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Seismic Safety Margins Research Program

**Seismic Structural Fragility Investi-
gation For The San Onofre Nuclear
Generating Station, Unit 1 (Project I)**

Songs-1 AFWS Project

D. A. Wesley and P. S. Hashimoto

Prepared for
U.S. Nuclear Regulatory Commission

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EXECUTIVE SUMMARY

The evaluation of the potential seismic failure modes of a system as complex as those at the San Onofre Nuclear Generating Station, Unit 1 (SONGS-1) involves consideration of a great many items such as strengths and building response characteristics. In most cases, values of many of the parameters affecting structural seismic capacity levels are not known exactly and substantial dispersion may exist. This dispersion can result from sources of inherent randomness and from uncertainty concerning the values of the sources. Randomness is considered to represent those sources of dispersion which cannot be reduced by additional analysis or more data. Uncertainty can be considered to be the result of lack of knowledge of such parameters as material properties, as well as approximations in analytical modeling.

An overall probabilistic methodology was developed by Lawrence Livermore National Laboratory (LLNL) as a part of the Seismic Safety Margins Research Program (SSMRP) to evaluate the sources of reactor risk from a seismic event. Seismic structural fragilities to be used as input into this methodology are defined by estimated median seismic capacities and their variabilities due to randomness and uncertainty. The median capacities and their variabilities account for structure strength and inelastic energy absorption capabilities. Estimation of these quantities is dependent to a significant extent on engineering judgment in conjunction with available test data. To determine the earthquake level at which structural failure occurs, structural capacity is compared to estimates of seismic response which account for damping, modal combination, earthquake directional components, and soil-structure interaction. Structure strength and ductility are assumed to be lognormally distributed since this distribution seems to provide a good representation of the sources of structural variability. The lognormal distribution is also mathematically convenient for defining fragility curves.

SONGS-1 structures evaluated in this study included the reactor building, containment sphere, sphere enclosure building, control-administration building, diesel generator building, turbine-generator pedestal, north turbine extension, west heater platform, and the fuel storage building. Structural

failure is defined as occurring when inelastic seismic structural deformations are of sufficient magnitude to potentially interfere with the operability of attached equipment. These deformation limits correspond to the onset of significant structural damage. Potential failure modes were identified for each of the SONGS-1 structures included in this study by establishing the seismic load-paths and comparing expected load distributions to available capacities for the elements of each load-path. Particular attention was given to possible weak links and details. The more likely failure modes were screened for more detailed investigation.

Determination of median capacities for fragility curve formulation is dependent on estimates of structural response due to seismic excitation. Final results from the SONGS-1 seismic structural analyses, which were to have been conducted by others, were not available prior to the discontinuation of work on this study. These analytical results were to provide the correlation between earthquake input and structural failure. Calculations of median structure strengths and variabilities for the identified failure modes were only partially completed before work was stopped on this project. These median strengths account for median material properties estimated from actual plant-specific test data when available or by comparison to other data bases. Predictive models derived from actual test data were used whenever possible to estimate median element strengths. These models eliminate design conservatism normally associated with building code requirements.

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ABSTRACT

An evaluation of the seismic capacities of several of the San Onofre Nuclear Generating Station, Unit 1 (SONGS-1) structures was conducted to determine input to the overall probabilistic methodology developed by Lawrence Livermore National Laboratory. Seismic structural fragilities to be used as input consist of median seismic capacities and their variabilities due to randomness and uncertainty.

Potential failure modes were identified for each of the SONGS-1 structures included in this study by establishing the seismic load-paths and comparing expected load distributions to available capacities for the elements of each load-path. Particular attention was given to possible weak links and details. The more likely failure modes were screened for more detailed investigation.

Final results from the SONGS-1 seismic structural analyses, which were to have been conducted by others, were not available prior to the discontinuation of work on this study. These analytical results were to provide the correlation between earthquake input and structural failure. Calculations of median structure strengths and variabilities for the identified failure modes were only partially completed before work was stopped on this project. These median strengths account for median material properties estimated from actual plant-specific test data where available or by comparison to other data bases. Predictive models derived from actual test data were used whenever possible to estimate median element strengths. These models eliminate design conservatism normally associated with building code requirements.

1. INTRODUCTION

The evaluation of the potential seismic failure modes of a system of structures as complex as those composing the San Onofre Nuclear Generating Station Unit 1 (SONGS-1) involves consideration of a great many items such as strengths and building response characteristics. This is true even for the case of only one major subsystem such as the Auxiliary Feedwater System (AFWS). In most cases, values of many of the parameters affecting structure seismic capacity levels are not known exactly and substantial dispersion in both material and structure response characteristics exists. Variations can occur in virtually every aspect of the seismic fragility evaluation including structure frequencies and dynamic response, strength and inelastic energy absorption capacity, and structure response effects resulting from different earthquake dynamic characteristics. The dispersion can result from both sources of inherent randomness and from uncertainty concerning the values of the sources. In this context, inherent randomness is considered to represent those sources of dispersion which cannot be reduced by additional analysis or more data. Modeling uncertainty, on the other hand, is considered to be the result of lack of knowledge of material properties and other parameters, and approximations in analytical modeling.

The Seismic Safety Margins Research Program (SSMRP) was initiated to investigate the effects of variability in the seismic methodology and obtain quantitative estimates of the conservatism introduced throughout the seismic design and analysis of a nuclear power facility. An overall probabilistic methodology was developed by Lawrence Livermore National Laboratory (LLNL) as a part of Phase I of the SSMRP. Input into this methodology includes estimates of structural fragilities for structures whose failure could influence the operability of critical systems and components.

Attention has recently been focused on the auxiliary feedwater system which functions as part of the decay heat removal system in pressurized water reactors. LLNL was asked by the U.S. Nuclear Regulatory Commission to apply the probabilistic methodology developed by the SSMRP to the risk study of an auxiliary feedwater system (Reference 1). The facility selected for evaluation was SONGS-1.

The SONGS-1 facility consists of several structures. Those whose failure could result in a loss of function of the auxiliary feedwater system include the reactor building, containment sphere, sphere enclosure building, control-administration building, diesel generator building, turbine pedestal, north turbine extension, west heater platform, and the fuel storage building. The reactor building and the containment sphere are founded on a common base mat and are covered by the sphere enclosure building. The turbine pedestal, north turbine extension, and west heater platform house the turbine-generator and some of its related equipment. They are all seismically separated from each other and are treated as individual structures. Included with the turbine pedestal is the turbine gantry crane that travels on the turbine deck. Seismic upgrade modifications to some of these structures are currently being designed. The influences of these modifications on structural capacity and response have been incorporated in this evaluation based on the latest design drawings available.

This investigation is directed principally towards seismically-induced failure of structures. Equipment anchorage failure involving shear or pullout of concrete anchor bolts or local equipment failures are not discussed in this report. Although such details may be within the civil/structural scope of supply, this type of failure results from equipment response which is not considered here. Also, the emphasis is on seismically-induced modes rather than on those resulting from combined Loss-of-Coolant-Accident (LOCA) or other extreme load combinations. This assumption was based on the low probability of the maximum earthquake and LOCA loads occurring at the same time. Even if a LOCA occurs as a result of an earthquake, sufficient time may be expected to elapse such that the maximum seismic and LOCA loads are not expected to occur together. Should such an event occur,

however, as for instance in the possible case of aftershocks, some capacity of the containment vessel would not be available to withstand the seismic loads. For the most part, failure of equipment is not expected to significantly affect the seismic fragility levels of the structures within which the equipment is housed. This is due to the relatively small mass of most equipment items. However, some exceptions exist such as damage to the pressurizer or steam generators with subsequent LOCA, which would affect the seismic capacity of the containment sphere. Another example may be the turbine gantry crane whose failure could possibly damage such structures as the condensate storage tank.

The investigations discussed in this report are limited to the potential failure modes of the primary structures. Failure modes involving soil bearing capacity, soil liquefaction, surface faulting, tsunamis, etc., are considered outside the scope of this evaluation. The effects of inelastic energy dissipation are included in the determination of the structural fragility levels by means of simplified approaches based on reductions from linear response levels. No nonlinear analysis of any of the SONGS-1 structures was conducted during this project. In actuality, very few structures respond completely elastically, even at very low response levels. Variations in dynamic response characteristics of structures would normally be expected during and after virtually any level of earthquake. Often, however, the structure will shake down for low-to-moderate level earthquakes so that variations due to subsequent excitations may not be as large as from the initial excitation.

Definition of structural fragilities for input into the LLNL probabilistic risk analysis includes estimated median seismic capacities and variabilities (Reference 41). Median capacities are expressed in terms of reference structural response quantities and are thus a function of seismic response as well as structure strength and ductility. Results of the seismic response analyses for the SONGS-1 structures were to have been provided by others. However, this effort was suspended, and SMA was directed to discontinue effort to determine the structural fragilities prior to the receipt of final structural seismic analytical results.

Correlation of structure capacities to responses was therefore not completed. At the time the effort was discontinued, calculations to estimate median strengths for the potential failure modes were completed for most of the structures influencing the auxiliary feedwater system. Analytical response results for the reactor building developed for a comparison between different mathematical models were received. However, they provide only preliminary estimates of reactor building response and were not used in fragility calculations. Since work was discontinued before final structure responses were provided, this report presents only the results of calculations conducted to estimate median strengths and some variabilities for potential SONGS-1 structure failure modes.

2. METHOD OF FRAGILITY EVALUATION

The original scope of this evaluation included the development of fragility levels for the SONGS-1 structures whose failure would influence the auxiliary feedwater system. Although effort was discontinued prior to the completion of this task, the following discussion is included to describe the basis for structural fragilities derivation. The approach used in the evaluation of structures fragility levels is to compute median factors of safety as opposed to the original design values for the important parameters and determine the variability associated with each parameter. Variability in the dynamic response and failure levels of structures can originate from a great many potential sources. Many of these sources are associated with the random variability resulting from material properties and geometry. Other sources are introduced by modeling uncertainties caused by lack of precise knowledge by the analyst. Often, these modeling uncertainties are recognized, and conservative assumptions are introduced to account for them at various stages throughout the analysis process. These tend to introduce a systematic bias in many aspects of the analysis, which may or may not be recognized. In many cases, this bias is legislated by code or licensing requirements. It is often difficult to completely separate random variability and uncertainty. An attempt to do so has been maintained throughout this investigation, however.

In order to develop quantitative estimates of the variability, it is necessary to define a mean or median value of the parameter and a measure of dispersion in terms of standard deviation depending on whether a normal, lognormal, or other distribution is used. For most sources of random variability, inadequate information exists to exactly define which statistical distribution most accurately represents the data. For most sources of structural variability, the lognormal distribution provides a good representation (References 2 and 3), so long as the extreme tails are not of primary concern. Furthermore, the central limit theorem states that a distribution consisting of products and quotients of distributions of several variables tends to be

lognormal even if the individual variable distributions are not lognormal. For all variables discussed in this evaluation of the dynamic characteristics of structures, a lognormal distribution is assumed. For values of the logarithmic standard deviation less than about one-half, the logarithmic standard deviation and the coefficient of variation (COV) are approximately equal and are often used interchangeably.

2.1 SONGS-1 DESIGN BASIS

Most of the SONGS-1 structures were designed in the early 1960's in accordance with criteria and codes in effect at that time (Reference 4). Reevaluation of some of these structures has subsequently been performed and reevaluation of the remaining safety-related structures is in progress. The original plant construction was designed to withstand both a Design Earthquake equivalent to the present Operating Basis Earthquake (OBE) and a Maximum Earthquake equivalent to the present Safe Shutdown Earthquake (SSE). The peak ground acceleration levels used in the design of the original structures were:

	Horizontal (g)	Vertical (g)
Design Earthquake	0.25	0.17
Maximum Earthquake	0.50	0.33

The original design ground response spectra for the Design Earthquake (equivalent to OBE) and Maximum Earthquake (equivalent to SSE) were Housner response spectra scaled to the peak ground acceleration levels listed above. The Design Earthquake response spectra are shown in Figure 2-1.

Reevaluation of the reactor building and containment sphere was subsequently conducted in the early 1970's using criteria and codes currently in effect at that time. Seismic input to the analysis consisted of Housner response spectra scaled to an increased peak ground acceleration of 0.67g (Reference 5). The sphere enclosure and diesel generator buildings were added following original plant construction. Seismic response of these

structures was generated using site specific response spectra scaled to a peak ground acceleration of 0.67g developed for the design of the SONGS Units 2 and 3 structures (Reference 5).

Structures included in the original plant construction were originally divided into three seismic categories. Definitions of these categories are:

- Category A: Components, systems, and structures that are important to the safety of the plant.
- Category B: Components, systems, and structures that are important to the continuity of power generation or whose activity is such that failure would not constitute a health hazard.
- Category C: All others.

Original SONGS-1 structures included in this study were categorized as noted in Table 2-1.

2.2 DIFFERENCES BETWEEN DESIGN CRITERIA AND FRAGILITY EVALUATION

A number of differences exist between criteria used for design of structures and those used for evaluation of failure levels. For design, conservative assumptions are made where uncertainties exist. In determining fragility levels, an attempt is made to determine median centered properties for all parameters. Since the design of SONGS-1, a number of changes in design and qualification criteria for structures have been implemented. The general trend has been to increase allowable stresses but also to increase the number of loading conditions to be considered simultaneously.

The degree of sophistication used in analysis has also increased since the original design of the SONGS-1 structures. Many uncertainties still exist. Only the uncertainties associated with the seismic capacities of the structures are considered in this report. Others, including variables such as structure damping, earthquake directional components, load combinations, seismic input characteristics, and soil-structure interaction modeling procedures, are important primarily when determining structure response, and are not discussed in this report.

2.3 FORMULATION USED FOR FRAGILITY CURVES

Seismic-induced fragility data are generally unavailable for specific plant components and are certainly unavailable for the specific SONGS-1 structures. Thus, fragility curves must be developed primarily from analysis combined heavily with engineering judgment supported by very limited test data. Such fragility curves will contain a great deal of uncertainty, and it is imperative that this uncertainty be recognized in all subsequent analyses. Because of this uncertainty, great precision in attempting to define the shape of these curves is unwarranted. Thus, a procedure which requires a minimum amount of information, incorporates uncertainty into the fragility curves, and easily enables the use of engineering judgment was used in this study.

Seismic fragility curves for the various modes of failure are normally keyed to some characteristic of the earthquake, such as peak ground acceleration, or a characteristic of the structure response such as response acceleration or displacement at a specific location within the structure. Depending on the mode of failure and the type of analysis used in developing the structure seismic response loads, one characteristic may be more effective than another in describing the structure fragility. However, if the transfer function between structure response and input is known, the different response and input characteristics may be readily interchanged. For the purposes of discussion in this section, median ground acceleration is used, but the methodology is identical for other parameters.

The entire fragility curve for any mode of failure and its uncertainty can be expressed in terms of the best estimate of the median ground acceleration capacity, $\check{A}$, times the product of random variables. Thus, the ground acceleration, A , corresponding to failure is given by:

$$A = \check{A} \epsilon_R \epsilon_U \quad (2-1)$$

in which ϵ_R and ϵ_U are random variables with unit median representing the inherent randomness (failure fraction) about the median and the uncertainty (probability) in the median value, respectively. Equation 2-1 enables the

fragility curve and its uncertainty to be represented as shown in Figure 2-2; i.e., as a set of shifted curves with attached uncertainty levels. Thus, it is assumed that all uncertainty in the fragility curves can be expressed through uncertainty in the median alone.

Next, it is assumed that both ϵ_R and ϵ_U are lognormally distributed with logarithmic standard deviation of β_R and β_U , respectively. The advantages of this formulation are:

1. The entire fragility curve and its uncertainty can be expressed by three parameters - $\check{A}$, β_R , and β_U . With the very limited available data on fragility, it is much easier to only have to estimate three parameters rather than the entire shape of the fragility curve and its uncertainty.
2. The formulation in Equation 2-1 and the lognormal distribution are very tractable mathematically.

Another advantage of the lognormal distribution is that it is easy to convert Equation 2-1 to a deterministic composite "best estimate" fragility curve (i.e., one which does not separate out uncertainty from underlying randomness) defined by:

$$A = \check{A} \epsilon_C \quad (2-2)$$

where ϵ_C is a lognormal random variable with unity median and logarithmic standard deviation β_C given by:

$$\beta_C = \sqrt{\beta_R^2 + \beta_U^2} \quad (2-3)$$

This composite fragility curve (shown in Figure 2-2) can be used in preliminary deterministic safety analyses if one only needs a "best estimate" on failure fraction and does not desire an estimate of uncertainty.

The lognormal distribution can be justified as a reasonable distribution since the statistical variation of many material properties (References 2 and 3) and seismic response variables may reasonably be represented by this distribution so long as one is not primarily concerned with the extreme tails of the distribution. In addition, the central limit theorem states that a distribution consisting of products and quotients of distributions of several variables tends to be lognormal even if the individual distributions are not lognormal. Use of this distribution for estimating failure fractions on the order of one percent or greater is considered to be quite reasonable. Lower fraction estimates which are associated with the extreme tails of distribution must be considered more suspect. However, use of the lognormal distribution for estimating very low failure fractions of components or structures associated with the tails of the distribution is considered to be conservative because the low-frequency tails of the lognormal distribution generally extend further from the median than actual structural resistance or response data might extend since such data generally shown cut-off limits beyond which there is essentially zero frequency of occurrence.

Characteristics of the lognormal distribution which are useful to keep in mind when generating estimates of $\bar{X}$, β_R , and β_U are summarized (References 6 and 7). A random variable X is said to be lognormally distributed if its natural logarithm Y given by:

$$Y = \ln(X) \quad (2-4)$$

is normally distributed with the mean of Y equal to $\ln \bar{X}$ where $\bar{X}$ is the median of X , and with the standard deviation of Y equal to β , which will be defined herein as the logarithmic standard deviation of X . Then, the coefficient of variation, COV, is given by the relationship:

$$COV = \sqrt{\exp(\beta^2) - 1} \quad (2-5)$$

For β values less than about 0.5, this equation becomes approximately:

$$\text{COV} \approx \beta \quad (2-6)$$

and COV and β are often used interchangeably.

For a lognormal distribution, the median value is used as the characteristic parameter of central tendency (50 percent of the values are above the median value and 50 percent are below the median value). The logarithmic standard deviation, β , or the coefficient of variation, COV, is used as a measure of the dispersion of the distribution.

The relationship between the median value, $\check{X}$, logarithmic standard deviation, β , and any value x of the random variable can be expressed as:

$$x = \check{X} \cdot \exp(n \cdot \beta) \quad (2-7)$$

where n is the standardized Gaussian random variable, (mean zero, standard deviation one). Therefore, the frequency that X is less than any value x' equals the frequency that n is less than n' where:

$$n' = \frac{\ln(x'/\check{X})}{\beta} \quad (2-8)$$

Because n is a standardized Gaussian random variable, one can simply enter standardized Gaussian tables to find the frequency that n is less than n' which equals the probability that X is less than x' . Using cumulative distribution tables for the standardized Gaussian random variable, it can be shown that $\check{X} \cdot \exp(+\beta)$ of a lognormal distribution corresponds to the 84 percentile value (i.e., 84 percent of the data fall below the $+\beta$ value). The $\check{X} \cdot \exp(-\beta)$ value corresponds to the value for which 16 percent of the data fall below.

One implication of the usage of the lognormal distribution is that if A, B, and C are independent lognormally distributed random variables, and if

$$D = \frac{A^r \cdot B^s}{C^t} q \quad (2-9)$$

where q, r, s and t are given constants, then D is also a lognormally distributed random variable. Further, the median value of D, denoted by $\check{D}$, and the logarithmic variance β_D^2 , which is the square of the logarithmic standard deviation, β_D , of D, are given by:

$$\check{D} = \frac{\check{A}^r \cdot \check{B}^s}{\check{C}^t} q \quad (2-10)$$

and

$$\beta_D^2 = r^2 \beta_A^2 + s^2 \beta_B^2 + t^2 \beta_C^2 \quad (2-11)$$

where $\check{A}$, $\check{B}$, and $\check{C}$ are the median values, and β_A , β_B , and β_C are the logarithmic standard deviations of A, B, and C, respectively.

The formulation for fragility curves given by Equation 2-1 and shown in Figure 2-2 and the use of the lognormal distribution enables easy development and expression of these curves and their uncertainty. However, expression of uncertainty as shown in Figure 2-2 in which a range of peak accelerations are presented for a given failure fraction is not very usable in the systems analyses for frequency of radioactive release. For the systems analyses, it is preferable to express uncertainty in terms of a range of failure fractions (frequencies of failure) for a given ground acceleration. Conversion from the one description of uncertainty to the other is easily accomplished as illustrated in Figure 2-3 and summarized below.

With perfect knowledge (i.e., only accounting for the random variability, β_A), the failure fraction, $f(a)$, for a given acceleration a can be obtained from:

$$f(a) = \Phi \left(\frac{\ln(a/\bar{A})}{\beta_R} \right) \quad (2-12)$$

in which $\Phi(\cdot)$ is the standard Gaussian cumulative distribution function, and β_R is the logarithmic standard deviation associated with the underlying randomness of the capacity.

For simplicity, denote $f = f(a)$. Similarly, f' is the failure fraction associated with acceleration a' , etc. Then, with perfect knowledge (no uncertainty in the failure fractions) the ground acceleration a' corresponding to a given frequency of failure f' is given by:

$$a' = \bar{A} \exp \left[\beta_R \Phi^{-1}(f') \right] \quad (2-13)$$

The uncertainty in ground acceleration capacity corresponding to a given frequency of failure as a result of uncertainty of the median capacity can then be expressed by the following probability statement:

$$P[A > a'' | f'] = 1 - \Phi \left[\frac{\ln(a''/\bar{A})}{\beta_U} \right] \quad (2-14)$$

in which $P[A > a'' | f']$ represents the probability that the ground acceleration A exceeds a'' for a given failure fraction f' . This probability is shown shaded in Figure 2-3. However, one wishes to transform this probability statement into a statement on the probability that the failure fraction f is less than f' for a given ground acceleration a'' , or in symbols $P[f \leq f' | a'']$. This probability is also shown shaded in Figure 2-3. It follows that:

$$P[f \leq f' | a''] = P[A > a'' | f'] \quad (2-15)$$

Thus, from Equations 2-13 and 2-14:

$$P[f \leq f' | a''] = 1 - \Phi \left[\frac{\ln(a''/\bar{A} \exp[\beta_R \Phi^{-1}(f')])}{\beta_U} \right] \quad (2-16)$$

from which:

$$P[f > f' | a''] = \Phi \left(\frac{\ln(a''/\bar{A} \exp[\beta_R \Phi^{-1}(f')])}{\beta_U} \right) \quad (2-17)$$

which is the basic statement expressing the probability that the failure fraction exceeds f' for a ground acceleration a'' given the median ground acceleration capacity $\bar{A}$, and the logarithmic standard deviation β_R and β_U associated with randomness and uncertainty, respectively.

As an example, if:

$$\bar{A} = 0.77, \quad \beta_R = 0.36, \quad \beta_U = 0.39$$

then from Equation 2-17 for typical values of f and a'' ,

$$P[f > 0.5 | a'' = 0.40g] = 0.05$$

which says that there is a 5 percent probability that the failure frequency exceeds 0.5 for a ground acceleration of 0.40g.

2.4 BOUNDS ON CAPACITY

As discussed in the previous sections, the lognormal distribution is expected to provide reliable results for seismic capacity estimates for a number of reasons. For variables where sufficient information is available to assess the distribution, the lognormal distribution has been shown to represent the data at least as well as other possible distributions. This is particularly true in areas related to material strengths and earthquake characteristics. Also, the lognormal distribution is mathematically tractable, and finally, distributions consisting of products and quotients

of distributions tend to be lognormally distributed even if the individual distributions are not. The final point applies to virtually all fragility descriptions developed for this evaluation since the methodology used consists of identifying factors of safety and their associated variabilities for each of the principal steps in the seismic design qualification chain. The overall factor of safety consists of the product of a number of contributing factors of safety and the central limit theorem states the final distribution will tend to be lognormal even though not all the distributions associated with the individual contributing factors of safety are lognormal.

The lognormal distribution can therefore be expected to provide reliable results through the majority of the fragility curve so long as one is not primarily interested in the tails of the distribution below one to two percent. In these regions, use of the lognormal distribution for estimating low failure fractions is considered to be conservative. The degree of conservatism introduced into the probability of release is dependent not only on the conservatism in the fragility description but also on the seismic hazard description at the low seismic levels. However, if the seismic hazard for low input levels is large enough, it is apparent that very low level earthquakes can govern the seismic-induced release.

This is considered unrealistic for engineered structures and equipment found in nuclear power plants. Structures and equipment are subjected to static one g loads continually without failure. Low level dynamic loads resulting from cranes, fork lifts, and other component handling equipment produce loads which occur on a regular basis but which have never been known to produce structure failures. Also, wind loads produce lateral forces in structures which may be comparable in magnitude to those developed by earthquakes. However, for low level winds, it is inconceivable that well-engineered structures will exhibit even a very low frequency of failure, irrespective of the number of events.

Similarly, for low level earthquakes, it is expected that below some threshold, there is virtually no chance of failure due to seismic excitation. Material strength data, for instance, normally does not fall to

very low values compared to the median value but, instead, normally exhibits some lower bound. Other variables such as damping also indicate both lower and upper bounds which are not zero or infinite damping. Extensive studies have been conducted to develop response spectra from available earthquake records and while dispersion exists about the median values, spectra with essentially zero or infinite response do not occur. For these, as well as other variables contributing to the seismic fragility of a given structure or component, it is apparent that some lower and upper bound cut-offs on the tails of the dispersion exist. Since the overall fragility curves are based on a combination of these variables, it is expected a threshold exists below which no failures will occur and this is supported by experience. Although quantitative data is lacking, this threshold value is expected to be approximately minus two lognormal standard deviations using the "best estimate" or composite fragility variability. The composite lognormal standard deviation, β_c , was used on the basis for the cut-off rather than randomness or uncertainty since the composite value combines the effects of both dispersions. A similar cut-off may be expected to exist on the upper bound of the frequency of failure. In this case, plus three lognormal standard deviations based on the composite fragility curve is expected to provide a more accurate estimate.

It is therefore recommended that where a significant contribution to risk arises from very low level earthquakes, that the above truncation be applied in order to eliminate any excess conservatism introduced by the use of the lognormal distribution at very low frequencies of failure. An example is shown in Figure 2-4 for a typical fragility curve. In this example, the median effective peak ground acceleration for failure is approximately 1.1g with a random lognormal standard deviation (β_R) of 0.12 and lognormal standard deviation associated with uncertainty (β_U) of 0.20. The "best estimate" or composite standard deviation (β_c) is therefore approximately 0.23. Using the approach discussed above, the cut-off ground acceleration associated with minus two lognormal standard deviations is approximately 0.69g and intersects the composite fragility curve at a frequency of failure of approximately 0.02. The upper bound cut-off at plus three standard deviations occurs at approximately 2.2g.

TABLE 2-1
CATEGORIZATION OF STRUCTURES

STRUCTURES	CATEGORY	REMARKS
Reactor Building and Containment Sphere	A	Needed for nuclear safety and orderly shutdown
Fuel Storage Building	A	Needed for nuclear safety
Control-Administration Building		
Control Room	A	Control of orderly shutdown and safety injection
Switchgear Room	A	Auxiliary electric power for orderly shutdown and safety injection
Battery Room	A	D-C control power for orderly shutdown and safety injection
Turbine Pedestal	A	Connection to main steam lines
Heater Platforms	B	
Other Structures	C

FIGURE 2-1: ORIGINAL SONGS-1 DESIGN EARTHQUAKE RESPONSE SPECTRA

FIGURE 2-2: SIMPLIFIED REPRESENTATION OF FRAGILITY CURVE

FIGURE 2-3: RELATIONSHIP BETWEEN UNCERTAINTY IN GROUND ACCELERATION FOR A GIVEN FAILURE FRACTION AND UNCERTAINTY IN FAILURE FRACTION FOR A GIVEN GROUND ACCELERATION

FIGURE 2-4. TYPICAL FRAGILITY CURVE

3. CAPACITY EVALUATION

In this chapter, the methods used in the development of the median factors of safety and logarithmic standard deviations for the SONGS-1 structures are discussed. These median factors of safety were to be used to define the structure capacity in terms of the reference in-structure response quantity. Although the median factors of safety were never calculated due to the lack of final structural response analytical results, the methodology for determining them is presented.

In order to estimate the median factor of safety against structure or component failure, it is necessary to define what constitutes failure. For purposes of this study, structures are considered to fail functionally when inelastic deformations of the structure under seismic loads are estimated to be sufficient to potentially interfere with the operability of safety-related equipment attached to the structure. These limits on inelastic energy absorption capability (ductility limits) chosen for the structures are estimated to correspond to the onset of significant structural damage. This is believed to represent a conservative bound on the level of inelastic structural deformation which might interfere with the operability of components housed within the structure. It is important to note that considerably greater margins of safety against structural collapse are believed to exist for these structures than most cases reported within this study. Thus, the conditional probabilities of failure for a given reference in-structure response reported herein for structures are considered appropriate for operability limits and should not necessarily be inferred as corresponding to structure collapse.

The development of the seismic capacities together with the expected dispersion for the various failure modes is conveniently treated using factors of safety based on the SSE. The overall factor of safety, as well as the variability for each potential failure mode, may consist of a large number of contributing parameters. The factor of safety for the structure seismic capacity consists of two main parts:

1. The strength factor, F_s , based on the ratio of actual member strength to the load corresponding to the SSE ground acceleration of 0.67g and
2. The inelastic energy absorption factor, F_μ , related to the ductility of the structure.

Associated with the median strength factor $\check{F}_s$ and the median ductility factor F_μ are the corresponding logarithmic standard deviations β_s and β_μ . The structure strength factors of safety and logarithmic standard deviations vary from structure to structure and according to the different failure modes within a given structure.

Based on the characteristics of the lognormal distribution, factors of safety and logarithmic standard deviations for the various contributing effects can be combined to yield the overall effects. For instance, the capacity factor of safety of a structure F_c is obtained from the product of the strength and ductility factors of safety which in turn, may include effects of more than one variable.

$$F_c = F_s \cdot F_\mu \quad (3-1)$$

The methods of determining the strength and ductility safety factors are discussed in the following sections. The logarithmic standard deviation on capacity, β_c , is found by:

$$\beta_c = \sqrt{\beta_s^2 + \beta_\mu^2} \quad (3-2)$$

As discussed in Section 2.3, the logarithmic standard deviations are composed of both an inherent randomness and uncertainty in the median value.

3.1 Structure Strength

The primary lateral load-carrying systems of the SONGS-1 structures included in this study are constructed from both concrete and structural steel. Concrete block walls occur in portions of some of the structures. For a particular structure, failure modes associated with the various lateral load-carrying systems can be postulated. To determine the structure strength corresponding to a failure mode, it is necessary to estimate the strength of the materials composing the load-carrying system and the load capacity of the elements of the system.

3.1.1 Concrete Compressive Strength

The evaluation of the strength of most concrete elements whether loaded in compression or shear, is based on the concrete compressive strength, f'_c . Concrete compressive strength used for design is normally specified as some value at a specific time from mixing (for example, 28 days). This value is verified by laboratory testing of mix samples. The strength must meet specified values allowing a finite number of failures per number of trials. There are two major factors which justify the selection of a median value of concrete strength somewhat above the design strength.

1. To meet the design specifications, the contractor attempts to create a mix that has an "average" strength somewhat above the design strength.
2. As concrete ages, it increases in strength.

For the original SONGS-1 construction, different mixes meeting certain requirements were used. The minimum specified 28-day compressive strengths and primary uses for these mixes are listed in Table 3-1. The results of strength testing conducted to verify the adequacy of concrete supplied are available for most of the mixes. Mean 28-day strengths and coefficients of variation based on this data are also listed in Table 3-1. The median value can be taken as being approximately equal to the mean for a small logarithmic standard deviation (approximate coefficient of variation). The factor relating mean (or median) test strength to minimum specified test strength ranges from 1.2 to 1.7.

As concrete ages, its strength increases and this must also be accounted for in determining the median strength compared to the design strength. Figure 3-1 from Reference 8 shows the increase of the concrete compressive strength with time for several different curing methods. It is assumed that the concrete poured in the field is represented by the curve designated as "air cured, dry at test". At 28 days, the concrete has a relative strength of 50% which approaches 60% asymptotically. The factor relating the strength of aged concrete to the 28-day design strength is therefore 1.2. At 90 days, the concrete has a relative strength of 55% so that the factor relating the strength of aged concrete to 90-day design strength is 1.09. These factors are believed to be median-centered. No information is available on the standard deviation expected for aging. The logarithmic standard deviations for the 28- and 90-day aging factors are estimated to be 0.10 and 0.05, respectively.

Increasing the mean (approximate median) 28-day compressive strengths for the original SONGS-1 construction mixes listed in Table 3-1 by the median 28-day aging factor provides estimates of the median compressive strengths.

$$\check{f}'_c = \check{F}_{a,28} \cdot \check{f}'_{c,28} \quad (3-3)$$

where $\check{f}'_c$ = median concrete compressive strength
 $\check{F}_a$ = median 28-day aging factor
 $\check{f}'_{c,28}$ = median 28-day compressive strength

The logarithmic standard deviations can be estimated by combining the test logarithmic standard deviations listed in Table 3-1 with the logarithmic standard deviation associated with aging in accordance with Equation 2-11.

$$\beta = \sqrt{\beta_{\text{test}}^2 + \beta_a^2} \quad (3-4)$$

where β = logarithmic standard deviation
 β_{test} = logarithmic standard deviation determined by testing
 β_a = logarithmic standard deviation associated with aging

The estimated median compressive strengths and logarithmic standard deviations for each mix are listed in Table 3-2. Values have been estimated for mixes for which no test data was supplied. The ratio of median-to-minimum specified strength for the concrete mixes of primary interest in this study ranges from 1.3 for Mix No. 11 to 1.8 for Mix No. 7.

SONGS-1 structures not included in the original plant construction include the sphere enclosure and diesel generator buildings. The concrete used in more recent construction was not identified by the mix categories of the original construction. The walls and roof slab of the sphere enclosure building were poured with concrete identified as Mix T-C-1P-D. This mix had a specified minimum compressive strength of 6,000 psi at 90 days. Test reports supplied for the sphere enclosure building show a mean compressive strength at 90 days of 7,120 psi with a coefficient of variation of 0.05. When these values are combined with the median 90-day aging factor and its variability, a median compressive strength of 7,700 psi with a logarithmic standard deviation of 0.07 is estimated for the sphere enclosure building.

Several different concrete mixes were permitted for construction of the diesel generator walls, slabs, and columns. In general, these mixes were required to have a minimum compressive strength of 4,000 psi at 28 days. Due to the limited number of test reports supplied for the diesel generator building, test strengths for these different mixes were combined to form a single data set. A mean compressive strength at 28 days of 5,300 psi with a coefficient of variation of 0.05 is indicated. When these values are combined with the median 28-day aging factor and its variability, a median compressive strength of 6,300 psi with a logarithmic standard deviation of 0.11 is estimated for the diesel generator building.

Other effects which could conceivably be included in the concrete strength evaluation include some decrease in strength in the in-place condition as opposed to the test cylinder strength, and some increase in strength resulting from rate of loading at the seismic response frequencies of the structures. Although experimental data on these effects are extremely limited, what is available would tend to indicate these effects are relatively small and of the same order of magnitude and since the two effects are opposite, they were neglected.

3.1.2 Reinforcing Steel Yield Strength

Grade 40 reinforcing steel was used for concrete structures that were part of the original SONGS-1 construction. Grade 60 steel was used exclusively in the sphere enclosure building. For the diesel generator building, bar sizes No. 11 and smaller were specified to be Grade 40 while bar sizes Nos. 14 and 18 were specified to be Grade 60. Reference 9 reported mean yield strengths and coefficients of variation for reinforcing steel of different grades and sizes and from various sources. Mean yield strengths and coefficients of variation for Grade 60 reinforcing steel applied to other nuclear power plants were presented in References 10 to 12.

Two other effects must be considered when evaluating the yield strength of reinforcing steel. These are the variations in the cross-sectional areas of the bars and the effects of the rate of loading. A survey of information (Reference 9) determined that the ratio of actual to nominal bar area has a mean value of 0.99 and a coefficient of variation of 0.024. The same reference notes that the standard test strain rate of loading is 0.0104% strain/sec. The rate of loading anticipated in seismic response of structures is expected to be greater than this rate.

Based on the data available from other structures and the influence of actual bar size and loading rate effects, the following values are estimated for the SONGS-1 structures.

	$\bar{f}_y$ (ksi)	β_{fy}
Grade 40 steel	48	0.10
Grade 60 steel	66	0.08

3.1.3 Structural Steel Yield Strength

Structural steel used in the SONGS-1 structures generally conformed to ASTM A36 or A572 Grade 50 specifications. Actual test data for the SONGS-1 structural steel was not available. Reference 13 reported mean yield strengths and coefficients of variation from a number of sources and recommended the following values:

Flanges of rolled shapes	$\check{f}_y = 1.05 f_y$	$\beta_{fy} = 0.10$
Plates and webs of rolled shapes	$\check{f}_y = 1.10 f_y$	$\beta_{fy} = 0.11$

These estimates are for static yield stresses and reflect a 4 ksi reduction from mill test values due to rate of loading effects. This reduction corresponds to a mill test strain rate of 800×10^{-6} in/in/sec. Strain rates of loading due to seismic response of steel structures are expected to be greater than this value. Median yield strengths for seismic structure fragility calculations are thus conservatively predicted by mill testing.

The differentiation between flanges and webs of rolled sections is inconvenient for fragility calculations. Since significantly less test data were available for flanges, only the webs and plate data were considered. Based on a survey of this data presented in Reference 13, the following values were used for the SONGS-1 structures:

	$\check{f}_y$ (ksi)	β_{fy}
A36 steel	44	0.08
A572 Grade 50 steel	57	0.06

3.1.4 Concrete Block Compressive Strength

Concrete block walls, both load bearing and non-load bearing, are located in some of the SONGS-1 structures included in this study. Similar to concrete, the strength of concrete block walls whether loaded in compression or shear, is correlated to the compressive strength f'_m . The compressive strength is a function of the strengths of the block units and mortar used. Specifications for the original concrete block wall construction

(Reference 14) required that the block units conform to ASTM C-90 Grade A. Limited test data for the concrete block units indicated a compressive strength based on net cross-sectional area of approximately 2,800 psi. The mortar mix design, equivalent to Type S per current ASTM C-270 specifications, was required to have a minimum 28-day compressive strength of 2,000 psi.

Code requirements for concrete block design presented in ACI 531-79 (Reference 15) permit two methods for the determination of the compressive strength f'_m . One involves testing of prisms constructed of the same materials and in the same manner as the actual member and comparing the test strength to the design strength. The other estimates the compressive strength from the unit strength and mortar type through Table 4-3 of Reference 15 which is reproduced in Table 3-3. These values are dependent on construction inspection meeting requirements to ensure adequate workmanship. The block wall workmanship on nuclear power plants is typically good. For the SONGS-1 concrete block walls, a design compressive strength of approximately 1,600 psi would be predicted. A median compressive strength of 1,800 psi was used in strength calculations with the increase of 10% accounting for design conservatism. Possible strength increase due to aging was not included because the age of the units at testing was unknown. Logarithmic standard deviations associated with randomness and uncertainty were estimated to be 0.10 and 0.15, respectively.

3.1.5 Shear Strength of Concrete Walls

There are several potential modes of failure for each of the SONGS-1 structures evaluated in this study. To determine the fragility curve for each failure mode, it is necessary to estimate the median strength capacities associated with the structural elements involved. As an example, this section illustrates the derivation of the median shear strength of a reinforced concrete wall. Development of median strength capacities for other failure modes is similar.

Recent studies have shown that the shear strength of low-rise concrete shear walls with boundary elements is not accurately predicted by the ACI 318-77 Code provisions (Reference 16). This is particularly true for

walls with height-to-length ratios in the order of 1 or less. Barda, et al (Reference 17), determined that the ultimate shear strength of low-rise walls they tested, could be represented by the following relationship:

$$\begin{aligned} V_u &= V_c + V_s \\ &= 8.3 \sqrt{f'_c} - 3.4 \sqrt{f'_c} (h_w/\ell_w - 0.5) + \rho_u f_y \end{aligned} \quad (3-5)$$

where

- V_u = Ultimate shear strength, psi
- V_c = Contribution from concrete, psi
- V_s = Contribution from steel reinforcement, psi
- f'_c = Concrete compressive strength, psi
- h_w = Wall height, in
- ℓ_w = Wall length, in
- ρ_u = Vertical steel reinforcement ratio
- f_y = Steel yield strength, psi

The contribution of the concrete to the ultimate shear strength of the wall as a function of h_w/ℓ_w is shown in Figure 3-2. Also shown in Figure 3-2, are the applicable test values (References 17 to 20) and the corresponding ACI 318-17 formulation. The tests included load reversals and varying reinforcement ratios and h_w/ℓ_w ratios. Web crushing generally controlled the failure of the test specimens. Testing was performed with no axial loads, but an increase in shear capacity of $N/4\ell_w h$ was recommended, where N is the axial load in pounds, and h is the wall thickness in inches.

The contribution of the steel to the ultimate shear strength according to ACI 318-77 is

$$V_s = \rho_h f_y \quad (3-6)$$

where ρ_h = horizontal steel reinforcement ratio.

Furthermore, one of the conclusions reached in Reference 19 is that for low-rise shear walls (specifically $h_w/\ell_w \leq 1$), vertical steel has no effect, and the entire contribution to shear strength is due to the horizontal steel.

In order to estimate the effect that the horizontal and vertical steel have, the steel contribution to wall shear strength was determined from test values for the range $0.5 < h_w/\ell_w < 2$. Test data from References 17 to 20 were used. The effective steel shear strength was assumed in the form

$$V_{se} = AV_{su} + BV_{sh} \quad (3-7)$$

where A, B are constants

$$V_{su} = \rho_u f_y = \text{vertical steel contribution to shear strength}$$

$$V_{sh} = \rho_h f_y = \text{horizontal steel contribution to shear strength}$$

The constants A and B were then calculated assuming the concrete contribution to the ultimate strength is given as shown in Equation 3-5. Based on the results of this evaluation, the constants A and B can be shown to be

$A = 1$	$B = 0$	$h_w/\ell_w \leq 0.5$
$= 2.0 (h_w/\ell_w) + 2.0$	$= 2.0 (h_w/\ell_w) - 1.0$	$0.5 < h_w/\ell_w \leq 1.0$
$= 0$	$= 1$	$1.0 < h_w/\ell_w$

and the median ultimate shear strength is given by

$$\begin{aligned} V_u &= V_c + V_{se} \\ &= 8.3 \sqrt{f'_c} - 3.4 \sqrt{f'_c} \left[(h_w/\ell_w - 0.5) + \frac{N}{4\ell_w h} \right] + \rho_{se} f_y \end{aligned} \quad (3-8)$$

where $\rho_{se} = A\rho_u + B\rho_h$ with A and B determined as shown above.

Based on an evaluation of the same experimental data, the logarithmic standard deviation was calculated to be 0.15.

3.2 STRUCTURE DUCTILITY

A much more accurate assessment of the seismic capacity of a structure can be obtained if the inelastic absorption of the structure is considered in addition to the strength capacity. No inelastic analyses are being performed during this phase of the AFWS project, so that these effects must be estimated by other means. One tractable method involves the use of ductility modified response spectra to determine the deamplification effect resulting from the inelastic energy dissipation. Early studies indicated the deamplification factor was primarily a function of the ductility ratio, μ , defined as the ratio of maximum displacement to displacement at yield. Recent studies (Reference 21) have shown that for single-degree-of-freedom systems with resistance functions characterized by elastic-perfectly plastic, bilinear, or degrading models, the shape of the resistance function is, on the average, not particularly important. However, as opposed to the earlier studies, more recent analyses have shown the deamplification factor is also a function of the system damping. For systems in the acceleration region of the spectrum (i.e., approximately 2 hz and above), Figure 3-3 from Reference 21 shows the deamplification function for several damping values as a function of the ductility ratio.

The studies on inelastic structures conducted to date have been primarily concerned with single-degree-of-freedom systems. Consequently, considerable uncertainty exists in the application of these techniques to multi-degree-of-freedom systems. Questions of the appropriate system ductility to use once individual element or story ductilities have been determined exist, and the relationships between deamplification factors for acceleration and shear forces for MDOF systems is unknown. The assumption made in this investigation is that the average or system ductility expected will be close to the story ductility ratio for well-designed structures. For buildings with large localized ductilities, however, this assumption can be nonconservative.

Ductility ratios for reinforced concrete shear walls failing in shear from reversed loads were calculated from load deflection envelopes given in Reference 20. Only walls tested with axial loads were included in this evaluation. Assuming a lognormal distribution and using the applicable test data from Reference 20, a median shear wall ductility ratio of approximately 4 was calculated. The corresponding logarithmic standard deviation was calculated to be 0.18. The system ductility of structures as complex as some of the SONGS-1 concrete structure may be less than the individual wall ductilities. Expected ductilities for the individual failure modes for the SONGS-1 structures were not evaluated. These ductilities require a consideration of the load distribution throughout the structures which were not available at the time the program was discontinued. Also, some consideration to the relative order of the failure modes together with expected structural damage resulting from lower capacity modes is necessary which again depends on the seismic load distribution.

3.3 IDENTIFICATION OF POTENTIAL FAILURE MODES

Under a seismic event, the load-resisting elements of redundant structures such as those at SONGS-1 are stressed to different levels. A number of failure modes can be postulated for each structure. The likelihood of occurrence for a particular failure mode is dependent on the seismic load paths available, the distribution of inertial loading to these load paths, the resistances of the individual structural elements involved, and the degree of inelastic energy absorption available. Each structure was screened based on estimates of these variables to compile a list of potential failure modes that may be expected due to an earthquake. Again, it should be emphasized that structural failure is defined as occurring when inelastic seismic deformations increase to the extent that the operability of safety-related equipment cannot be guaranteed and does not necessarily imply collapse of the structure. In addition, several of the failure modes included for each structure may not necessarily cause damage to the auxiliary feedwater system directly. However, their occurrence could lead to a loss-of-function of other equipment housed within the

structure whose failure could influence the operation of the auxiliary feedwater system (i.e., impact of the missile shield onto the reactor pressure vessel).

Potential failure modes for the SONGS-1 structures are described in the following chapters. As previously noted, final analytical results for seismic response were not made available. Because of this, there is no way to rank the likelihood of occurrence of the failure modes for a particular structure relative to each other, nor is it possible to determine if the failure modes postulated occur at earthquake levels that can be rationally expected.

Identification of these failure modes was conducted with a significant degree of engineering judgment to predict load distributions to the different elements within each structure. If analytical results indicate load distributions different from those assumed, it may be necessary to reevaluate some of these failure modes. For each failure mode, only median strengths and variabilities, calculated by the time effort on this study was discontinued, are presented. Many of these strengths are dependent on the relative magnitudes of loads due to different earthquake directional components occurring simultaneously. Because final analytical results were not available, phasing between the individual directional component responses is unknown. It may be necessary to reevaluate some of these median strengths to account for response phasing if this information becomes available.

TABLE 3-1

ORIGINAL SONGS-1 CONCRETE MIXES

MIX NO.	PRIMARY USE	MINIMUM 28 DAY STRENGTH (PSI)	MEAN 28 DAY STRENGTH (PSI)	COV
1	Electrical duct encasement	2000	3450	0.081
2	Containment vessel foundation	2000	—	—
3	Miscellaneous yard structures, slabs on grade, building and equipment foundations	2500	—	—
4	" "	2500	3650	0.024*
5	Supported floor slabs (except prestress), beams, walls, retaining walls	3000	3740	0.12
6	Turbine pedestal foundation, intake structure, anchor blocks	3000	3600	0.11
7	Shielding concrete (except pumped concrete)	3000	4450	0.11
8	Pumped concrete in containment vessel	3000	—	—
9	Prestressed decks	4000	5160	0.096
10	Circulating water system gates	4000	5180	0.11
11	Turbine pedestal superstructure	4000	4370	0.089

*-Limited data set

TABLE 3-2

MEDIAN CONCRETE COMPRESSIVE STRENGTHS

MIX NO.	MEDIAN COMPRESSIVE STRENGTH (PSI)	β
1	4100	0.13
2	4100	0.13
3	4400	0.13
4	4400	0.13*
5	4500	0.15
6	4300	0.15
7	5300	0.15
8	5300	0.15
9	6200	0.14
10	6200	0.14
11	5200	0.13

*-Increased slightly since data set was small

Table 3-3

Masonry Compressive Strength For Design

(From Reference 15)

Compressive test strength of masonry units, psi, on the net cross-sectional area	Compressive strength of masonry f'_m , psi	
	Type M and S mortar	Type N mortar
6000 or more	2400	1350
4000	2000	1250
2500	1550	1100
2000	1350	1000
1500	1150	875
1000	900	700

FIGURE 3-1. EFFECTS OF TIME AND CURING CONDITIONS ON CONCRETE STRENGTH (AFTER REFERENCE 8)

FIGURE 3-2. STRENGTH OF CONCRETE SHEAR WALLS

FIGURE 3-3. DEAMPLIFICATION FACTORS FOR ELASTIC-PERFECTLY PLASTIC SYSTEMS IN THE ACCELERATION AMPLIFIED RANGE (FROM RIDDELL AND NEWMARK, (REFERENCE 21))

4. REACTOR BUILDING

4.1 REACTOR BUILDING DESCRIPTION

The SONGS-1 reactor building is the concrete internal structure within the steel containment sphere. It provides support for the reactor vessel, steam supply system, refueling cavity, and service crane. The reactor building structure includes the primary shield wall, secondary shield walls, refueling canal, refueling deck, steam generator compartments, and missile shields. Figure 4-1 shows the reactor building structure and the location of the major pieces of equipment that it supports. Figure 4-2 identifies the major structural elements comprising the reactor building.

The reactor building has an interior concrete foundation that was poured against the steel containment sphere. The interior foundation transmits loads from the reactor building through the containment sphere to an exterior foundation. There is no structural connection between the containment sphere and either foundation.

The primary shield wall encloses the reactor vessel and provides radiation protection. It is cylindrical in shape with outside and inside radii of 16'-3" and 10'-0", respectively and rises from the foundation to Elev. 17'-3". Steel reinforcement was placed in both the tangential and meridional directions at both faces. The primary shield wall was poured integrally with some of the refueling canal walls (see Figure 4-3).

The secondary shield walls are typically 5 feet thick and form a rectangular section 80'-0" by 84'-6" in plan as shown in Figure 4-3. They enclose much of the equipment located in the

reactor building. These walls rise from the foundation to the refueling deck and provide support for the refueling deck as well as other intermediate floors.

The refueling canal walls are also shown in Figure 4-3. They provide support for the refueling deck as well as forming the boundaries of the refueling cavity. Portions of these walls are supported by the primary shield wall while the rest descend to the floor of the refueling canal at Elev. 2'-0". The fuel transfer tube from the fuel storage building enters through the south side of the reactor building. Substantial reinforcement was provided to resist hydrostatic pressures during refueling operations.

The refueling deck is a 3 foot thick slab at Elev. 42'-0" that provides working space for refueling operations. There are several openings in the slab to provide room for the steam generators and access to equipment located below (see Figure 4-1). The service crane travels on runways rising above the refueling deck. The runways are constructed from structural steel that consist of a series of A-shaped frames, diagonally braced longitudinally, that support a single crane girder spanning E-W. A sketch of a runway is shown in Figure 4-4. There is an extension to the refueling deck at the south side of the reactor building that is partially supported by steel columns spanning to the foundation. Traffic from the personnel airlock and equipment hatch pass over the extension.

The steam generator compartments are concrete wall and slab enclosures included for missile protection of the steam generator units. The compartment walls also provide lateral support for the upper ends of the steam generators. The compartments are supported by the refueling deck and rise to Elevs. 64'-6" and 66'-6".

The missile shield and auxiliary shield were included for additional protection of the reactor vessel. They are both removable rectangular reinforced concrete slabs. The missile shield spans horizontally directly over the reactor vessel and is supported by the refueling deck. A sketch of the missile shield and its holdown detail is shown in Figure 4-5. The auxiliary shield spans vertically to the south of the reactor vessel. It is lowered by the service crane onto tracks embedded in the east and west refueling canal walls.

4.2 POTENTIAL REACTOR BUILDING FAILURE MODES

Primary resistance to seismic loading for the reactor building is provided by the heavy reinforced concrete walls which rise from the foundation to the refueling deck. These walls include the primary and secondary shield walls and the refueling canal walls. Although it is discontinuous at some locations due to slab openings, the refueling deck acts, at least in part, as a diaphragm distributing seismic inertial load from supported equipment, the steam generator compartments, and its own mass to the major shear walls. Seismic forces resisted by the shear walls are collected by the interior foundation and transmitted through the steel containment sphere to the exterior foundation and out to the supporting soil. Due to the lack of a positive, mechanical connection between the interior foundation and the steel containment sphere, the possibility of these two structures sliding relative to each other at the interface exists.

In addition to failure modes associated with elements composing the overall structural system, there are other potential failure modes associated with substructures supported by the reactor building. These smaller structures include the missile shield over the reactor pressure vessel and the service crane runway. Their failure was considered in this evaluation following the selection criteria described in Section 3.3. Although these structural failures are localized and do not directly lead to loss-of-function of the auxiliary feedwater system, their

occurrence could cause damage to other safety-related systems located in the reactor building.

As noted previously, seismic response results for the reactor building produced by an analysis conducted to provide comparison to other models were received. However, these results were supplied only to give preliminary estimates of seismic response and were not used to develop structural fragilities. Only estimated median element strengths and their variabilities are presented in the following sections.

4.2.1 Primary Shield Wall Failure

The primary shield wall functions as part of the overall structural system resisting seismic loads. Seismic forces are transmitted to the primary shield wall from the refueling deck through some of the refueling canal walls. Between Elevation -3'-6" (bottom of refueling canal slab) and Elevation 17'-3" (top of primary shield wall), the primary shield wall and the east and west refueling canal walls form an integral structure. The primary shield wall can be considered load-bearing since the refueling canal walls above provide vertical support for the refueling deck slab.

Failure of the primary shield wall is expected to lead to damage to piping penetrating the concrete walls through to the reactor pressure vessel and to other attached equipment. Because it is a major component of the overall lateral load-resisting system, primary shield wall failure may also lead to gross structural failure of the reactor building, although this result is dependent on the relative magnitudes of loads compared to capacities in the secondary shield walls as well. Primary shield wall failure can occur in tangential shear under horizontal shear forces or in flexure due to overturning moments. Experimental research conducted to study the strength of thick-walled reinforced concrete cylinders was reported in Reference 22. The data available and conclusions reached formed

the basis for estimating the median shear strength of the primary shield wall. In addition, data in Reference 22 seems to confirm the application of the basic design principles of Reference 16 with modifications to eliminate design conservatisms to predict the flexural strength of thick-walled cylinders. Shear failure is expected to occur in the section independent of the refueling canal walls between Elevations -3'-6" and 17'-3". The median shear strength is estimated to be approximately 14,000 k with logarithmic standard deviations associated with randomness and uncertainty of approximately 0.10 and 0.18, respectively. Flexural failure is expected to occur where the primary shield wall intersects the interior foundation (Elevation -10'-0"). The median flexural strength is estimated to be approximately 280,000 k-ft with logarithmic standard deviations associated with randomness and uncertainty of approximately 0.13 and 0.15, respectively.

4.2.2 Secondary Shield Wall Failure

The secondary shield walls also function as part of the overall reactor building lateral load-resisting system. They are located around the perimeter of the reactor building as shown in Figure 4-3 and together form a hollow box-shaped section. The secondary shield walls provide vertical support for the refueling deck slab and thus can be considered load-bearing. Failure of the secondary shield walls is expected to lead to gross structural failure of the reactor building due to the large portion of capacity associated with them.

The secondary shield walls are expected to receive much of their seismic load from the refueling deck. Failure of these walls is expected to occur due to in-plane shear rather than flexure because of the relatively low wall height and the additional flexural capacity provided by the transverse, intersecting walls. The location of failure is expected to be somewhere between the slab poured on the interior foundation (Elevations 1'-6" or 2'-9") and the refueling deck. Approximate median

shear strengths of the secondary shield walls, listed in Table 4-1, were estimated using the equations presented in Section 3.1.5. Logarithmic standard deviations associated with randomness and uncertainty are estimated to be approximately 0.10 and 0.15, respectively for these walls.

4.2.3 Refueling Canal Wall Failure

The refueling canal walls also provide vertical support for the refueling deck and receive part of its seismic inertial loading. Failure of the refueling canal walls is expected to lead to gross structural failure of the reactor building. This failure should occur due to in-plane shear force rather than overturning and should be located through the section of wall from the top of the primary shield wall to the refueling deck. Approximate median shear strengths were estimated using the equations presented in Section 3.1.5 and are listed in Table 4-1. Logarithmic standard deviations associated with randomness and uncertainty are estimated to be approximately 0.10 and 0.15, respectively, for these walls.

4.2.4 Refueling Deck Diaphragm Failure

The refueling deck slab provides vertical support for the steam generator compartments and equipment mounted at the refueling deck. It is punctuated by several openings for the refueling canal, steam generators, and equipment access hatches. Although the primary purpose of the refueling deck slab is to serve as a vertical load-carrying member spanning between the secondary shield and refueling canal walls, it must also transfer horizontal seismic inertial forces from the mass it supports to these same walls through in-plane shear. Because of the discontinuity in the load paths caused by the slab openings, it was necessary to review the sources of diaphragm shear as well as diaphragm strength. Based on this review, diaphragm failure is expected to occur at the southeast and northwest corners of the refueling deck. Both of these locations are adjacent to openings for steam generator units

which reduce the length of slab available for transmitting stresses to the secondary shield walls. Shear strength of diaphragms is believed to be adequately predicted by equations developed for estimating the strength of shear walls (Section 3.1.5). The slabs of the southeast and northwest corners of the refueling deck are estimated to have approximate median shear strengths of 5500 k and 3100 k, respectively. Logarithmic standard deviations associated with randomness and uncertainty are estimated to be approximately 0.10 and 0.18, respectively, for these diaphragms.

Another possible location for diaphragm failure is at the interface between the south secondary shield wall and the refueling deck extension. The refueling deck extension slab extends off the south side of the refueling deck and is supported by steel framing. The steel columns are not laterally braced so in-plane diaphragm shears must be transferred back to the south secondary shield wall. A vertical construction joint was provided at the interface between the refueling deck extension and the south secondary shield wall. Diaphragm shear stresses are passed across this interface by the steel reinforcement acting in shear friction. Data from research conducted to determine shear-friction strength was reported in Reference 23 and formed the basis for estimating median strengths compared to design values. Dowels were provided for the top and bottom layers of slab reinforcement, but the splice length for the bottom dowels is probably too short to develop the full yield strength. Accounting for possible reduction in the stress that the bottom dowels can develop and for possible strength degradation of shear-friction details under cyclic loading, the median shear strength of this detail is estimated to be approximately 4100 k with logarithmic standard deviations associated with randomness and uncertainty of approximately 0.18 and 0.20.

4.2.5 Sliding At The Interior Foundation - Containment Interface

The interior foundation of the reactor building bears on the steel containment sphere under its own weight. No mechanical connectors (i.e. shear studs or anchors) welded to the steel sphere and embedded in the concrete foundation were provided to transmit radial tensile or tangential shear stresses. At the steel-concrete interface, the weight of the reactor building develops tangential shear stresses that are self-equilibrating and radial compressive stresses that are passed on to the supporting soil. Seismic response of the reactor building will generate tangential shear stresses and radial stresses at the interface. Resistance to sliding under tangential shear stresses can only be provided by static friction developed by the radial compressive stresses. At a sufficiently high seismic response level, static friction will be overcome and the interior foundation will slide against the steel containment sphere with the reactor building pivoting about the sphere center. Sliding is expected to lead to damage to the fuel transfer tube and piping penetrating containment and anchored to the reactor building due to relative end displacements. Using an estimate of the static coefficient of friction for concrete on steel, the resistance against sliding developed by dead weight of the structure, expressed as a moment about a horizontal axis through the sphere center, is estimated to be approximately 630,000 k-ft with logarithmic standard deviations associated with randomness and uncertainty of approximately 0.15 and 0.27, respectively. Calculations indicate that sliding is likely to occur prior to uplift of the foundation due to overturning.

4.2.6 Service Crane Runway

Each service crane runway, shown in Figure 4-4, consists of a W36 x 260 continuous girder supported by a series of steep, A-shaped steel frames. Each frame is composed of a pair of W14 x 111 columns welded together where the flanges meet. The column base detail includes four anchor bolts, 1.25 inches in diameter by 24 inches long, fabricated

from A307 steel. The runways are laterally braced in the longitudinal direction by diagonal steel members. In the transverse direction, resistance to seismic loads is provided by frame action. Because of the steep inclination of the W14 x 111 columns, substantial column axial forces are expected to occur under transverse seismic excitation. Based on preliminary transverse response calculations, failure of the service crane runway is expected to be initiated by yielding of the anchor bolts at a median uplift force of approximately 220 k. Logarithmic standard deviations associated with randomness and uncertainty for this capacity are estimated to be approximately 0.10 and 0.10.

4.2.7 Missile Shield

The missile shield, shown in Figure 4-5, is a rectangular reinforced concrete slab 24 inches thick, spanning between the east and west refueling canal walls above the reactor pressure vessel. Also shown in Figure 4-5 is the holdown detail provided to attach the missile shield to the refueling deck slab that allows removal of the shield for maintenance access. The missile shield essentially acts as a one-way slab spanning between simple supports. Out-of-plane forces are generated in the missile shield as the vertical seismic excitation is amplified up through the reactor building and as the slab vibrates due to its own dynamic characteristics. In addition, in-plane forces are generated by horizontal seismic accelerations at the refueling deck with the missile shield acting as a rigid body for these motions.

Failure of the missile shield is expected to occur either in the slab itself or in the bolts of holdown detail. The median out-of-plane flexural strength of the missile shield was estimated to be approximately 2700 k-ft across the total slab width with logarithmic standard deviations associated with randomness and uncertainty of approximately 0.13 and 0.12.

The two holdown bolts provided at each detail are one inch in diameter and specified to be fabricated from high strength steel for a friction-type connection. These bolts are believed to have been preloaded to function as friction-type, so they are loaded in both tension and shear under seismic response. Assuming these bolts were fabricated from A325 steel, the tensile strength in the absence of shear load was estimated to be 79 k and the shear strength in the absence of tensile load was estimated to be 47 k. Failure of bolts under combined shear and tension forces, as defined by the interaction model presented in Reference 24, is dependent on the magnitudes of the shear and tension forces occurring simultaneously. It is necessary to estimate the preload tension applied to the bolts. Because final seismic response analytical results giving simultaneously shear and tensile bolt forces were not supplied, final failure capacity of the bolts could not be predicted.

TABLE 4-1
REACTOR BUILDING MEDIAN SHEAR WALL STRENGTHS

WALL	APPROXIMATE MEDIAN SHEAR STRENGTH (k)
North Secondary Shield Wall	40,000
South " " "	39,000
East " " "	42,000
West " " "	40,000
North Refueling Canal Wall	10,000
East " " "	30,000
West " " "	35,000

FIGURE 4-1. SONGS-1 REACTOR BUILDING AND SUPPORTED EQUIPMENT

FIGURE 4-2. REACTOR BUILDING STRUCTURE

FIGURE 4-3. MAJOR REACTOR BUILDING WALLS

Section A

FIGURE 4-4. SERVICE CRANE RUNWAY

Plan View

Detail 1

Section A

FIGURE 4-5. MISSILE SHIELD

5. CONTAINMENT SPHERE

5.1 CONTAINMENT SPHERE DESCRIPTION

The SONGS-1 containment sphere (Figure 4-1) is a 140 foot diameter spherical steel shell approximately 1-inch-thick. The sphere center is located at Elevation 50'-0". Shell plate was required to conform to ASME specification SA-212 Grade B Fire Box Quality produced to SA-300 specification with fine grain melting practice and with guaranteed Charpy values of minus 50⁰F from transverse specimens. Seam welds in the shell were specified to be of the double bevel butt type.

The sphere is supported by a concrete foundation mat having an outside radius of 73'-6" up to a tangent point at Elevation 2'-7". A partial cross-section of the sphere and its foundation near grade is shown in Figure 5-1. The foundation is reinforced only above Elevation 12'-0". There are no positive connections between the shell and the foundation and interface stresses are transferred by bearing or friction. Near grade, the shell and the foundation are separated by a 1-foot-wide sand cushion intended to reduce discontinuity stresses. As noted in Section 4.1, the reactor building foundation was poured against the shell.

Major penetrations include the equipment hatch, personnel airlock, escape airlock, and steam and feedwater lines. The equipment hatch is 15 feet in diameter and located at Elevation 47'-0" on the south side along the sphere axis. The personnel airlock is approximately 10 feet in diameter and located at Elevation 47'-6" just to the east of the equipment hatch. The escape airlock is approximately 7 feet in diameter. It penetrates the sphere just above grade in the northeast quadrant. The steam and feedwater lines enter containment from the north turbine extension parallel to the N-S axis.

5.2 POTENTIAL CONTAINMENT SPHERE FAILURE MODES

The containment sphere resists its gravity and seismic loads as a spherical shell. These loads are transmitted down to the foundation primarily by shell membrane stresses. Additional local stresses, including bending stresses, are caused by the discontinuity occurring where the sphere contacts the foundation. The sand cushion between the shell and the exterior foundation was included as a transition in order to reduce these discontinuity effects. Additional local stresses are introduced by other discontinuities such as pipe penetrations and hatches.

Design of the containment sphere was dictated, for the most part, by a Loss-of-Coolant-Accident (LOCA). This study did not attempt to consider simultaneous LOCA and earthquake loadings. Tensile yielding of the sphere due to dead and seismic loads is expected to occur at relatively high earthquake levels. The most likely mode of failure for the containment sphere is buckling due to membrane compressive stress.

5.2.1 Failure Due to Buckling

Overturning moment due to horizontal seismic response is expected to be resisted for the most part by the membrane stresses in the sphere distributed according to the first harmonic. Membrane compressive stresses may lead to buckling of the sphere if seismic loading is significant. Closed-form solutions for the buckling stress of a spherical shell with the particular loading and boundary conditions of the SONGS-1 containment are not available. The buckling stress would be best predicted by a computer-generated solution modeling the actual loading and boundary conditions. However, this type of analysis had not been performed at the time work on this study was discontinued.

An estimate on the possible range of the buckling stress may be obtained from the upper and lower bounds. A possible upper bound buckling stress corresponds to the solution for a complete spherical shell subjected to uniform external pressure. Using the equation presented in Reference 29 for this case, an upper bound buckling stress of 21,000 psi

is estimated. This solution does not account for the influence of imperfections or local membrane or bending effects. A possible lower bound solution may be taken from design allowable stress presented in Reference 40 for shallow, spherical caps. The equation presented in Reference 40 includes some factor of safety against buckling and was developed from available test data. The lower bound buckling stress estimated from this equation is 6000 psi.

6. SPHERE ENCLOSURE BUILDING

6.1 SPHERE ENCLOSURE BUILDING DESCRIPTION

The sphere enclosure building was added to the SONGS-1 plant after original construction was completed to provide additional radiation shielding for the area surrounding containment. It encloses the containment sphere, but is structurally separated except for common penetrations such as the escape lock. The structure is a concrete circular cylinder having an inside diameter of 145 feet with a conical roof that rises to approximately 177 feet above grade. The centerline axis coincides with that of the containment sphere.

A typical section through the sphere enclosure building wall is shown in Figure 6-1. Foundation support is provided by a continuous wall footing. The top of the footing lies just below grade. The footing and the containment sphere foundation are typically separated by soil backfill. Where the wall footing is widened, a gap filled with expansion joint material isolates the two structures. Sections through these two types of footings are shown in Figure 6-2.

The cylinder wall is typically 3 feet thick. It is conventionally reinforced by tangential and meridional steel bars at both the inside and outside faces. Large openings which permit the passage of piping to the containment sphere occur at the south and west sides of the sphere enclosure building. As shown in Figure 6-3, wall reinforcement above and on the sides of the openings are increased and tied together to transfer stresses around these discontinuities. Also, at the top of the wall, reinforcement is increased and tied together to form a ring girder. This ring girder, shown in Figure 6-4, helps to resist tensile tangential membrane stresses due to the thrusts from the inclined roof.

The roof slab, shown in Figure 6-5, consists of 18 inches of concrete poured in two lifts on 3 inch deep metal decking which serves as a form only to give a total thickness of 21 inches. The roof is inclined at a slope of 1 to 3. Radial and tangential steel reinforcement was included in three separate layers. Dowels from the wall pass through a vertical construction joint at the wall-roof interface - vertical stirrups hooking around the bottom steel layer serve to tie the two concrete lifts together.

Structural steel framing was included to support the roof slab prior to setting of the concrete. W 36 x 160 girders span radially and support intermediate beams between them (see Figure 6-6). The girders rest on corbels cast integrally with the wall at one end and are pinned to a central compression ring at the other. 7/8 inch diameter headed studs welded to the top flanges of the steel roof members are embedded in the concrete slab. The compression ring is a box-shaped section fabricated from steel plate.

6.2 POTENTIAL SPHERE ENCLOSURE BUILDING FAILURE MODES

Most of the mass of the sphere enclosure building is associated with the structure itself as there is relatively little supported equipment. Seismic inertial forces acting on the roof and wall of the sphere enclosure building are transmitted by these same systems down to the foundation. Potential structural failure modes considered the most likely include failure of wall or roof and failure of the soil beneath the wall footing. As previously noted, failure does not necessarily imply structural collapse. However, occurrence of any of the failure modes described in the following sections may lead to damage to the containment sphere or any of the piping penetrating the sphere, including the auxiliary feedwater line.

6.2.1 Wall Failure

Gross failure of the sphere enclosure building wall may occur as seismic forces are transmitted through the wall down to the foundation. Gross failure is characterized by cracking or crushing of the concrete and yielding of the steel reinforcement throughout much of the wall as opposed to local failure which is associated with structural discontinuities. Horizontal seismic shear action on the cylindrical cross-section is expected to be resisted primarily by tangential shear stresses. Reference 25 reported the results of testing conducted on thin-walled reinforced and prestressed concrete cylinders loaded to failure in shear. Data summarized in this study was used by the authors of Reference 25 to correlate test shear strength to different models for predicting shear strength. The conclusions reached were used to estimate the median shear strength of the sphere enclosure building based on a truss analogy described in Reference 25. It was assumed that sufficient additional wall reinforcement was added at the large openings to prevent premature localized failure at the discontinuities prior to development of the overall strength. Redistribution of stresses following local nonlinearity is also expected to reduce the discontinuity effects. Accounting for the reduction in cross-sectional area due to the presence of the large openings, a median shear strength of approximately 17,000 k was estimated.

The overall overturning moment due to horizontal seismic response is expected to be resisted primarily by vertical membrane stresses. Gross flexural failure should occur near the wall foundation where overturning moment is the highest. As with gross shear failure, additional reinforcement and stress redistribution is expected to reduce the discontinuity effects of the large openings. A median flexural strength of approximately 4,700,000 k-ft was estimated by extending the basic principles developed for predicting the strength of rectangular reinforced concrete beams to a hollow, thin-walled, cylindrical section.

This value does not include the possible reduction in strength due to uplift vertical seismic response since the relative magnitudes of horizontal and vertical loads occurring simultaneously were not supplied.

As shown in Figure 6-3, additional horizontal wall reinforcement was provided at the top of the south opening. This reinforcement is tied by stirrups to form a beam spanning across the opening to distribute vertical membrane stresses from the wall above around the discontinuity. In addition to the weight of the wall above the opening and the section of roof it supports, the beam must also carry vertical wall membrane stresses due to horizontal and vertical seismic response. As loads are increased, yielding of the steel reinforcement is expected to lead to stress redistribution which will force the wall directly above the opening to behave as a deep beam. The effective cross-section of the deep beam will be from the top of the opening to the top of the top of the ring girder. Using data available on the strength of deep reinforced concrete beams (Reference 38), the flexural strength at midspan of the deep beam, for the case of tension at the top of the opening, was estimated to be approximately 160,000 k-ft. Flexural strength at the supports (faces of the opening) is dependent on the stress level in the ring girder reinforcement due to dead load and simultaneous vertical seismic response. This term cannot be determined without knowledge of the relative magnitudes of beam forces due to horizontal and vertical response. The shear strength of the deep beam near the supports was estimated to be approximately 21,000 k. Because final seismic response analytical results were not supplied, it was not determined if failure of the wall above the large opening could occur at a credible earthquake excitation level.

6.2.2 Roof Failure

The steel roof framing and metal deck was provided to support the roof slab prior to setting of the concrete and development of its

strength and stiffness. The steel framing is thus expected to carry much of the static roof weight, the magnitude depending on the level to which the first concrete slab lift had hardened prior to pouring of the second lift. Once both lifts have set, subsequent loading is resisted primarily by membrane stresses in the concrete slab which behaves as an edge-supported conical shell. Maximum membrane stresses occur in the shell at the wall support and are compressive in both the hoop and meridional directions for downward loading. In addition, bending moments are developed near the support due to the discontinuity.

Potential failure modes of the roof system due to vertical seismic response include crushing or cracking of the roof slab concrete and yielding of the wall reinforcement in the vicinity of the roof connection. Of these possibilities, yielding of the wall reinforcement is expected to be the most likely. Crushing of the roof slab concrete occurs for seismic inertial loads acting downward. The steel framing does not significantly participate during this type of response, but the capacity of the concrete acting in membrane compression is large compared to the forces generated at credible earthquake levels. If vertical seismic response is sufficient, inertial loads may act upward on the roof. For this direction of loading, membrane stresses in the conical shell are tensile and cracking at the concrete followed by yielding of the reinforcement can occur. However, the additional capacity associated with the steel roof girders is mobilized once the stiffness of the slab is reduced by cracking.

The most likely roof failure mode is expected to be initiated by yielding of the horizontal wall reinforcement in the vicinity of the roof slab connection. Under seismic inertial loading acting downward on the roof, the meridional membrane force in the conical shell must be reacted by a radial shear and a vertical force acting on the wall.

A certain amount of radial shear is also generated by horizontal seismic load from the roof, although most of this load is resisted by tangential shear in the wall. The radial shear induces meridional and hoop membrane forces and bending moments in the wall which decrease away from the roof connection. Additional bending moments caused by the discontinuity of the roof slab are expected to be relatively minor for the ultimate load condition. A specially reinforced ring girder was constructed at the top of the wall to resist the hoop force and moment. The ring girder, shown in Figure 6-4, is 7'-0" deep and reinforced by #11 bars at six inch spacing at both faces with #4 stirrups tying the horizontal reinforcement together. Lap splices were specially detailed to ensure performance under direct tension load. Failure of the roof system due to the lack of restraint against excessive deformations at the top of the wall is expected to be initiated by tensile yielding of the ring girder reinforcement. As the ring girder reinforcement yields, there will be a certain degree of stress redistribution that will activate the additional capacity provided by hoop reinforcement in the wall below the ring girder. The median capacity for failure of the sphere enclosure building roof, expressed in terms of the radial shear acting outwards on the wall, is estimated to be approximately 58 k/ft, including radial shear due to thrust from the steel girders resisting the roof dead load.

6.2.3 Soil Failure

Failure of the soil foundation material, although not a structural failure mode, is dependent on seismic response of the sphere enclosure building and should be considered in a fragility evaluation of SONGS-1. Uplift of the foundation and corresponding decrease in the foundation-soil contact area may occur under base overturning moment at sufficiently high earthquake input levels. As the contact area decreases, contact soil pressure increases significantly and eventually reaches the soil bearing pressure at the toe of the foundation.

Ultimate bearing capacity of the soil foundation material beneath the sphere enclosure building was not available so the overturning moment capacity was not calculated at the time the project was discontinued. The significant lateral deformations associated with foundation uplift can lead to impact with the north turbine extension slab. This event may cause concrete to spall from the structure wall and impact piping entering the containment sphere below. Overturning instability is not a credible mode of failure. This is because the rocking period of the structure becomes very long at large excursions compared to the earthquake forcing frequencies. Resonance cannot occur and the structure tends to become an isolated dynamic system rocking about its c.g. rather than overturning.

FIGURE 6-1. TYPICAL WALL SECTION

FIGURE 6-2. WALL FOOTING

6-11

FIGURE 6-5. CONCRETE ROOF SLAB

FIGURE 6-6. STEEL ROOF FRAMING

7. CONTROL-ADMINISTRATION BUILDING

7.1 CONTROL-ADMINISTRATION BUILDING DESCRIPTION

The control-administration building houses the safety-related control equipment for SONGS-1 reactor activity. It is located to the southeast of the containment sphere and adjacent to the north turbine extension and the east heater platform. The control-administration building is approximately tee-shaped at grade. The safety-related equipment is generally contained in the western portion of the structure which stands three stories high. The switchgear and cable spreading room is in the first story, the HVAC room in the second story, and the control room in the third story. The eastern portion of the structure is primarily devoted to office space for plant administration.

The control-administration building is mostly of reinforced concrete wall and slab construction. A plan view of the first floor is shown in Figure 7-1. Continuous wall footings are located just below grade. The first floor rooms rest on slab on grade four to six inches thick. Walls at the north and west elevations of the western portion of the structure were made thicker than typical for radiation protection. A light structural steel frame with no lateral load-carrying capability was included in the switchgear and cable spreading room to support electrical equipment such as conduit and cable trays. Much of the east elevation at the first story consists of non-structural elements, probably leading to a certain degree of torsional eccentricity under N-S seismic response. The battery room, which contains batteries for backup electrical power, is located at the southeast corner of the control-administration building. The battery room is partially enclosed by reinforced load-bearing concrete block wall.

The second floor, shown in Figure 7-2, is composed of floor slabs located at different elevations. The switchgear and cable spreading room is open at this floor. Steel beams span between the surrounding walls and

steel columns to carry electrical equipment. The floor slab for the classroom is effectively isolated from the other floor diaphragms by openings and grating. The classroom slab, as well as the south and west walls enclosing it, are supported by beams framing into other walls and a single concrete column. This column rises from an isolated square footing just below grade. The eastern portion of the building is covered by a concrete slab approximately 7-inches-thick at Elevation 35'-6". There is a vertical offset of 3'-6" between the roof and the adjacent floor slab. The roof slab is supported by steel beams spanning between walls and steel columns.

The control room and related equipment is located at the third floor (see Figure 7-2). The control room area concrete slab is 12-inches-thick and rests on steel members that are part of the frame also carrying electrical equipment. The north and west walls surrounding the control board access area are lined with solidly grouted 6-inch-thick concrete block units. The concrete block liner is reinforced and anchored to the concrete wall by a grid of 1/2-inch-diameter wej-it gunite hook anchors. Vertical construction joints were typically used at wall-slab intersections throughout the control-administration building. This detail occurring at the west side of the control board access area is shown in Figure 7-4. The floor slab bears on a short ledge formed during pouring of the wall. Dowels were only provided for the top layer of slab reinforcement.

The roof of the control-administration building is located at two separate elevations as noted in Figure 7-3. The slab over the control room area is at Elevation 56' 7-1/2" and is about 2-feet-thick. The rest of the roof is at Elevation 54' 9-1/2".

7.2 POTENTIAL CONTROL-ADMINISTRATION BUILDING FAILURE MODES

The main lateral load-resisting elements of the control-administration building are the reinforced concrete walls which act as shear walls in transmitting horizontal seismic forces down to the foundation. These walls are

tied together horizontally by the concrete floor and roof slabs which behave as diaphragms distributing inertial loads to the vertical elements. Failure of the shear walls is expected to lead to gross structural failure and damage to equipment located throughout the control-administration building. Depending on the nature of their detailing, diaphragm failure may or may not be localized with only damage to equipment located within the vicinity of the local failure sustained.

Other potential failure modes not associated with the overall structural system include concrete block wall failure and failure of the isolated column at the bottom story. The load-bearing concrete block walls enclosing the battery room are tied into the floor diaphragm above. The possibility of these walls receiving seismic shear and failing under in-plane loading exists. They may also fail in flexure due to out-of-plane vibration. The isolated concrete column at the second floor classroom supports a discontinuous shear wall above. Large axial forces leading to failure may be generated in the column as it attempts to resist the overturning moment in the shear wall.

7.2.1 Shear Wall Failure

Reinforced concrete walls are distributed throughout the control-administration building. They function as load-bearing walls as well as forming the primary lateral load-resisting system for horizontal seismic response. The major shear walls are shown in Figures 7-1 to 7-3. Most of the walls are on the order of 1'-0" $\pm$ thick, with the exception of those intersecting at the northwest corner of the switchgear and cable spreading room which are 2'-10" thick.

Failure of the shear walls is expected to occur in shear rather than overturning due to the relatively low height of the structure compared to its plan dimensions. Also, shear walls oriented in the direction of horizontal seismic load are typically intersected by transverse walls which act as boundary elements increasing the flexural wall strength. Identification of the wall that initiates failure is not possible without analytical

results indicating the distribution of seismic loads through each story. However, this wall is expected to be located in one of the two lower stories where seismic shears are greatest. A certain amount of torsional response is anticipated due to changes in the distribution of wall stiffnesses through the height of the structure with corresponding shifts in the centers of rigidity relative to the centers of mass. The influence of torsional moment is greater on exterior walls than interior walls. Using judgment to estimate the load distribution among the walls at each story, the walls identified by number in Figures 7-1 and 7-2 were selected for preliminary capacity evaluation. Using the shear strength equations presented in Section 3.1.5, the median shear strengths of these walls, listed in Table 7-1, were calculated. Because each of these walls resist a significant portion of the overall story shear, the failure of any of them is expected to lead to gross structural failure of the control-administration building.

7.2.2 Diaphragm Failure

For the control-administration building, vertical construction joints were used at the slab-wall interfaces. At this detail, the bottom layer of slab reinforcement was typically discontinued without a dowel provided for continuity of the bars into the wall. This detail is adequate for transfer of slab vertical load moment into the wall since only the top layer of reinforcement is in tension at the support. However, in-plane diaphragm shears due to seismic response must be transferred across the vertical construction joint by shear-friction of the steel reinforcement crossing the plane of slippage. The reduction in reinforcement area available to provide the clamping action necessary for shear-friction indicates a potential for diaphragm failure where this detail occurs.

As with shear wall failure, the location of diaphragm failure cannot be identified without analytical results indicating the distribution of seismic loads. Diaphragm failure is expected to occur at the upper floors where seismic accelerations and thus inertial loads are greatest. Also, failure is expected to occur at the exterior walls where the load-paths must cross the vertical construction joints noted above and torsional

moments increase the diaphragm shear forces. Using judgment to estimate the distribution of seismic load, the diaphragm locations identified by number on Figures 7-2 and 7-3 were selected for capacity evaluation. These connection details are similar to those encountered at the refueling deck extension in the reactor building and similar calculations were conducted to determine diaphragm capacities (see Section 4.2.4). Estimated median diaphragm shear strengths are listed in Table 7-2.

Diaphragm failure does not necessarily lead to gross structural failure. Once the load-path to a wall is lost, the floor inertial load may be resisted by the remaining walls and diaphragms through load redistribution provided they have sufficient capacity. For this case, only damage to equipment located within the immediate vicinity of the failed construction joint may occur. However, if the remaining walls and diaphragms do not have sufficient capacity to resist forces from the altered load distribution upon initial diaphragm failure, floor inertial loads must be carried by the walls transverse to the direction of load in out-of-plane flexure. It is unlikely that the transverse walls have sufficient strength to function in this manner, and gross structural failure may occur as a result of diaphragm failure.

7.2.3 Block Wall Failure

The north and east walls of the battery room located at the first floor of the control-administration building are constructed of concrete block. These walls are load-bearing and reinforced by #4 vertical bars in every third cell and Duowall reinforcement in every other horizontal joint. The vertical reinforcement is doweled into the diaphragm above so horizontal seismic shear will be transmitted into these walls. Failure under in-plane loading may occur in shear or overturning. Available test data and recommended predictive models contained in References 27 to 31 were surveyed to determine in-plane strengths of the reinforced concrete block walls of the control-administration building. For the east and north battery room walls, median in-plane shear strengths were estimated to be approximately 220 k and 180 k, respectively. Median in-plane flexural strengths were estimated to be approximately 3700 k-ft and 3500 k-ft for the east and north walls. These

capacities include the influence of dead load from the roof, but do not reflect possible reduction in strength due to vertical or out-of-plane seismic loads since the relative magnitudes of these loads occurring simultaneously with in-plane loads were not supplied.

Under horizontal seismic excitation, response of the concrete block walls will be amplified as they vibrate out-of-plane. Research conducted on reinforced concrete block walls (summarized in Reference 26) indicates that the ultimate strength approach for reinforced concrete beams provides a reasonable model for the prediction of out-of-plane masonry flexural strength. Using the basic principles described in Reference 16, median flexural strengths, accounting for the difference between median and design strength, possible strength reduction under cyclic load, and vertical dead load only, of approximately 1800 lb-ft/ft and 2300 lb-ft/ft were estimated for the east and north battery room walls, respectively. These capacities do not include possible reduction in strength due to vertical or in-plane seismic loads since the relative magnitudes of these loads occurring simultaneously with the maximum out-of-plane response were not available. Due to the conditions, this capacity can be developed at the top and bottom of the wall as well as midheight with hinging necessary to cause failure.

Upon failure of the concrete block walls, equipment, piping, and conduit attached to the walls will sustain damage due to excessive deformations. Failure of the walls is essentially a vertical collapse. Equipment located at the floors immediately adjacent to the walls will also be damaged. However, equipment separated from the walls by an aisle width or greater is expected to have a good chance of survival.

7.2.4 Isolated Column Failure

The reinforced concrete exterior wall at the southern wing of the control-administration building is continuous from Elevation 32'-0" up to the roof. Below Elevation 32'-0" to grade, the western half of this wall is discontinued with a single concrete column spanning down to a footing below in-

cluded instead at the southwestern corner of the classroom. Although seismic response analytical results for the control-administration building were not supplied, this wall may receive significant seismic load from the floors above Elevation 32'-0". Overturning moment at Elevation 32'-0" must be resisted in part by axial load in the isolated concrete column due to discontinuity of the wall. A similar condition occurs for the wall at the western side of the classroom, but seismic load on this wall is not expected to be as great since it is completely discontinuous at Elevation 32'-0" with a corresponding reduction in overall stiffness.

This column is 18 inches square and reinforced by eight #10 bars with #2 bar ties spaced 18 inches apart. The footing is 5'-6" square in plan and 1'-6" thick with the top of footing at Elevation 12'-0". It is overlain by an 8" slab-on-grade and a thin layer of soil backfill. Calculations indicate that uplift capacity is limited by the gravity load supported by the column rather than the tensile strength of the longitudinal steel reinforcement. As tension force in the column due to wall overturning is increased, the dead weight of the structure carried by the column will eventually be overcome and the footing will lift up off the soil below. The slab-on-grade has insufficient strength to hold the column down and will ride up with the footing. The column axial force due to structure dead load, calculated on the basis of tributary areas, is estimated to be approximately 150 k. This value represents the maximum tensile force that can be developed in the column.

Since the construction of the control-administration building, special design requirements have been adopted to ensure ductile behavior of reinforced concrete structures. These requirements are described in Appendix A of References 16 and 32. Columns supporting discontinuous walls are currently required to have special confinement reinforcement (Section A.6.5.5 of Reference 16). For rectangular hoop reinforcement, a minimum bar area is required at a maximum spacing of four inches. It is clear that the hoop reinforcement in this isolated column (#2 bars spaced at 18 inches) does not meet this requirement and the ability of the column to sustain more than one yield

cycle in compression is doubtful. The median column axial load compressive strength is estimated to be approximately 1700 k total. Once this capacity is reached, significant spalling through the column core and buckling of the longitudinal reinforcement will probably occur. It is expected that any load-carrying capability of the column will be lost in the first or second compressive yield cycle.

Failure of the column may not necessarily lead to further structural damage in this region of the control-administration building because the discontinuous portion of the wall may be able to transfer loads formerly carried by the column back to the portion of wall continuous to the foundation. Whether this can occur, depends on stress levels in the discontinuous wall due to horizontal and vertical seismic response which must be superposed on the gravity load stresses. These stresses are unknown since analytical results were not supplied. Also, the portion of wall which is continuous down to the foundation must be able to resist an increased overturning moment due to horizontal seismic following column failure. Failure of the walls supported by the isolated column will result in damage to equipment and piping mounted on these walls as well as on adjacent floor slabs. It is not known if wall failure will lead to gross structural failure of the control-administration building because analytical results, which would indicate the proportion of total load resisted by these walls and thus their importance to the overall system, were not supplied.

TABLE 7-1
ESTIMATED MEDIAN SHEAR WALL STRENGTHS

WALL IDENTIFICATION NO.*	ESTIMATED MEDIAN SHEAR STRENGTH (k)
1	8,800
2	7,100
3	4,400
4	3,200
5	13,000
6	12,000
7	3,700
8	2,800
9	5,100
10	3,800
11	13,000
12	14,000

See Figures 7-1 and 7-2

*- for locations of walls corresponding to these identification numbers.

TABLE 7-2

ESTIMATED MEDIAN DIAPHRAGM STRENGTHS

DIAPHRAGM IDENTIFICATION NO.*	ESTIMATED MEDIAN DIAPHRAGM SHEAR STRENGTH (k)
1	1,200
2	650
3	1,200
4	140
5	1,100
6	350
7	300

* See Figures 7-2 and 7-3 for locations of diaphragms corresponding to these identification numbers.

○ Wall Identification Number

FIGURE 7-1. GROUND FLOOR PLAN

[illegible]

☐ Wall Identification Number

☐ Diaphragm Identification Number

FIGURE 7-2. SECOND AND THIRD FLOOR PLANS

☐ Diaphragm Identification Number

FIGURE 7-3. ROOF PLAN

FIGURE 7-4. TYPICAL WALL-SLAB JOINT

8. DIESEL GENERATOR BUILDING

8.1 DIESEL GENERATOR BUILDING DESCRIPTION

The diesel generator building was added to the SONGS-1 plant in the late 1970's. It houses the twin diesel generator units and related equipment which are designed to supply power for reactor shutdown in case off-site power is lost. The structure is located northeast of containment.

A plan view of the northern half of the diesel generator is shown in Figure 8-1. It is symmetrical about an E-W centerline axis with each identical half containing one of the diesel generator units. The structure is founded on a single 5-foot-thick concrete base mat. The diesel generator units are anchored directly into this base mat. The central portion of the structure housing the diesel generators is enclosed by 1'-6"-thick concrete walls on all four sides. A central wall 12-inches-thick separates the two units. These walls stand a single story up to the 1'-9"-thick roof slab at Elevation 47'-3". Minor sub-floors and walls are included to house and support mechanical equipment.

The north and south wings to the central portion of the diesel generator building contain the water-air heat exchangers. A cross-section through the structure looking east is given in Figure 8-2. The wings are enclosed by solid concrete walls from Elevation 31'-6" to Elevation 61'-6". These walls are supported by concrete columns down to the base mat. Concrete louvers span in the N-S direction above the heat exchangers and are cast integrally with the supporting walls.

8.2 POTENTIAL DIESEL GENERATOR BUILDING FAILURE MODES

The primary lateral-load resisting system of the diesel generator building is composed of the concrete roof slab acting as a diaphragm distributing seismic inertial forces to the concrete walls. Failure of the shear walls will lead to gross failure of the structure as a whole with damage to

contained equipment. Some of these walls have large openings for personnel access and roll-up doors. These openings influence the behavior and strength at the walls under lateral load.

The open, box-shaped structures enclosing the heat exchangers consist of three concrete walls, the fourth being the exterior walls of the diesel generator building, supported by concrete columns. Under horizontal seismic excitation, the walls will behave as relatively rigid bodies translating on the relatively flexible columns. Some of the seismic load may be transmitted to the exterior diesel generator building walls by the concrete louvers. Loss-of-function of the heat exchangers and related equipment may occur due to failure of the concrete columns.

The roof slab is nearly square in plan and is expected to be more likely to fail due to vertical response rather than under diaphragm loads. The roof slab is supported around the perimeter by the exterior walls and along the E-W centerline by the central interior wall. The plan aspect ratio is such that, towards the middle, the slab must span nearly 37 feet as a one-way member for the most part. Vertical seismic excitation will be amplified as the slab vibrates out-of-plane. Flexural failure may occur if vertical response is substantial.

The diesel generator building is a relatively new structure designed to meet more current codes and criteria than much of the rest of the SONGS-1 plant. It is anticipated that it will have capacities greater than those of the other structures. Some of these failure modes may occur at earthquake levels that are not credible. However, in general, there was no basis to screen out more unlikely potential failure modes because no seismic response analytical results were made available where load could be compared to capacities.

8.2.1 Shear Wall Failure

Most of the seismic load for the diesel generator building is resisted by the concrete walls. The shear walls most likely to initiate failure are the exterior walls and the interior wall along the building axis of symmetry be-

cause of their large contributions to the overall structural stiffness. Most of the seismic loads are transmitted to these walls by the roof slab acting as a horizontal diaphragm. The stress distributions and strengths of the shear walls are influenced to a certain degree by the larger openings provided for personnel and equipment access. The behavior, as well as expected mode of failure, may be primarily flexural or shear depending on the loading, the relative size of the opening and the reinforcement around the opening faces. The north and south walls each have a single 8'-0" by 10'-0" opening that is small compared to the overall wall dimensions. Failure of this wall is expected to occur in shear rather than flexure due to the relatively low wall height. The west wall has a pair of 9'-0" by 11'-0" openings located side-by-side near the middle of the wall. The location and size of these openings is such that this wall is expected to behave as a pair of individual walls linked together. Failure of these walls is also expected to occur in shear. The east wall has pairs of 14'-0" by 17'-0" and 9'-0" by 11'-0" openings. These openings separate the wall into three distinct piers linked together by a single, relatively rigid section of wall above. A condition of rotational fixity is expected to be developed at the top of the openings for each of the piers with double curvature occurring under lateral translation at the roof. Flexural failure is expected for each of these piers, with plastic hinges forming at the top and bottom. The central wall is solid and should fail due to shear.

Estimated median shear wall strengths for the walls identified above are listed in Table 8-1. Median shear strengths were calculated following the approach outlined in Section 3.1.5. Median flexural strengths of the east wall piers were determined by applying the basic principles of Reference 16, with modifications accounting for the difference between median and design strength, possible strength degradation under cyclic loading, and additional capacity of transverse intersecting walls behaving as flanges. Pier flexural strengths are expressed as shear forces corresponding to the piers hinging top and bottom when bent in double curvature. Median capacities are based on in-plane seismic and estimated dead load forces only and do not reflect any influence due to vertical or out-of-plane seismic response since analytical results giving simultaneous load magnitudes were not supplied. Shear wall failure is expected to lead to gross structural failure of the diesel generator building.

8.2.2 Heat Exchanger Column Failure

The heat exchanger units are enclosed by walls protruding as wings from the north and south walls of the diesel generator building. These walls are discontinuous and are supported from the base mat by a series of reinforced concrete columns. Under N-S horizontal seismic response, most of the inertial load associated with the heat exchangers and their support structure is expected to be carried back to the roof of the diesel generator building through the east and west heat exchanger walls. Under E-W seismic excitation, some of the inertial load may be transmitted to the north and south diesel generator building walls by the concrete louvers. However, some of this E-W load may be resisted by the concrete columns along Lines 5.1 and 10.1 in flexure. For E-W horizontal seismic response, the wall may act as a rigid body translating and rotating in-plane on the relatively flexible columns. The columns should be deformed in double curvature and loaded by moments and shears as well as some axial forces due to wall rotation. These columns are typically reinforced by large numbers of #18 bars vertically with several #5 stirrups spaced at six inches included to restrain the vertical bars. Median flexural strengths of the columns were estimated by modifying the basic design principles of Reference 16 to account for differences between median and design strength and strength degradation under cyclic load. A comparison of concrete shear strength data against the design provisions of Reference 16 is contained in Reference 38. This data was used to predict median shear strength of the columns, with adjustment made for possible strength degradation under cyclic loading. Calculations indicate that shear rather than flexural failure is more likely. Median shear strengths were estimated to be approximately 870 k for the corner columns and 1100 k for the other columns on Lines 5.1 and 10.1. Column failure is expected to lead to damage to the heat exchangers and related equipment attached to the support structure.

8.2.3 Roof Slab Failure

The roof of the diesel generator building is a 21-inch-thick concrete slab typically reinforced by #11 bars spaced at 12 inches in each direction of the top and bottom faces. Vertical support for the roof slab is provided by the exterior and central interior wall with continuity available

at the intersections. Small concrete enclosure structures rise above the roof slab with additional slab reinforcement included beneath them.

Under vertical seismic excitation, response of the roof slab will be amplified as it vibrates out-of-plane. In plan, the slab resembles an edge-supported plate with a two-to-one aspect ratio. For this aspect ratio, both dynamic response and out-of-plane strength can be more simply estimated by reducing the central portion of the slab to a one-dimensional beam spanning in the N-S direction between the exterior and interior walls. By extending the basic design principles of Reference 16 to account for the difference between median and design strength and possible strength degradation under cyclic loading, a median flexural strength of approximately 110 k-ft/ft of slab was estimated for the typical slab conditions. Ductile behavior of the slab can be developed, with the nature of the boundary conditions such that hinges can form at the interior and exterior wall supports and at midspan. This capacity does not include the influence of horizontal seismic response since analytical results indicating the magnitudes of loads due to different earthquake components occurring simultaneously were not supplied. Failure of the roof slab can be expected to result in damage to suspended piping and equipment mounted on the slab below.

TABLE 8-1

ESTIMATED MEDIAN SHEAR WALL STRENGTHS

Wall	Estimated Median Shear Strength (k)
North Wall	18,000
South Wall	18,000
East Wall	3,200*
West Wall	12,000
Central Interior Wall	13,000

* This is the sum of the individual pier strengths.
Failure of these piers is in flexure.

FIGURE 8-2. CROSS-SECTION LOOKING EAST

9. TURBINE-GENERATOR PEDESTAL

9.1 TURBINE-GENERATOR PEDESTAL DESCRIPTION

The turbine-generator pedestal is the massive reinforced concrete structure supporting the SONGS-1 turbine-generator unit above grade. The axis of the turbine-generator is aligned in the N-S direction and the structure is symmetrical about this axis. The pedestal is surrounded on all four sides by the east and west heater platforms and the north and south turbine extensions and is structurally isolated from these other structures by expansion joints 1.5-inches-thick.

The turbine-generator pedestal is founded on a 5'-0"-thick continuous base mat. Top of the base mat is at Elevation 8'-6". The base mat is reinforced in both directions, top and bottom, with vertical stirrups included at the east and west faces of the piers and columns. The top of the turbine-pedestal deck is at Elevation 42'-0". Large openings are provided in the deck for the steam inlet, turbine exhaust, and the generator, with the turbine-generator bolted to the deck around these openings by anchors embedded in the concrete. The deck is typically 7'-0"-thick with haunches at the supports. Reinforcement is provided mainly in the longitudinal direction, depending on the slab width, with stirrups provided to resist shear stresses. The turbine gantry crane travels in the longitudinal direction on crane rails at the east and west sides of the deck. The turbine pedestal deck is supported by piers oriented in the transverse direction and columns. The columns are 8'-0" by 8'-2" in plan and are located at the corners of the turbine-generator pedestal. The piers are typically 30'-0" long and are of different thickness varying from 4'-6" to 7'-0". The piers constitute the interior supports of the turbine-generator pedestal. These vertical supports combine with the deck to form a frame for resisting longitudinal seismic load. In the transverse direction, the piers behave as isolated shear walls. A typical pier is shown in Figure 9-1. The columns also combine with the deck to form frames for resisting transverse seismic load. A typical transverse frame is shown in Figure 9-2.

9.2 POTENTIAL TURBINE-GENERATOR PEDESTAL FAILURE MODES

The primary system for resistance to seismic inertial loads associated with the turbine-generator, related equipment, and the pedestal structure itself, is composed of the vertical piers and columns and the horizontal turbine deck. In the longitudinal direction, these components combine to form a moment-resisting frame capable of load redistribution through hinging. Failure in the longitudinal direction is defined by the overall frame capacity. Transverse seismic loads are resisted by a combination of piers and frames. Failure in the transverse direction is expected to be controlled by failure of one of these individual elements.

Another potential source of turbine-generator pedestal failure is failure of the foundation base mat in the transverse direction of the face of one of the interior piers. Overturning moment at the base of each pier is transmitted to the soil by the base mat in the immediate vicinity of the pier. Special reinforcement was included in the base mat beyond the ends of the piers. As soil bearing pressure acting upward on the base mat is increased, the base mat may fail due to the out-of-plane forces generated at the face of the pier. Another failure mode related to pier overturning is failure of the soil foundation material.

Possible collapse of the turbine gantry crane which travels on the turbine-generator pedestal deck and north turbine extension should be considered a potential failure mode. Earthquake input is amplified by response of the supporting structure as well as that of the crane itself. It is not known if restraints preventing the crane wheels from lifting up off the crane rails were included in the wheel assemblies. Without these restraints, it is possible for the crane to topple in the short direction and impact exposed structures such as the condensate storage tank or equipment related to the auxiliary feedwater system.

9.2.1 Longitudinal Pedestal Failure

In the longitudinal direction, the axial stiffness of the pedestal deck is much greater than the lateral stiffnesses of the vertical elements. As a result, the piers and columns will be constrained to displace uniformly at the deck through continuity of the element connections. The lateral load-resisting system for longitudinal seismic response is a moment frame composed of the horizontal deck acting as a beam supported by the vertical piers and columns. In this direction, the piers behave as one-way slabs loaded out-of-plane. For elastic behavior, the seismic inertial loads due to the turbine-generator unit and the structure itself are collected by the deck and distributed to the vertical elements. Distribution of moments and shears through the longitudinal frame is dependent on the individual element stiffnesses. Yielding of individual elements will be followed by load redistribution, provided that post-yield behavior of the elements is ductile. The capacity of the frame as a whole, is dependent on the ability or inability of this load redistribution to occur.

Identification of the element that initiates longitudinal pedestal failure is dependent on distribution of the longitudinal load between the individual elements. However, yielding is most likely to occur in one of the piers which form the interior supports of the frame. Out-of-plane strength of these piers is believed to be limited by flexure. Median flexural strengths at the bottom of the piers were estimated to be approximately 25,000 k-ft, 34,000 k-ft, and 36,700 k-ft for the piers on Lines F, H, and K, respectively. These values were determined by extending the basic flexural design principles of Reference 16 as noted in Section 9.2.2.

Hinging at the piers is expected to be followed by failure of the beams or columns of the exterior bays from Lines E to F and Lines K to M. Failures of the beam and column in the bay from Lines E to F are expected to occur in shear. Median shear strengths of approximately 1600 k and 2500 k were determined for the column and beam, respectively. Identification of the element that fails is in part dependent on the relative stiffness of these members. For the exterior bay from Lines K to M, shear failure of the column is more likely to occur. A median shear strength of approximately 1600 k was

estimated for this member. Median shear strengths for these elements were determined by extending the design principles of Reference 16 as noted in Section 9.2.2.

The capacities listed above do not reflect the weight of the turbine-generator unit that would have been known had details of the analytical model been available. Also, they do not include the influence on strength of simultaneous vertical and transverse seismic response since analytical results were not supplied.

9.2.2 Transverse Pedestal Strength

In the transverse direction, the lateral load-resisting system consists of a series of piers and columns linked together by the pedestal deck. Large openings occur in the deck between each vertical element to the extent that the deck is reduced to a pair of beams spanning longitudinally at the east and west sides of the pedestal. Any load transfer between the piers and columns must be accomplished by these beams in flexure. Because the stiffness of these beams is expected to be on the same order or less than the stiffnesses of the vertical elements, load redistribution following initiation of yielding in any of the piers or columns should be minor. Failure of any of the vertical elements can be taken to be failure of the pedestal as a whole with subsequent damage to the turbine-generator.

The columns at the ends of the pedestal combine with the slab of the deck to form single bay moment-resisting frames. Reinforcement of the columns at the north end of the pedestal consists of a total of 30 #18 bars distributed around the column faces with #5 ties spaced vertically at 24 inches. The pedestal deck slab above these columns, which serves as a beam of the moment frame, is approximately 15'-3" wide by 8'-0" deep and is reinforced by 35 #18 bars and 6 #11 bars distributed around the beam faces with #6 ties spaced at 12 inches. The columns at the south end of the pedestal are reinforced by 28 #18 bars with #5 ties spaced at 24 inches. The concrete width of the beam above these columns varies. However, the section specially reinforced to resist the beam forces is

8'-0" wide by 7'-0" deep. This section is reinforced by 14 - #18 bars top and bottom and 3 #18 bars at each face with #6 ties spaced at eight inches near the supports and 12 inches at midspan. Haunches were formed at the inside corner of the beam-column connections to reduce stress concentrations. The basic principles and equations presented in Reference 16 for the design of reinforced concrete members loaded axially and in flexure, with design conservatisms removed, form the basis for determination of median strengths of the moment frames. Accounting for the difference between median and design strengths and possible strength degradation under dynamic loading, calculations indicate that shear in the columns may initiate failure of the moment frames at the north and south ends of the pedestal. Median strengths, expressed as total shear resisted by each frame, are estimated to be approximately 3300 k and 3200 k for the north and south frames, respectively. These capacities assume that forces due to dead load are small compared to the element strengths. This assumption was necessary because details of the dynamic turbine-generator pedestal model, which would indicate the mass of the turbine-generator unit, were not available. Also, the influence of forces due to longitudinal and vertical response was not considered because the magnitudes of these loads occurring simultaneously with the transverse loads were unknown.

The piers subjected to transverse loading behave as isolated shear walls. They are typically reinforced vertically by #18 bars spaced at 12 inches around the perimeter and with #5 ties spaced at 24 inches provided in both horizontal directions. Failure of these piers can occur in shear or flexure. The median shear strength is predicted by the equations developed for low-rise shear walls (Section 3.1.5) with the wall height taken as the distance from the base mat to the deck. The median flexural strength can be developed from the basic design principles presented in Reference 16 with design conservatisms removed and consideration given to other factors noted in the discussion on moment frame strength above. Due to the lack of specially reinforced vertical boundary elements at the ends of these piers, failure is expected to occur in flexure. Median strengths, expressed in terms of in-plane shear loads

resulting in flexural failure, are estimated to be 5200 k, 5700 k, and 6600 k for the three piers from north to south. These capacities do not include the dead load of the turbine-generator unit for the reasons noted above. Also, the influence of vertical and longitudinal seismic response was not included.

9.2.3 Base Mat Failure

Overturning moments from the columns and piers must be transmitted to the underlying soil by the base mat. Significant compressive soil bearing pressures may be developed due to transverse seismic response. This may be especially true near the piers due to their substantial in-plane rotational stiffness. Additional base mat reinforcement was provided in the transverse direction below the piers. This reinforcement typically consisted of #18 horizontal bars and #6 ties.

Base mat failure at the faces of the piers may occur due to transverse seismic response if soil bearing capacity can develop the full strength of the base mat. Median base mat shear and flexural strengths were determined at the ends of the piers following the procedures described in Section 9.2.2. Median shear strengths of approximately 1000 k, 1800 k, and 1500 k were estimated for the piers on Lines F, H, and K, respectively. Median flexural strengths were estimated to be approximately 7600 k-ft, 11,000 k-ft, and 9,400 k-ft for these same piers. The widths of base mat for which these capacities are effective correspond to the bands of special reinforcement below each pier. Base mat failure may result in excessive pedestal deformations damaging equipment and possibly causing impact with adjacent structures.

Depending on the dead load of the turbine pedestal and the level of transverse seismic response, uplift of the heel of the base mat may occur at the piers. Soil failure similar to that described for the sphere enclosure building in Section 6.2.3 may result, causing excessive lateral deflections. This failure mode should be considered in fragility calculations for the turbine-generator pedestal. However, no capacity was calculated in this evaluation since the soil bearing capacity and weight of the turbine-generator unit were not known.

FIGURE 9-1. TYPICAL PIER

FIGURE 9-2. TYPICAL TRANSVERSE FRAME

10. NORTH TURBINE EXTENSION

10.1 NORTH TURBINE EXTENSION DESCRIPTION

The north turbine extension shelters and supports equipment that participates in turbine-generator operation. It is located south and adjacent to containment and just north of the turbine pedestal. This structure was included in the original SONGS-1 construction. Modifications are currently being designed to meet new seismic criteria.

The north turbine extension consists of structural steel framing and a prestressed concrete deck. A plan view of the foundation is shown in Figure 10-1. Some of the columns are supported by individual footings, some by strip footings, and some on base mats of other structures. Ground floor equipment rests on a slab-on-grade. The north turbine extension is enclosed at the north boundary by a reinforced concrete block wall. This wall cantilevers off a concrete retaining wall and is independent of the rest of the structure. The auxiliary feedwater line is connected to this wall.

The original structural design called for moment-resisting steel frames to carry both vertical and lateral loads. Columns resist moments in strong-axis bending only. Intermediate beams spanning between the moment frames support the deck. Miscellaneous steel structures located intermediately between the slab-on-grade and the deck are supported off the main frames. The turbine deck crane travels on crane rails attached to girders of the moment frames. The crane rails allow continuous travel between the north turbine extension and the turbine pedestal with the joint between the two structures allowing relative longitudinal rail displacements. Structural modifications currently being designed include the addition of vertical braces between columns of the moment frames and upgrading of some of the moment connections. An example of the proposed upgrading is shown in Figure 10-2. Connection details for the bracing were not provided.

A plan view of the 8.5-inch-thick concrete deck is shown in Figure 10-3. The deck is separated from the surrounding structures by 1.5-inch-wide expansion joints. The slab is prestressed by main cables in the N-S direction and secondary cables in the E-W direction. Each cable consists of ten 1/4-inch-diameter wires drawn from A421-59T Type BA steel. The main cables were stressed from the side of the deck adjacent to the containment sphere. Non-prestressed areas of the deck are conventionally reinforced. Slab inserts embedded in the prestressed concrete and welded to the top flanges of the steel beams and girders are located as shown in Figure 10-4. Three different inserts, Types E, F, and G (shown in Figure 10-5), were considered in this evaluation. These inserts were not welded until a year after pouring of the deck concrete so that secondary stresses due to concrete creep and shrinkage would not be induced in the attached steel members. Non-prestressed portions of the deck are anchored to the steel framing by hooked No. 4 bars welded to the top flanges.

10.2 POTENTIAL NORTH TURBINE EXTENSION FAILURE MODES

The primary lateral load-resisting system for the north turbine extension is composed of a combination of steel braced and moment frames in both horizontal directions. Under horizontal seismic excitation, slab inertial loads are transmitted to these frames by the slab insert details, with most of the load being resisted by the braced frames due to their greater lateral stiffnesses. The identification of potential failure modes for the north turbine extension and determination of strength capacities of elements involved in these failure modes are based on the original construction drawings and preliminary design drawings of proposed structural modifications. None of the modification drawings made available indicate any remedial work intended to increase the ability of the slab to transfer its inertial load into the steel framing or to strengthen column base connections after diagonal braces are added. Also, brace connection details are not shown on the modification drawings. Failure of the slab inserts is expected to initiate failure of the north turbine extension.

The concrete block wall enclosure at the north side of the north turbine extension is structurally independent of the steel framing and must resist its own loads. Under horizontal seismic excitation in the N-S direction, this wall will vibrate out-of-plane. Out-of-plane flexural failure of the wall with subsequent damage to the auxiliary feedwater line itself is another potential failure mode.

10.2.1 Slab Insert Failure

Slab insert Types F and G, shown in Figure 10-5, were supplied to connect the north turbine extension prestressed slab to the steel framing below at the locations shown in Figure 10-4. These inserts provide a positive, mechanical connection for transferring horizontal shears from the slab into the steel framing. As shown in Figure 10-5, these inserts typically consist of rectangular bars acting as shear lugs and U-shaped anchors welded to a rectangular plate. This assembly is embedded in the bottom of the slab and welded to the top flange of the steel beam or girder below. As noted on p. 203 of Reference 33, shear lugs are relatively ineffective in resisting shear and that ultimate shear strength of embedded plates is primarily dependent on the embedded anchors. Shear strength of the slab inserts is taken as the shear strength of the U-shaped anchors that resist horizontal load in a manner similar to shear-friction. Using the results of research conducted to determine the strength of concrete inserts (References 34 to 37), the individual median shear strengths of the Types E, F, and G inserts were estimated to be approximately 41 k, 41 k, and 21 k, respectively.

Failure of the north turbine extension due to horizontal seismic response in both the N-S and E-W directions is expected to be initiated by slab insert shear failure. In the initial elastic stages of response, most of the seismic load will be resisted by the vertical braced steel frames. Once the slab inserts welded to the girders of these braced frames fail, load will be resisted primarily by the moment frames with a corresponding frequency shift due to the change in lateral stiffness. Once the capacities of the inserts welded to the girders of the moment

frames are exceeded, the only load-paths available from the slab to the vertical frames are static friction between the slab and steel framing and the slab inserts welded to the beams oriented transverse to the direction of horizontal load. Static friction cannot be relied upon due to its relatively low magnitude and the presence of Visqueen included at the slab-steel interface to permit creep and shrinkage of the prestressed concrete. The remaining slab inserts welded to the transverse beams apply horizontal forces to the top flanges of these members which are carried back to the vertical frames in the direction of load. These transverse beams are loaded in weak-axis bending and torsion which contributes to a reduced lateral stiffness and accompanying frequency shift. Presumably, these members could be stressed to failure. However, it is believed that slab lateral displacements will be large for this structural condition due to the small stiffness and the expansion joints separating the north turbine extension from the turbine-generator pedestal and the fuel storage building will close. Impact of the slab with these other structures is expected to lead to concrete crushing and spalling at the regions of impact. Consequences of slab impact include possible damage to equipment located below and damage to prestress tendon anchorages with subsequent slab failure.

Estimated median frame strengths, expressed as horizontal frame forces, are listed in Table 10-1. All of these strengths correspond to failure of the attached inserts.

10.2.2 Concrete Block Wall Failure

The freestanding block wall is located at the north side of the structure along Line A. This wall cantilevers 10'-0" up from a concrete retaining wall and is independent from the rest of the north turbine extension. Vertical control joints occur at every four feet along the wall. Reinforcement consists of #5 vertical bars spaced at 24 inches and #5 horizontal bars spaced at 48 inches.

For earthquake excitation in the N-S direction, seismic input will be amplified as the wall responds out-of-plane as a cantilever. Flexural failure is expected to occur at the base of the wall at a median moment capacity of approximately 2200 lb-ft/ft. This moment capacity was calculated similar to the control-administration block walls described in Section 7.2.3. Although failure of the block wall does not necessarily imply collapse, damage to the auxiliary feedwater line which is attached directly to this wall may be expected.

TABLE 10-1

ESTIMATED MEDIAN FRAME STRENGTHS

Frame	Estimated Median Strength (k)
Line 6	62
Line 7	83
Line 8	62
Line A	82
Line B	21
Line D	100

FIGURE 10-1. FOOTING LOCATIONS

a) Column Line 6 Looking East

b) Column Line B Looking North

FIGURE 10-2. TYPICAL PROPOSED SEISMIC MODIFICATIONS

FIGURE 10-3. PLAN VIEW OF CONCRETE DECK

10-10

FIGURE 10-4. SLAB INSERT LOCATIONS

a) Type E

b) Type F

c) Type G

FIGURE 10-5. SLAB INSERTS

11. WEST HEATER PLATFORM

11.1 WEST HEATER PLATFORM DESCRIPTION

The west heater platform shelters and supports equipment that participates in turbine-generator operation. It is located adjacent to the west side of the turbine pedestal. The top of the deck is about 6.5 feet below that of the pedestal. Modifications similar to those planned for the north turbine extension are being designed for the west heater platform.

A plan view of the foundation is shown in Figure 11-1. Some of the columns are supported by individual footings, some by strip footings, some on the turbine pedestal base mat, and some by Anchor Block No. 1, a large mass of concrete that also forms the circulating water system intake culvert. The ground floor is located at either Elevation 8'-6" (top of the anchor block) or Elevation 14'-0". The floor at Elevation 14'-0" is slab-on-grade. The auxiliary feedwater pumps are mounted on pedestals founded directly on soil. Reinforced concrete block walls enclose the west heater platform. Each wall is supported at the base by a continuous wall footing and at the top by the steel framing. Upgrading of the block wall-steel framing connection is currently being designed.

The west heater platform structure is similar to that of the north turbine extension (see Section 10.1). Moment-resisting steel frames were designed to resist both vertical and lateral loads. Bracing such as that shown in Figure 10-2 is being designed to provide additional seismic resistance.

A plan view of the deck is shown in Figure 11-3. It is prestressed by main cables in the E-W direction and secondary cables in the N-S direction.

11.2 POTENTIAL WEST HEATER PLATFORM FAILURE MODES

Construction of the west heater platform is similar to that of the north turbine extension described in Section 10. Seismic loads due to horizontal response will be primarily resisted by the braced frames to be added as a part of the design modifications with the existing moment frames participating to a lesser extent. Identification of potential failure modes and calculation of failure mode capacities were conducted using original construction drawings and design drawings of proposed structural modifications. It is not known if work is planned to upgrade the connection between the slab and steel framing or the column base connections, and it was assumed in this evaluation that no such work will be performed. Diagonal brace connections, not shown on the modification drawings, were assumed to be able to develop the full brace strengths. The potential failure modes for the west heater platform are expected to be the same as those for the north turbine extension (see Section 10.2). Load-carrying capacity of vertical frames is expected to be limited by the ability of the slab inserts to deliver inertial forces or by the ability of the existing column base connections to resist uplift forces. Out-of-plane flexural failure of the concrete block walls around the structure perimeter is also possible.

11.2.1 Slab Insert Failure

Under horizontal seismic excitation, inertial loads generated by the west heater platform will be resisted by the vertical braced frames created by the structural modifications currently being designed and to a lesser extent, by the moment frames of the original construction. The ability of the frames to resist lateral loads is limited by capacities of the individual members and their connection details and the ability of the slab to transmit its inertia load into the frames. Slab inserts, described in Section 10.2.1 on the north turbine extension, were also used for the west heater platform.

Calculations to evaluate the overall frame load-carrying capacities were conducted similar to those for the north turbine extension (Section 10.2.1). Potential failure modes for each frame were compared to

determine which will initiate frame failure. These included brace and column yielding or buckling, yielding of the column anchor bolts due to uplift force, flexural yielding of the moment-frame members, and slab insert shear failure. It was assumed that the brace connection details will be sufficient to develop the brace strength. This study indicated that individual frame load-carrying capacities are likely to be limited by slab insert shear failure. The type and number of inserts attached to each frame as well as the total estimated frame strengths are listed in Table 11-1. The prediction of this failure mode assumes that no modifications are planned to increase the ability of the slab to transmit its inertial force into the frames. This assumption was necessary since no such work is indicated on the modification drawings received. If details are developed to strengthen the slab-frame connections, it will be necessary to reevaluate the frame capacities.

The failure scenario for the west heater platform will be similar to that for the north turbine extension (Section 10.2.1). Slab insert failure should first occur at the braced frames and will be followed by insert failure at the moment frames. Load transfer must then be accomplished by the inserts to members normal to the direction of load. Static friction cannot be relied upon due to the presence of Visqueen at the interfaces between the bottom of the slab and top flanges of the beams. Members transverse to the direction of load and loaded at their top flanges by the remaining slab inserts will be deformed in torsion and weak-axis bending. Due to the inherent flexibility of this system, the west heater platform slab may impact the adjacent structures. Impact is expected to lead to cracking and spalling of the concrete at the locations of impact with possible damage to equipment below and to prestress tendon anchorages.

11.2.2 Concrete Block Wall Failure

Concrete block walls were constructed around the west and south boundaries of the west heater platform to serve as a non-load bearing weather enclosure. The walls are founded on their own strip footings and

typically span about 20'-8" high. These walls are reinforced by #5 vertical bars spaced at 32 inches and #5 horizontal bars spaced at 48 inches. Vertical control joints were included at every four feet for the full wall height. At the top of the walls, masonry anchors welded to the steel framing of the west heater platform and embedded into the walls were spaced about four feet apart. These anchors provide the only tie to the west heater platform superstructure.

The inclusion of the control joints forces the block walls to behave as one-way members under out-of-plane loading. For horizontal seismic excitation, response of the walls is amplified as they vibrate out-of-plane. The flexural strength of these walls was determined similar to the block walls of the control-administration building as described in Section 7.2.3. A median out-of-plane flexural strength of approximately 1300 lb-ft/ft was estimated for the block walls enclosing the west heater platform. Failure of the block walls does not necessarily imply collapse; however, damage to equipment and piping mounted on the wall and to the slab within the immediate vicinity of the wall may be sustained.

TABLE 11-1

ESTIMATED MEDIAN FRAME STRENGTHS

Frame	Type and Number of Inserts	Estimated Median Frame Strength (k)
Line 9	3 - Type E	120
Line 12	4 - Type G	82
Line 13	3 - Type E	120
Line C	2 - Type G	41
Line E	2 - Type G	41
Line F	2 - Type G	41
Line G	3 - Type G	62
Line H	2 - Type G	41
Line J	2 - Type G	41
Line K	2 - Type G	41

FIGURE 11-1. FOOTING LOCATIONS

a) Column Line 13 Looking West

b) Column Line H Looking North

FIGURE 11-2. TYPICAL PROPOSED SEISMIC MODIFICATIONS

FIGURE 11-3. PLAN VIEW OF CONCRETE DECK

12. FUEL STORAGE BUILDING

12.1 FUEL STORAGE BUILDING DESCRIPTION

The fuel storage building provides storage for both the new and spent fuel rods used for SONGS-1. It is located southwest of containment. The spent fuel pit composes the northern half of the structure. Switch-gear equipment and new fuel rods are contained in the southern half. The auxiliary feedwater line is attached to walls at the north and west elevations of the fuel storage building.

A plan view of the fuel storage building at Elevation 17'-0" is shown in Figure 12-1 and a transverse cross-section is shown in Figure 12-2. The spent fuel pit is an open-topped rectangular concrete box 30'-0" by 73'-4" in plan and rising to Elevation 42'-0". The spent fuel storage racks are mounted on the base mat and braced against the walls. The pit is lined with stainless steel plate and filled with water. The fuel transfer tube from the reactor building enters at the northeast corner. Thickness of the base mat varies from 4'-9" to 5'-9" with the slab bottom located at Elevation 2'-9". Wall thickness is typically 4'-0".

The southern half of the fuel storage building and the portion above the spent fuel pit consists of a structural steel framing enclosed by non-load bearing reinforced concrete block walls. The ground floor, located at Elevation 14'-2", which supports switchgear equipment is a six-inch slab-on-grade. The block walls rest on continuous wall footings. The steel columns are founded on square footings that also support the adjacent block wall and slab-on-grade. A similar foundation is used for the west heater platform columns along the south fuel storage building wall. The isolated column of the southeast corner shares a footing with a west heater platform column.

Floors above grade are located at Elevations 31'-0" and 42'-0". At Elevation 31'-0", 1.25 inch galvanized grating is carried by steel beams spanning between steel columns and the south spent fuel pit wall. The new fuel racks are anchored to the nine-inch-thick concrete slab at Elevation 42'-0". The slab rests on steel floor members spanning between steel columns and the south spent fuel pit wall.

Concrete block walls are typically reinforced by #7 vertical bars spaced at 32 inches and #5 horizontal bars spaced at 48 inches. Block walls enclose the southern half of the structure from grade to the roof (about Elevation 65'-0"). They are laterally supported as shown in Figure 12-3. The east and west walls are doweled into the spent fuel pit walls. Block walls enclose the spent fuel area from the tops of the pit walls to the roof.

Modifications to the structure above Elevation 42'-0" intended to meet new seismic criteria are currently being designed. Existing roof decking over the spent fuel pit is to be removed and replaced with two criss-crossed layers of decking after additional steel roof framing members have been installed. A horizontal roof truss is to be included adjacent to the north wall. Existing roof decking over the southern half of the building will be left in place with new steel beams added beneath it. Block wall surrounding the spent fuel pit will be braced by two criss-crossed layers of metal decking through-bolted to the inside face. Other block wall is to be braced by horizontal beams at midheight spanning between existing and new columns. The block wall at the east elevation from Elevation 42'-0" and up will be replaced by a 14-inch-thick concrete wall anchored into the spent fuel pit wall.

12.2 POTENTIAL FUEL STORAGE BUILDING FAILURE MODES

Much of the inertial load generated by seismic response of the fuel storage building will be associated with the walls of the fuel pit and the water it encloses. The heavy concrete walls of the fuel pit are expected to provide much of the resistance to these loads as well as to seismic loads associated with the floor slabs and concrete block walls. These walls may fail under in-plane shear or overturning. In addition, hydrodynamic pressures due to response of the water contained in the fuel pit may result in out-of-plane flexural failure of these walls.

Failure of the concrete block walls may occur as a result of in-plane or out-of-plane response. The walls enclosing the southern half of the structure are connected into the floors and roof and will receive some of their inertial loads as in-plane forces. The east and west block walls may serve only as collectors, transferring their in-plane loads to the more massive fuel pit walls. However, the south block wall must carry its in-plane load down to the foundation. In-plane failure of this wall was investigated. Also, the concrete block walls of the fuel storage building will vibrate out-of-plane between their lateral supports as horizontal seismic excitation is amplified up through the structure. Failure of selected block walls due to out-of-plane forces was also investigated.

The possibility of diaphragm failure was checked at the roof and floor at Elevation 42'-0". Modifications to the roof of the fuel storage building are currently being designed. Roof diaphragm capacities were based on preliminary drawing supplied with necessary assumptions made when information was insufficient. Diaphragm failure of the roof may occur in the new or old metal decking or in the connections between the block walls and the steel beams at the roof perimeter. The connection detail between the floor slab at Elevation 42'-0" and the concrete block wall on Line B.7 (south wall) is similar to that employed throughout the control-administration building (see Section 7.2.2). Failure of the diaphragm at this connection was also evaluated.

12.2.1 Shear Wall Failure

Due to their greater stiffnesses compared to the other elements of the fuel storage building, the concrete walls forming the fuel pit are expected to attract much of the seismic load of this structure. Because of this, their failure is expected to lead to gross structural failure of the fuel storage building. Failure of these walls may occur due to in-plane shear or overturning. Shear failure, rather than flexural failure, is expected for the north and south walls since their low heights relative to their lengths reduce the significance of overturning. This failure is anticipated to occur in the height of wall above Elevation 12'-0". Using the equations described in Section 3.1.5, median shear strengths of approximately 30,000 k and 32,000 k were estimated for the north and south walls, respectively. Failure of the east and west walls may occur in shear or flexure. Shear failure is expected to occur in the height of wall above Elevation 3'-0". Median shear strengths of approximately 7000 k were estimated for both the east and west walls. Flexural failure due to overturning moment is expected to occur at Elevation 12'-0". Median flexural strengths for these walls were calculated using the basic principles of Reference 16 adjusted to eliminate design conservatisms and accounting for the behavior of the intersecting north and south walls as flanges providing additional resistance. A median flexural strength of approximately 110,000 k-ft was estimated for both the east and west walls.

The concrete walls of the fuel pit receive hydrostatic pressure from the contained water in at-rest conditions. Under seismic excitation, hydrodynamic pressures will be developed. These fluid pressures are resisted by out-of-plane forces in the fuel pit walls. Failure is expected to occur in the north wall since it has a longer horizontal span than the east and west walls and is not braced laterally by floors as is the south wall. Due to the stiffness of the fuel storage building, the rigid tank equations presented in Reference 42 are expected to adequately predict the magnitude and distribution of the hydrodynamic pressures. Wall resistances to out-of-plane forces were developed by extending the

basic principles of Reference 16 to account for the difference between median and design strength and possible strength degradation under cyclic loading. Out-of-plane yielding in flexure is expected to initiate at Elevation 22'-0" in the central portion of the north wall under hydrodynamic pressures corresponding to a peak ground acceleration of approximately 1.0g. The aspect ratio of the wall above this elevation is such that stress redistribution and development of yield lines will not significantly increase this yield capacity.

12.2.2 Block Wall Failure

As previously noted, the block walls enclosing the southern portion of the fuel storage building are structurally connected to the floor and roof elements. Typical connection details at the floors of the original construction are shown in Figure 12-3. These details, as well as those at the roof, are capable of transmitting in-plane shears from the floors to the block walls. The east and west concrete block walls will probably behave only as collectors, transferring their loads into the stiffer concrete fuel pit walls. The south wall must carry its in-plane loads down to the foundation. Similar to the concrete block walls of the control-administration building (Section 7.2.3), the median in-plane shear strength of this wall, based on a review of available test data and recommendations, was estimated to be approximately 500 k. This capacity does not reflect the possible reduction in strength due to out-of-plane load since the magnitude to this load occurring simultaneously with the in-plane shear was not supplied.

Under horizontal seismic excitation, the concrete block walls will vibrate out-of-plane in their natural modes as seismic input is amplified up through the structure. Potential locations of out-of-plane flexural failure are expected to be between the foundation and Elevation 31'-0" and between Elevation 42'-0" and the roof. Both of these walls are typically reinforced by #7 bars vertically spaced at 32 inches. Following calculations described in Section 7.2.3 for the control-administration building block walls, median out-of-plane flexural

strengths of 3700 lb-ft/ft and 3200 lb-ft/ft were estimated for the walls at the bottom and top stories, respectively. Due to continuity of reinforcement to the foundation and to story above, this flexural strength can be developed at the supports of the wall below Elevation 31'-0". Failure will occur only after hinging has developed at the supports and midheight. As noted in Section 7.2.3, estimated out-of-plane flexural capacities do not include the possible influence of vertical and in-plane seismic loads since load magnitudes occurring simultaneously were not supplied.

Additional lateral bracing is currently being developed for the block walls between Elevation 42'-0" and the roof. Strength calculations were based on the preliminary design drawings supplied. Two different types of bracing are indicated. Around the fuel pit, the block wall will be lined at the interior face by two layers of steel decking connected to the steel roof beams and through-bolted to the wall. At other locations, steem beams spanning between steel columns are to be added at mid-wall height and through-bolted. Failure of the wall at this latter detail is expected to be more likely. Support conditions for the upper half of this wall can be taken as fixed at the beam and simply supported at the roof. For failure to occur, hinging must develop at the beam and at an intermediate location between the support. As noted in Section 7.2.3, block wall failure is expected to lead to damage to attached equipment and piping and to floor-mounted equipment within the immediate vicinity of the walls.

12.2.3 Diaphragm Failure

Modifications to the fuel storage building are currently being designed to strengthen the roof for seismic loading, particularly the portion over the fuel pit. Capacity evaluations described in this section were based on information contained on preliminary design drawings. In general, these modifications include the addition of steel roof beams and columns spanning from Elevation 42'-0" to the roof, the addition of a horizontal truss along the north block wall (Line aA), and

the replacement of existing decking over the fuel pit with new decking. Existing perimeter beams and their connections to the adjacent block walls are to remain intact. Existing 1.5-inch-deep decking over the southern half of the fuel storage building is to remain in place with a few steel roof beams added from below.

Under horizontal seismic response, roof inertial loads will be resisted for the most part by the block walls above the stiff concrete walls of the fuel pit. The in-plane forces must be transmitted to these walls by the metal deck diaphragm-roof truss system. The horizontal roof truss acts as a collector for distributing in-plane diaphragm forces to the perimeter beams adjacent to north, east, and west exterior walls which, in turn, transmit the loads to these walls. Elsewhere, diaphragm forces are transmitted by the metal roof decking.

The modifications currently being designed are expected to have substantial capacity since they should meet current codes and criteria. A comparison of anticipated diaphragm loads to capacities indicates that diaphragm failure is more likely to occur at walls not receiving load through the new roof truss. Also, the connection detail between the collector beams and the block walls appear to have greater strength than the diaphragm itself. Diaphragm strength of metal deck is dependent on the type of deck, spans between supports, and the type and degree of attachment to the supports as well as between the deck units themselves. Only the spans of the existing deck are known for the new deck. To evaluate diaphragm capacities, unknown deck were estimated with an attempt made to keep these properties consistent with the overall structure design level, both new and old. Median diaphragm shear strengths were estimated to be approximately 58 k adjacent to the south wall (Line B.7), and 67 k and 270 k in the existing and new deck, respectively, adjacent to the central E-W block wall (Line A.9). Once diaphragm action is lost, the block walls transverse to the direction of load must resist roof inertial force in out-of-plane flexure. Failure of the block walls with subsequent damage to equipment and piping mounted to the walls and the floor in the immediate vicinity of the walls may result.

FIGURE 12-1. PLAN VIEW AT ELEV. 17'-0"

FIGURE 12-2. TRANSVERSE CROSS-SECTION

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