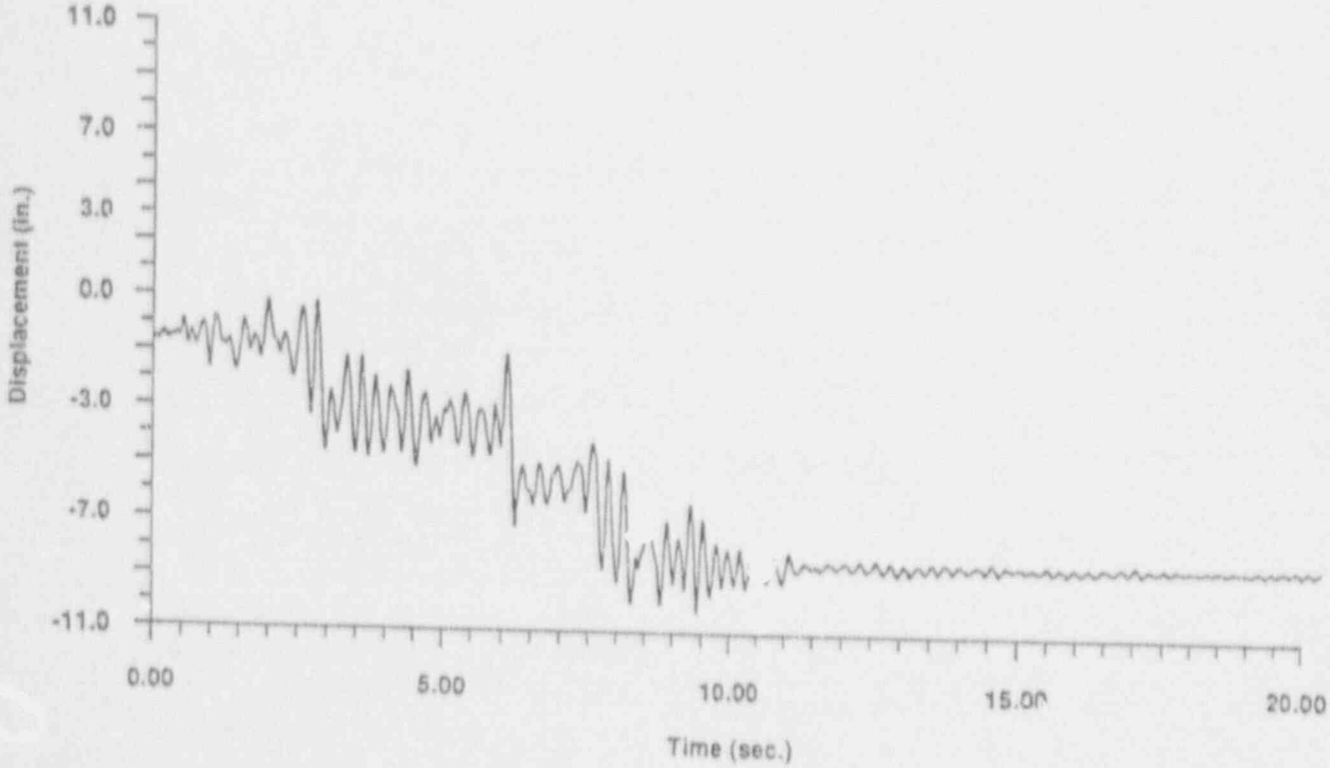
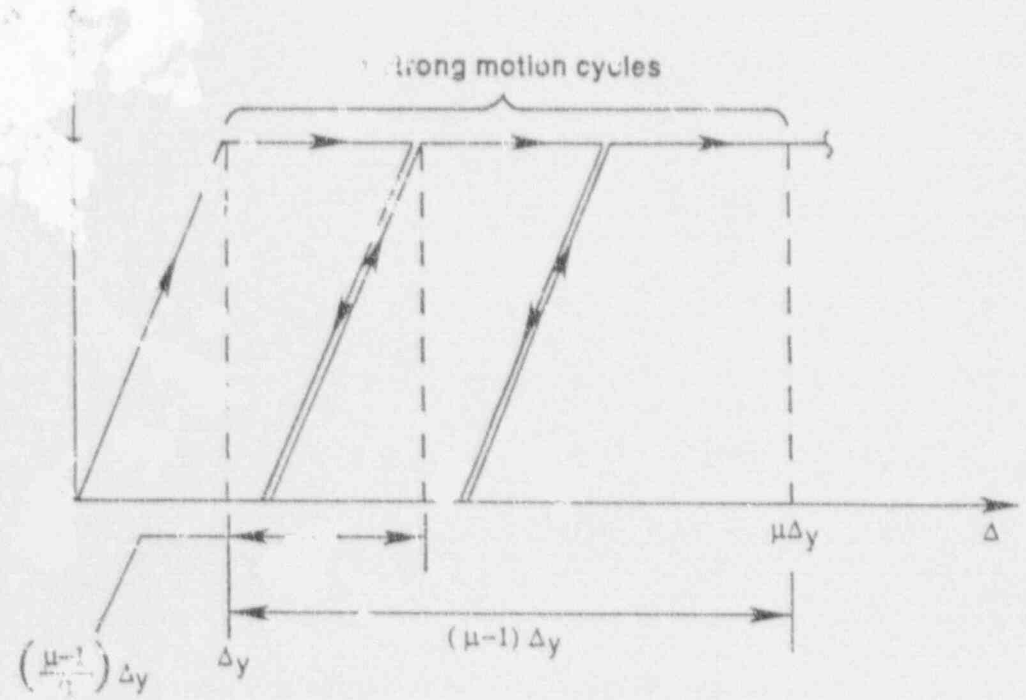


Figure A-5

Response time history for the case with dead load equal to 30 percent yield.



$$\text{Ductility per cycle} = \frac{\left(\frac{\mu-1}{n}\right)\Delta_y + \Delta_y}{\Delta_y}$$

$$\text{Effective ductility,}\quad \mu_e = \frac{1}{n}(\mu-1)+1$$

Figure A-6

Effective ductility considering multiple strong motion cycles.