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ASSESSMENT OF SEISMIC MARGIN CALCULATION METHODS

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ABSTRACT

Seismic margin review of nuclear power plants requires that the High Confidence of Low Probability of Failure (HCLPF) capacity be calculated for certain components. The candidate methods for calculating the HCLPF capacity as recommended by the Expert Panel on Quantification of Seismic Margins are the Conservative Deterministic Failure Margin (CDFM) method and the Fragility Analysis (FA) method. The present study evaluated these two methods using some representative components in order to provide further guidance in conducting seismic margin reviews. It is concluded that either of the two methods could be used for calculating HCLPF capacities.

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Seismic margin review, of nuclear power plants require that the High Confidence of Low Probability of Failure (HCLPF) capacity be calculated for certain components. Two candidate methods for calculating the HCLPF capacity have been recommended by the Expert Panel on Quantification of Seismic Margins. The present study evaluated these two methods: the Conservative Deterministic Failure Margin method and the Fragility Analysis method using some representative components in order to provide further guidance in conducting the seismic margin reviews. The following persons have participated in this project.

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EXECUTIVE SUMMARY

Seismic margin reviews of nuclear power plants require that the High Confidence of Low Probability of Failure (HCLPF) capacity be calculated for certain components. The candidate methods for calculating the HCLPF capacity as recommended by the Expert Panel on Quantification of Seismic Margins are the Conservative Deterministic Failure Margin (CDFM) method and the Fragility Analysis (FA) method. The HCLPF Study Group consisted of a Lawrence Livermore National Laboratory Project Manager and four independent consultants who evaluated these methods by performing the HCLPF capacity calculations on a representative set of components. The components selected were: flat-bottom vertical storage tank, auxiliary contactor for motor starter in an older motor control center, starting air tank, component cooling heat exchanger, and cantilevered reinforced block wall. Two different locations in the building (at grade and at a floor high in the building) were studied for the motor control center. First, the Study Group members calculated the median and HCLPF capacities of these components using either the CDFM or FA method or both. The results of the first round of calculations were reviewed and the sources of differences in the capacities calculated by the members were identified. Afterwards, each investigator was allowed to revise his calculations to determine if closer agreement could be obtained.

The four consultants differed from each other by a median ratio of the high/low values of 1.39 to 1.55 in their final estimates of the HCLPF capacity, the median capacity, and the ratio of median/HCLPF capacity. These differences are mainly due to the differences in models, parameters, and assumptions used by the investigators.

Based on these calculations, the HCLPF Study Group has concluded that any future effort in reducing the differences in calculated HCLPF capacities should be spent in reducing the differences between the assumptions and judgment of different analysts rather than in trying to reconcile the smaller differences between the CDFM and FA methods. It is concluded that either of the two methods could be used for calculating the HCLPF capacities in seismic margin reviews.

The Study Group recommended that HCLPF and median capacity estimates be independently performed for the selected components by a number of representatives from architect-engineering firms. Six Architect-Engineering firms have been selected to make independent calculations on the same five components. This would give an indication of any increased variability in the estimates to be expected in practice in future seismic margin reviews. This effort is expected to be completed in 1989 and a report will be issued summarizing the results and conclusions.

CHAPTER 1
INTRODUCTION

1.1 Background on Seismic Margin Methodology

In recent years there has been increasing interest in assessing the capability of nuclear power plants in the United States to withstand earthquakes beyond their original design bases. This interest has developed because of the following concerns:

a. The perception of seismic hazard in the plant vicinity has changed and in most cases increased since the design of the plant, and

b. The seismic design criteria have been revised substantially.

To resolve these concerns, a seismic margin study can be performed to estimate the seismic capacity of the plant. Seismic margin study methodology has evolved over the years, beginning with the Systematic Evaluation Program (SEP). More recently, the NRC formed an expert panel to develop a seismic margin review methodology and guidelines for application (Budnitz et al., 1985, Prassinos et al., 1986). A parallel effort was initiated by the Electric Power Research Institute (EPRI 1987). A discussion of the evolution of the seismic margin review methodology is given in Section 2.1 of this report.

A seismic margin review studies the question of whether the capacity of the plant exceeds target earthquake input selected for review. It is assumed that the regulatory agency and the plant owner jointly select the review earthquake level. The objectives are then to show that the plant can withstand the effects of this review earthquake level with high confidence and to identify seismic vulnerabilities. This is accomplished using the results and insights obtained from past seismic Probabilistic Risk Assessments (PRA), the data on actual performance of structures and equipment in recorded earthquakes, and analytical qualification and test data.

Although a seismic PRA would provide answers regarding the seismic capacities of components, systems, and the plant, the large uncertainties in the seismic hazard curves make decisions regarding seismic adequacy difficult. The large number of systems and components to be considered in a PRA limit the attention paid to the more critical components and systems in the plant. The seismic margin review, on the other hand, focuses on the few components and systems in the plant whose failure would lead to severe core damage. The output of a seismic margin review is an estimate of the plant seismic capacity, whereas the seismic PRA provides estimates of seismic risks of core damage and adverse public health effects.

1.2 Definition of Seismic Margin

The concept of a high confidence of low probability of failure (HCLPF) capacity is used in the seismic margin reviews to quantify the seismic margin of a nuclear power plant. This is a conservative capacity, and in simple terms it corresponds to the earthquake level at which, with high confidence, it is extremely unlikely that failure of the component, system, or plant will occur. The use of the term component refers to mechanical and electrical equipment, piping, structural elements, etc. When the component capacity is described in terms of probability distributions, the HCLPF capacity is equal to approximately a 95 percent confidence (subjective probability) of not exceeding approximately 5 percent probability of failure. The concept of HCLPF capacities of components is used in the seismic margin studies in (1) screening out certain components as having capacities generically higher than the review earthquake level and (2) evaluating the capacities of certain critical components in order to assess the seismic capacity of the plant.

Estimating the HCLPF of a component requires estimating the response of the component, conditional on the occurrence of the seismic margin earthquake, and estimating the capacity of the component. Two candidate methods for calculating the HCLPF capacities for components have been recommended: the Conservative Deterministic Failure Margin (CDFM) method and the Fragility Analysis (FA) method (Kennedy, 1984; Prassinos et al., 1986).

The fragility analysis method was used in the Maine Yankee seismic margin study (Ravindra et al., 1987). This method requires evaluation of parameters such as the median, β_R and β_U using considerable judgment.

The CDFM method prescribes the parameter values and procedures to be used in calculating the HCLPF capacities and requires less subjective judgment than the FA method, although, some subjective decisions were made in formulating the procedures used in the CDFM method.

1.3 Objectives of the Present Study

The original objectives of this study were to:

- perform a comparison of the HCLPF capacities obtained by using the CDFM and FA methods to study a representative set of components;
- modify the details of the CDFM method as given in the EPRI report based on the results of calculations, and
- compare the methods and provide additional guidance for use in seismic margin reviews.

1.4 Organization of this Study

A five-member HCLPF Calculation Study Group was assembled to perform this study. Members of the Group were:

R.P. Kennedy (Chairman)	RPK/Structural Mechanics Consulting
R.C. Murray	Lawrence Livermore National Laboratory
M.K. Ravindra	EQE Engineering Inc.
J.W. Reed	Jack R. Benjamin and Associates, Inc.
J.D. Stevenson	Stevenson & Associates

The Study Group laid out the approach followed in this study and performed the capacity evaluation. EQE prepared the draft reports for review and modification by the Study Group.

The Study Group met three times. In the first meeting, the ground rules for the study were established, and a set of five components were selected; the members of the Group agreed to independently calculate the median and HCLPF capacities of these components using either the CDFM or FA method or both. In the second meeting, the results of the first round of calculations were reviewed, and the sources of differences in the capacities calculated by different investigators were identified. Afterwards, each investigator was allowed to revise his calculations to determine if closer agreement could be obtained. A draft report was prepared summarizing the results of the two sets of calculations and the conclusions of the Group. In the third meeting the draft report was reviewed by the Group, and the final report was prepared. This report represents a consensus of all Study Group members.

CHAPTER 2
METHODS OF CALCULATING HCLPF CAPACITIES

2.1 Evolution of Methods

Kennedy (1984) and Prassinos, et al., (1986) have both recommended two possible approaches for estimating the High-Confidence-Low-Probability-of-Failure (HCLPF) seismic capacity of a component. These approaches are the Conservative Deterministic Failure Margin (CDFM) and the Fragility Analysis (FA) methods. This section gives a brief historical background on both approaches.

Traditionally, the seismic capacity of nuclear power plant components has been very conservatively evaluated by using design evaluation procedures which have gradually changed over the years (generally with increasing conservatism). Current seismic criteria used in the seismic evaluation of more recent plants are contained in the Standard Review Plan (NRC, 1981). At the same time, the perceived ground motion levels to which these plants might be subjected by some future earthquake have also been increasing over the years. Changed perception of ground motion levels and changes in seismic design evaluation procedures have led to questions concerning the seismic resisting capability of components, particularly in the older nuclear power plants. The need for studying the seismic capability of the oldest U.S. nuclear power plants because of these changes was first implemented in the U.S. Nuclear Regulatory Commission (NRC) Systematic Evaluation Program (SEP).

At its inception, NRC senior management recognized that seismic evaluations in the SEP should use evaluation criteria which were less conservative than the seismic design criteria contained in the Standard Review Plan. In 1978 a Senior Seismic Review Team (SSRT)[1] was retained to provide guidance and assist NRC during seismic evaluation of older plants. A set of more rational but still quite conservative seismic evaluation criteria came out of these reviews. These criteria are documented in the plant-specific reviews of Oyster Creek (Murray, et al., 1981), Ginna (Murray, et al., 1980), Dresden #2 (Newmark, et al., 1980), Millstone #1 (Nelson, et al., 1981a), and Palisades (Nelson, et al., 1981b) conducted by the SSRT in 1979 and 1980.

In 1982, as part of the Seismic Safety Margins Research Program, Lawrence Livermore National Laboratory published a Fragilities Development Report (Bohn, et al., 1982) which included an expert opinion survey of fragilities including HCLPF and median estimate for a large number of mechanical and electrical components found in nuclear power plants. An expert opinion or Delphi procedure coupled with plant walkdowns also have been used to evaluate seismic capabilities of DOE facilities at Hanford and Savannah River sites (Becker and Stevenson, 1984).

1 Senior Seismic Review Team consisted of chairman: N.M. Newmark; members: W.J. Hall, R.P. Kennedy, R.C. Murray, and J.D. Stevenson

In 1983, an extensive Seismic Margin Review was initiated on the Midland Nuclear Power Plant. The SEP seismic evaluation criteria were expanded, slightly modified in conjunction with discussions with the NRC, and documented by Wesley et al. (1983).

During this same period (1978 through 1984), a number of plants were undergoing Seismic Probabilistic Risk Assessments (SPRAs), which necessitated the development of seismic fragility curves for components to define their failure probability estimates. The methodology used in developing these seismic fragility curves was first presented by Kennedy et al. (1980) and expanded upon and updated by Kennedy and Ravindra (1984). This methodology was heavily based upon earlier work of Ang and Newmark (1977) and Cornell and Newmark (1978). The methodology presented in Kennedy et al. (1980) and Kennedy and Ravindra (1984) has become known as the Fragility Analysis (FA) method, and it is called such in Prassinos et al. (1986).

One aspect of the FA method is that it presents for each component a suite of curves (corresponding to different confidence levels) of probabilities of failure versus ground motion levels. This complexity is necessary for use in SPRAs, but it leads to great difficulty in making decisions as to whether an adequate seismic margin exits. Such decisions are easier when only a single conservative but realistic capacity is reported for each component. In order to discuss the adequacy of seismic margins with the NRC staff and the Advisory Committee on Reactor Safeguards (ACRS), it was found useful to convert the information displayed in the seismic fragility curves into a single seismic margin descriptor. The descriptor chosen was the High-Confidence-Low-Probability-of-Failure (HCLPF) capacity, which corresponds to about 95% confidence of less than about a 5% probability of failure. Such a descriptor is conservative because there is very little chance of failure below the HCLPF capacity; and yet it is realistic because it is an attempt to describe failure. These HCLPF capacities derived from the FA method fragility curves were first used to interpret the Limerick SPRA results in a Seismic Margin context, and they were subsequently used for the same purpose with the Millstone 3 SPRA results as described in Ravindra et al. (1984). In both cases, these HCLPF capacities were very useful in defining a conservative but adequately high seismic margin capability for each plant.

Although HCLPF capacities obtained from fragility curves using the FA method proved to be a useful descriptor of seismic margin, several potential deficiencies were identified in the method:

1. The method requires an excessive number of judgments and calculations because a median capacity, a randomness variability factor, and an uncertainty variability factor must each be estimated before the HCLPF capacity can be calculated. When a SPRA is already being performed on a plant, this condition is no deficiency since median, randomness, and uncertainty estimates are required for development of fragility curves to use in the SPRA. However, if one only needs the HCLPF capacity and does not need the entire fragility curve, there should be a more direct way to compute the HCLPF capacity.

2. There are a very limited number of practitioners making seismic fragility estimates. On the other hand, a large number of qualified engineers have substantial experience in making and reviewing deterministic seismic margin evaluations by using criteria similar to that used in the SEP.

3. Because of the requirement for a significant use of judgment in the estimation of median capacities, randomness, and uncertainty factors, and because of the dependence of the HCLPF capacity on all three, there was a lack of consistency in the estimated HCLPF capacities for different plants or different components in the same plant even when made by the same team of people. This situation is illustrated by the variation in HCLPF capacities reported for similar components in different nuclear plants in Appendix B of Budnitz et al. (1985). Each of these estimates was made by the same team of people following the methodology given in Kennedy et al. (1980) and Kennedy and Ravindra (1984), but the estimates were made at different times from 1978 through 1984, and they are often not consistent with each other.

4. At present time there is no consensus methodology available to develop randomness and uncertainty factors in a consistent manner.

Because of the considerations described above, Kennedy (1984) recommended that the HCLPF capacity be directly computed by using deterministic approaches similar to those used in the SEP and the Midland Seismic Margin Review (Wesley et al., 1983). Realistic HCLPF capacities being calculated from fragility curves using the FA method were somewhat more liberal (Kennedy, 1984) than seismic margin capacities obtained when using the deterministic SEP and Midland Seismic Margin Methodologies (Wesley et al., 1983). Therefore, Kennedy (1984) recommended using a Conservative Deterministic Failure Margin (CDFM) methodology to directly obtain HCLPF capacities. CDFM methodology was heavily based upon the SEP and Midland Seismic Margin Methodologies, which Kennedy (1984) called a Code Margin (CM) methodology. The CDFM method was to use the same building and component response criteria as the CM method, which had been laid out and accepted by the NRC for the Midland Seismic Margin Review. However, in lieu of using code seismic capacities, the CDFM method more liberally allowed the use of approximately 84% exceedance capacities based on actual test data, where such data existed. Furthermore, the CDFM method more liberally allowed the explicit incorporation of an inelastic energy absorption capacity increase factor for ductile failure modes. Otherwise, it was identical to the conventional CM method which had already had considerable use.

Only a philosophical and sketchy outline of the CDFM method is contained in Kennedy (1984), and a considerable number of details were left open to judgment and interpretation. Subsequently, in an Electric Power Research Institute (EPRI) Seismic Margin Methodology Project (EPRI, 1987), the CDFM method was slightly modified and the details of the approach were expanded. The slight modification consisted of changing the recommendation for structure and equipment damping to be used in response evaluations. Kennedy (1984) recommended such damping be at the 84% exceedance probability; EPRI (1987) recommends that a conservative estimate of the median damping be used. However, the specific example damping values contained in the two references are essentially identical to each other, and this change is therefore primarily a clarification to avoid the introduction of potentially excessive conservative damping values. Even though EPRI (1987) expands on the details of the CDFM method, the method is still not prescriptive in many details and is open to considerable interpretation and judgment, which leads to differences in HCLPF values calculated by different analysts.

Although the CDFM method in its current form (EPRI, 1987) requires fewer computations, is more prescriptive, and requires less subjective judgment than does the FA method in its current form (Kennedy, 1980; Kennedy and Ravindra, 1984; Prassinos et al., 1986), there is debate concerning the degree of reduction in computational effort and the requirement for individual judgments that has resulted. There is also debate about how prescriptive the CDFM and FA methods should become in the future. Undoubtedly, both methods will continue to evolve, and there will be pressure to make them more prescriptive in order to reduce the variability of HCLPF capacity estimates.

2.2 Description of CDFM Method

In the CDFM method, a set of deterministic guidelines (e.g., ground response spectra, damping, material strength, and ductility) have been recommended. The HCLPF capacity of the component is determined using these guidelines. The procedure is similar to that used in the Systematic Evaluation Program, although the choice of some of the parameter values (e.g., damping) may be more liberal in the CDFM method. The method is appealing because it is very similar to the design procedures followed in the industry, except that some of the parameter values have been liberalized.

The details of the CDFM method are given in Chapter 2 of the EPRI Seismic Margin Report (EPRI, 1987). The basic approach is to judiciously select the parameter values of different variables (e.g., strength, damping, ductility, load combination, and response analysis methods), taking into account the margins and uncertainties. The object is to obtain a conservative yet somewhat realistic assessment of the capacity. Table 2-1, reproduced from Kennedy (1984), gives the highlights of the CDFM method.

2.3 Description of Fragility Analysis Method

In many seismic PRAs, the fragility of a component has been represented by a double lognormal model using three parameters: (1) median ground acceleration capacity A_m, logarithmic standard deviations β_R and β_U representing, respectively, (2) randomness in the capacity and (3) uncertainty in the median value. Using the double lognormal model, the fragility curves as shown in Figure 2-1 are developed. The median capacity, β_R and β_U, are estimated using design-analysis information, test data, earthquake experience data, and engineering judgment (Kennedy et al., 1980; Kennedy and Ravindra, 1984). The median capacity can be estimated as a product of an overall median safety factor times the SSE peak ground acceleration for the plant. The overall safety factor is a product of a number of factors representing the conservatisms at different stages of analysis and design. When the linear scaling of response is not appropriate (e.g., soil sites), the median capacity is evaluated directly using median structural and equipment response parameters, median material properties and ductility factors, median static capacity predictions, and realistic structural modeling and method of analysis.

The HCLPF capacity is calculated using this fragility model as:

$$\text{HCLPF capacity} = A_m \exp [-1.65 (\beta_R + \beta_U)]$$

Further details on the development of A_m, β_R, and β_U values for a given component may be obtained from the cited references.

Table 2-1

SUMMARY OF CONSERVATIVE DETERMINISTIC FAILURE MARGIN APPROACH

Load Combination:	Normal + Seismic Margin Earthquake
Ground Response Spectrum:	Conservatively specified (preferably 84% Non-Exceedance Probability Site-Specific Spectrum, if available)
Damping:	Conservative estimate of median damping
Structural Model:	Best Estimate (Median) + Uncertainty Variation in frequency
Soil-Structure-Interaction:	Best Estimate (Median) + Parameter Variation
Material Strength:	Code Specified minimum strength or 95% exceedance actual strength if test data are available.
Static Capacity Equations:	Code ultimate strength (ACI), maximum strength (AISC), Service Level D (ASME), or functional limits. If test data are available to demonstrate excessive conservatism of code equations, then use 84% exceedance of test data for capacity equation.
Inelastic Energy Absorption: (ductility)	For non-brittle failure modes and linear analysis, use 80% of computed seismic stress in capacity evaluation to account for ductility benefits, or perform nonlinear analysis and go to 95% exceedance ductility levels.
In-structure (Floor) Spectra Generation:	Use frequency shifting rather than peak broadening to account for uncertainty plus use median damping.

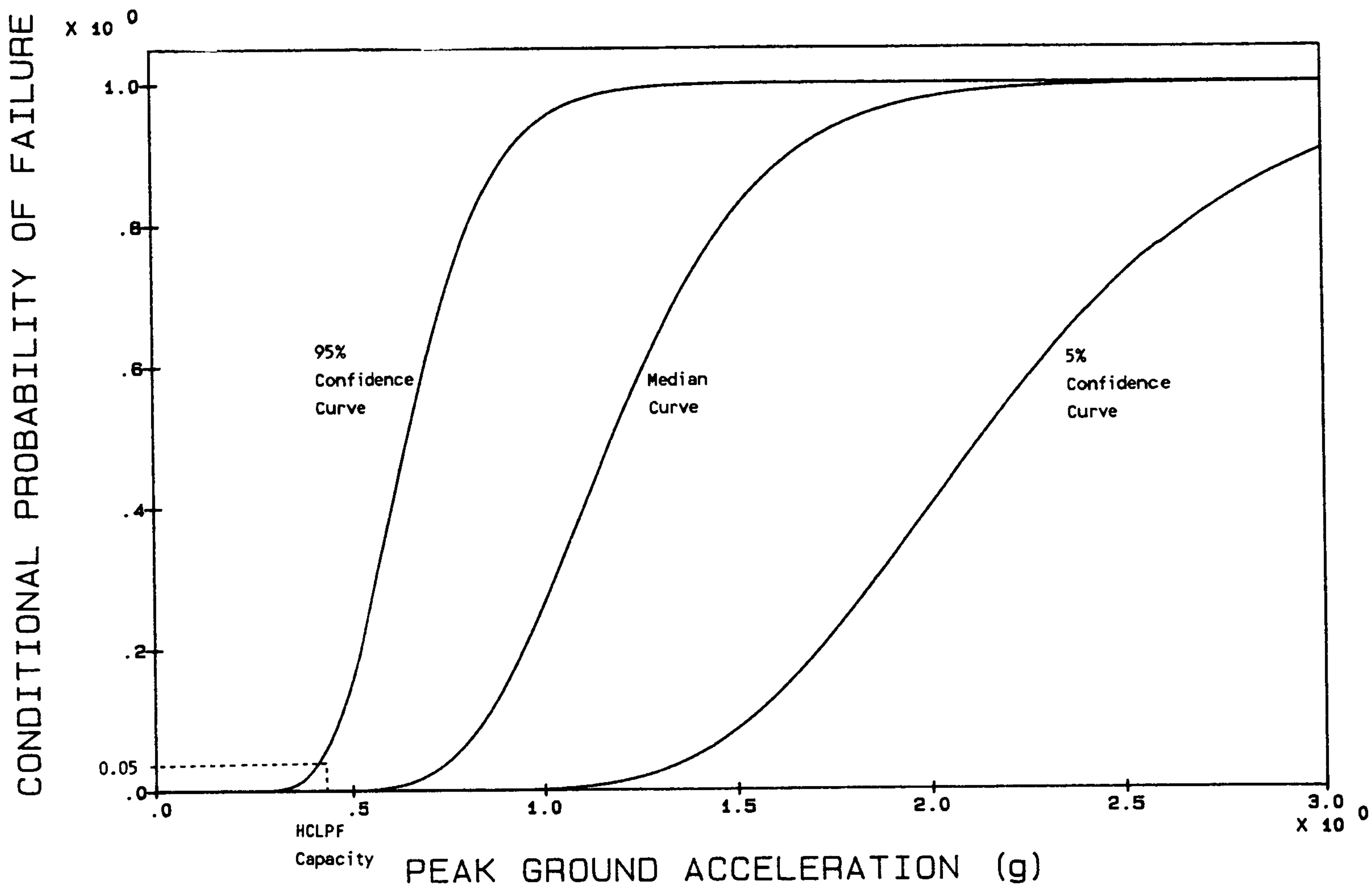


Figure 2-1: Typical Fragility Curves for a Component

CHAPTER 3
HCLPF CAPACITY CALCULATIONS PERFORMED BY THE STUDY GROUP

The calculations performed by the Study Group are discussed in this chapter. The basis for selection of components, description of components, and ground rules agreed upon by the Group for calculating the capacities are described. The results of the first round of calculations and the underlying reasons for differences in the capacities calculated by the investigators are discussed. The results of the second round of calculations are presented along with an analysis of the differences in the HCLPF capacities and median capacities as reported by the Study Group members.

3.1 Components Selected for Study

The objectives of the study were to compare the HCLPF capacity calculations performed by different analysts using either the CDFM method or the FA method. The comparison was performed by selecting a set of representative components. Also, the purpose of the comparison study was to modify the HCLPF capacity determination procedures if needed.

Components for which HCLPF capacity estimates are likely to be made in future seismic margin reviews include:

- o Block walls
- o Heat exchangers
- o Tanks
- o Active electrical equipment
- o HVAC fans and cooler units

Based on a review of these components, the Study Group selected the following components for performing seismic capacity calculations.

1. **Flat-Bottom Vertical Water Storage Tank at grade.** The failure mode to be considered is the combined bolt yielding and shell buckling. Water hold down force and the fluid stabilization effect should be taken into account.

2. **Auxiliary Contactor Chatter for Motor Starter in an older Motor Control Center (MCC). This example will focus on equipment qualified using Generic Equipment Ruggedness Spectrum (GERS). The failure mode to be investigated is the auxiliary contactor chatter. No qualification test data exists since the component is assumed to be in an older plant. The cabinet is well anchored and calculations were performed for cabinets mounted at grade and high up in the building. It is assumed to have an estimated frequency of 6.5 Hz. It was agreed that the seismic HCLPF capacity would be based upon the "Function-during GERS."**

3. **Diesel Generator Room Starting Air Tank mounted high in the structure.** This is an example of a vertical skirt-mounted pressure vessel on a concrete floor high up in the building. The failure mode to be investigated is the anchorage or support failure and not skirt buckling.

4. **Component Cooling Heat Exchanger mounted high in the structure.** This is an example of a component governed by ASME rules. It is a horizontal heat exchanger fixed on one saddle support and free longitudinally at the other support. It is assumed to be bolted to a rigid support frame. The area of concern is the saddle and anchorage.

5. **Cantilevered Reinforced Block Wall mounted high in the structure.** This example is selected to avoid the case of a wall where arching action may be present. If arching action is present, the wall would have higher capacity; therefore a cantilevered wall with reinforcing steel is chosen.

Three tanks or vessels (air receiver tank, heat exchanger, and flat-bottom vertical tank), one structural component (reinforced block wall) and one electrical component (auxiliary contactor in motor starter) are included in this set. The analysis includes two different locations in the building (at grade and at a floor high in the building). This set covers typical features of components that may require seismic margin evaluation.

Figures 3-1 through 3-4 schematically illustrate the important seismic capacity aspects of the storage tank, starting air receiver tank supports, heat exchanger supports, and block wall, respectively. Figure 3-5 shows the "Function-during" GERS as reproduced from the ANCO Report (1987).

The question of the adequacy of this set for comparing the CDFM and FA methods was discussed. The Study Group judged that substantial understanding of the applicability of the methods could be obtained by focusing on this limited set of components. Another important feature being investigated is how different investigators would apply the CDFM and FA methods to identical components and how much variation could be expected between the capacity predictions by different investigators.

3.2 Ground Rules for Capacity Calculations

The following ground rules were established for performing the seismic capacity calculations:

- **Each consultant agreed to perform seismic capacity calculations on the 5 components listed above using methods he would use in production type Seismic Margin Review computations.**

- **Each consultant agreed to use the same structural properties for each component. These are illustrated in Figures 3-1 through 3-4.**

- Both median and HCLPF capacities of the components were evaluated. In the fragility analysis, the median capacity, and β_R and β_U were estimated.

- In the first round of calculations, Kennedy and Ravindra agreed to use both CDFM and FA methods. Reed agreed to calculate the capacity using the FA method. Stevenson calculated the HCLPF capacity using the EPRI (1987) method with a probabilistic procedure used to determine median values (Stevenson 1985). Thus comparisons exist between different investigators using the same method and between the same investigators using the CDFM and FA methods.

- As a separate study, Reed took the results from the FA approach and determined the CDFM input necessary to derive the same HCLPF capacity; the purpose was to investigate the distribution of conservatisms in different parts of the CDFM method. The results of this study are included in Reed's calculations in the Appendix.

3.3 Ground Motion Aspects

For all HCLPF capacity computations, it was agreed to assume that the ground motion for the largest horizontal component was given by a uniform hazard spectrum defined at the 84% non-exceedance probability (NEP) at all frequencies. Furthermore, this uniform hazard spectrum shape was to be defined by the NUREG/CR-0098 (Newmark and Hall, 1978) median spectrum shape for rock sites (Figure 3-6). The 84% vertical response spectrum was then defined to be equal to 2/3 of the 84% NEP largest horizontal response spectrum.

The result of using the 84% NEP largest horizontal component response spectrum to determine the HCLPF capacity is that this capacity is conditional on this response spectrum not being exceeded at more than 16% of the frequencies in the frequency range and directions that dominate the component capacity.

The above ground motion information is sufficient to enable HCLPF capacity computations to be made using the CDFM method. However, the Fragility Analysis method requires additional description of the random variability of the ground motion response spectrum. Variability associated with peaks and valleys of actual response spectrum and directional response variability should be included. Each consultant using the Fragility Analysis method was expected to make his own estimate of these sources of response spectrum variability.

3.4 Floor Spectrum Aspects

It was agreed that the floor spectra shown in Figures 3-7 and 3-8 would be used to represent floor spectra obtained high in a structure from a median centered building response model subjected to median NUREG/CR-0098 response spectra (Figure 3-6) anchored to a horizontal peak ground acceleration (PGA) of 0.18 g (this was assumed to be the 84% NEP ground response spectrum as in Section 3.3).

Figures 3-7 and 3-8 represent horizontal and vertical floor response spectrum respectively. Since these floor spectra are assumed to represent unbroadened median response to an 84% NEP ground motion input, they may be used directly without scaling in the CDFM method or the Fragility Analysis method for a review earthquake of 0.18 g PGA. Linear scaling is used for other earthquake levels.

3.5 Discussion of Failure Modes

In the evaluation of seismic margin or seismic fragilities, it is important to define what is meant by failure of the component. In the following, the failure modes as identified by the Study Group members are discussed for the components analyzed.

Flat Bottom Tank. Failure of the vertical storage tank is defined to be gross loss of fluid contents. Horizontal seismic load initiates uplift of the tank shell from its foundation. This uplift is resisted by the anchor bolts, the tank bottom plate, and the tank weight. The anchor bolts are permitted to yield, so long as their behavior is ductile, since yielding does not directly result in loss of fluid contents. Shell compressive stresses progressively increase until buckling occurs.

Motor Control Centers. A functional failure mode associated with the motor control centers was assumed to be governed by chatter of auxiliary contactors for purposes of this study. This may result in spurious signals and may adversely affect any equipment controlled by auxiliary contactors.

Starting Air Receiver Tank. The starting air receiver tank is a vertical, skirt-supported cylindrical tank which is anchored to the building floor by three angles welded to the tank skirt and bolted to the floor. The leg of the angle was found to be much weaker in bending than the anchor bolts. The angle leg is very ductile in bending, and the failure mode is tearing the mounting angles. When this occurs, the air tank is assumed to fail through failure of attached piping.

Horizontal Heat Exchanger. The failure mode governing the median capacity of the horizontal heat exchanger was combined tension and shear induced failure of the anchor bolts. Tension results from overturning of the heat exchanger in the lateral direction, while shear results from inertial loads in both horizontal directions. When this anchorage failure occurs, the heat exchanger is assumed to fail through failure of the nozzles and attached piping.

Block Wall. The block wall is represented as a vertical cantilever fixed at its base. Failure is collapse or excessive lateral distortion of the wall.

3.6 Results of First Round of Calculations

The four investigators independently estimated the HCLPF and median seismic capacities of the components described in Section 3.1. The results of the calculations are summarized in Table 3-1, and the complete set of calculations are included in the Appendix. Kennedy and Stevenson initially performed the calculations using deterministic methods; Ravindra and Reed initially performed the calculations using the Fragility Analysis method. Kennedy also calculated the HCLPF capacities of components using the Fragility Analysis method; Ravindra also independently calculated the HCLPF capacities using the deterministic method. Kennedy, Ravindra and Stevenson performed the deterministic HCLPF calculations using their individual interpretations of the CDFM method as described in the EPRI Seismic Margin Methodology report (EPRI, 1987). Therefore, their results are directly comparable for the EPRI CDFM method. Kennedy, Reed and Ravindra each performed fragility analysis computations using their independent interpretations of the parameters involved, but they used a common methodology so that all HCLPF capacities computed by the FA method are comparable.

Failure modes for different components identified by the investigators are also included in Table 3-1. In general, the same failure mode was identified for each component type; however, there were differences in the estimation of the seismic capacity of the component for the identified failure mode.

3.6.1 Comparison of CDFM and FA HCLPF Capacities

Since Kennedy and Ravindra performed both CDFM and FA calculations following identical approaches (although making different judgments in their application), Table 3-2 compares the range of their two sets of results by both methods. This comparison was judged to be the most appropriate for HCLPF capacities by the CDFM and FA methods.

In all of the example cases, the CDFM method produced less difference in the HCLPF capacity estimates between Kennedy and Ravindra than did the FA estimate. This is to be expected since the parameters of the CDFM method are more specified than the parameters used in the FA method. The ratio of High/Low estimates by the CDFM method ranged from 1.00 to 1.29 with a median value of 1.11. For the FA method, this ratio ranged from 1.11 to 1.57 with a median of 1.30.

In all of Kennedy's computations the FA method produced a higher HCLPF capacity than did the CDFM method. In all of Ravindra's computations the FA method produced a lower or equal HCLPF capacity than did the CDFM method. Therefore, there is no conclusion as to whether the current CDFM method is more or less conservative than the FA method for estimating the HCLPF capacities.

3.6.2 Comparison of HCLPF and Median Capacities

The range of HCLPF capacities from the four investigators was larger than expected. The ratio of high to low HCLPF capacity estimates ranged from 1.23 to 1.82, with a median of 1.48 (Table 3-3). The differences in estimated HCLPF capacities between the four investigators was much larger than the differences in results between the CDFM and FA methods, and this was due primarily to differences in the subjective judgment and personal experience of the investigators. The differences were due to the relative degree of conservatism introduced into each investigator's computations.

Each investigator also independently estimated the median capacities. The high to low ratio of median capacity estimates ranged from 1.23 to 2.13, with a median of

1.47. Thus, there was no closer agreement in the estimates of the HCLPF capacities than there was in the estimates of the medians. This conclusion was surprising and contradictory to the judgment of the NRC Expert Panel on Seismic Margins, which believed that greater agreement would exist in the computations of HCLPF capacities than for median capacities.

There was also essentially as large a difference in the estimates of the ratio of median/HCLPF capacity. For the two components mounted at grade, the median of this ratio was 2.39, but the range for the four investigators was 1.96 to 4.05. For the four components high in the structure, the median ratio was 3.07 but ranged for the four investigators from 2.23 to 5.14. The ratio of the High/Low prediction of the ratio of median/HCLPF capacity for all components ranged from 1.25 to 1.80, with a median of 1.65. Therefore, even if the four investigators had agreed on estimates of either the median or HCLPF capacities, they would have had equally substantial differences in converting medians to HCLPF capacities, or vice versa.

3.6.3 Overall Conclusions from the First Round Calculations

The results of the first round calculations indicate that differences in the HCLPF capacity calculations using the CDFM method and the FA method are indeed small relative to the differences in capacity calculated by four independent investigators. The four investigators differ from each other by a median ratio of about 1.5 to 1.7 for estimates of the HCLPF capacity, the median capacity, and the ratio of median/HCLPF capacity. Thus, all three quantities are nearly equally uncertain. Hence, it was concluded that further comparisons of the HCLPF capacity calculations by the CDFM method versus the FA method is not beneficial. Further study should be directed at achieving greater agreement between investigators as opposed to refining differences between the CDFM and FA methods. The major differences are in the selection of models and parameter values.

3.7 Results of Second Round of Calculations

Based on a review of the first round calculations and subsequent discussions by the Study Group, the investigators were allowed to revise their calculations and state the HCLPF and median capacity for each component. The results of this round of calculations are summarized in Table 3-4. Further details are included in the Appendix. A review of the two sets of calculations (Tables 3-1 and 3-4) has indicated that the investigators converged in those cases where the differences were not initially large (flat bottom storage tank, starting air tank, and heat exchanger). In those cases where the differences were initially large, the second round calculations did not bring the capacities closer; in fact they moved farther apart in some cases (cabinet high up, and blockwall).

The four investigators differ from each other by a median of the high/low ratios of 1.39 to 1.55 for estimates of the HCLPF capacity, the median capacity, and the ratio of median/HCLPF capacity, (See Table 3-5). As before, these differences are mainly due to the differences in models, parameters, and assumptions used by the investigators.

Table 3-1

Comparison of HCLPF Computations For

Representative Components (First Round Calculations)

Component	HCLPF Capacity (g) CDFM	HCLPF Capacity (g) FA	Median Capacity (g)	Failure Mode	Comments/Remarks/Assumptions
Flat Bottom Storage Tank (At Grade)					
RPK	0.29	0.31	0.67	Combination of shell buckling and anchor bolt yields	
MKR/PSH	0.29	0.26	0.54		
JWR	----	0.27	0.53		
JDS	0.32	----	1.13	Yield of anchor bolts	
Auxiliary Contactor Chatter (Function during GERS lock-in circuit potential)					
a) Cabinet at Grade					
RPK	0.54	0.59	1.26	Contactor Chatter	
MKR/RDC	0.47	0.39	1.58	Contactor Chatter	(used 0.87 knock-down factor)
JWR	----	0.48	1.20	Contactor Chatter	
JDS	0.71	----	1.88	Contactor Chatter	
b) Cabinet High-up					
RPK	0.10	0.11	0.30	Contactor Chatter	
MKR/RDC	0.09	0.07	0.36	Contactor Chatter	(used 0.87 knock-down factor)
JWR	----	0.11	0.43	Contactor Chatter	
JDS	0.12	----	0.43	Contactor Chatter	

RPK Did calculations by CDFM first, then FA.

MKR/RDC/PSH Did calculations by FA first, then CDFM.

JWR Tabulated values are from HCLPFs calculations using input spectra as 84% NEP maximum horizontal direction.

JDS Did Calculations by CDFM.

Table 3-1 (Continued)

Comparison of HCLPF Computations For

Representative Components (First Round Calculations)

Component	HCLPF Capacity (g)		Median Capacity (g)	Failure Mode	Comments/Remarks/Assumptions
	CDFM	FA			
Starting Air Tank (High-up)					
RPK	0.48	0.50	1.07	Plastic Bending of Mounting Angles	
MKR/RDC	0.53	0.44	1.55	Plastic Bending of Mounting Angles	
JWR	----	0.43	1.40	Plastic Bending of Mounting Angles	
JDS	0.39	----	1.04	Plastic Bending of Mounting Angles	
Heat Exchanger (High-up, Bolted to Rigid Support Frame)					
RPK	0.40	0.42	1.18	Anchor Bolt Shear Failure; failure through the threads	
MKR/RDC	0.44	0.38	1.08	Anchor Bolt Shear & Tension Failure	
JWR	----	0.39	1.00	Anchor Bolt Shear Failure failure through the threads	
JDS	0.30	----	0.96	Anchor Bolt Shear Failure	
Block Wall (High-up)					
RPK	0.62	0.67	1.94	Out-of-Plane Bending	
MKR/PSH	0.48	0.48	1.55	Out-of-Plane Bending	
JWR	----	0.38	1.41	Out-of-Plane Bending	
JDS	0.51	----	1.34	Out-of-Plane Bending	

RPK Did calculations by CDFM first, then FA.
MKR/RDC/PSH Did calculations by FA first, then CDFM.
JWR Tabulated values are from HCLPF capacity calculations using input spectra as 84% NEP maximum horizontal direction.
JDS Did Calculations by CDFM.

Table 3-2

Comparison of EPRI CDFM and FA HCLPF
(Kennedy and Ravindra Comparable Computations)

	HCLPF Range (g)		High/Low Ratio	
	CDFM	FA	CDFM	FA
Flat Bottom Tank	0.29	0.26 - 0.31	1.00	1.19
Cabinet at Grade	0.47 - 0.54	0.39 - 0.59	1.15	1.51
Cabinet on Floor	0.09 - 0.10	0.07 - 0.11	1.11	1.57
Air Tank	0.48 - 0.53	0.44 - 0.50	1.10	1.14
Heat Exchanger	0.40 - 0.44	0.38 - 0.42	1.10	1.11
Block Walls	0.48 - 0.62	0.48 - 0.67	1.29	1.40
		Mean	1.13	1.32
		Median	1.11	1.30

Table 3-3

Comparison of HCLPF and Median Capacities

First Round Calculations

Component	Range (g)		High/Low Ratio		Median/HCLPF Ratio		
	HCLPF	Median	HCLPF	Median	Median	Range	High/Low Ratio
Flat Bottom Tank	0.26-0.32	0.53-1.13	1.23	2.13	2.20	1.96-3.53	1.80
Cabinet at Grade	0.39-0.71	1.20-1.88	1.82	1.57	2.58	2.33-4.05	1.74
Cabinet on Floor	0.07-0.12	0.30-0.43	1.71	1.43	3.75	3.00-5.14	1.71
Air Tank	0.39-0.48	1.04-1.55	1.23	1.49	2.96	2.23-3.52	1.58
Heat Exchanger	0.30-0.40	0.96-1.18	1.33	1.23	2.90	2.56-3.20	1.25
Block Walls	0.38-0.62	1.34-1.94	1.63	1.45	3.18	2.63-3.71	1.41
		Mean	1.49	1.55			1.58
		Median	1.48	1.47			1.65

Table 3-4

Comparison of HCLPF Capacity Computations For

Representative Components (Second Round Calculations)

Component	HCLPF Capacity (g)	Median Capacity (g)	Failure Mode	Comments/Remarks/Assumptions
Flat Bottom Storage Tank (At Grade)				
RPK	0.29	0.67	Combination of shell buckling and anchor bolt yields	
MKR/PSH	0.29	0.54		
JWR	0.28	0.55		
JDS	0.32	0.83	Yield of anchor bolts	
Auxiliary Contractor Chatter (Function during GERS lock-in circuit potential)				
a) Cabinet at Grade				
RPK	0.54	1.26	Contactor Chatter	
MKR/RDC	0.47	1.58	Contactor Chatter	(used 0.87 knock-down factor)
JWR	0.48	1.20	Contactor Chatter	
JDS	0.71	1.88	Contactor Chatter	
b) Cabinet High-up				
RPK	0.10	0.30	Contactor Chatter	
MKR/RDC	0.09	0.36	Contactor Chatter	(used 0.87 knock-down factor)
JWR	0.11	0.43	Contactor Chatter	
JDS	0.15	1.88*	Contactor Chatter	

* See calculations (Appendix) for further explanation.

Table 3-4 (Continued)

Comparison of HCLPF Capacity Computations For

Representative Components (Second Round Calculations)

Component	HCLPF Capacity (g)	Median Capacity (g)	Failure Mode	Comments/Remarks/Assumptions
Starting Air Tank (High-up)				
RPK	0.48	1.07	Plastic Bending of Mounting Angles	
MKR/RDC	0.53	1.55	Plastic Bending of Mounting Angles	
JWR	0.43	1.40	Plastic Bending of Mounting Angles	
JDS	0.42	1.10	Plastic Bending of Mounting Angles	
Heat Exchanger (High-up, Bolted to Rigid Support Frame)				
RPK	0.40	1.18	Anchor Bolt Shear Failure; failure through the threads	
MKR/RDC	0.44	1.08	Anchor Bolt Shear & Tension Failure	
JWR	0.39	1.00	Anchor Bolt Shear Failure; failure through the threads	
JDS	0.44	1.15	Anchor Bolt Shear Failure	
Block Wall (High-up)				
RPK	0.62	1.94	Out-of-Plane Bending	
MKR/PSH	0.63	2.10	Out-of-Plane Bending	
JWR	0.52	1.96	Out-of-Plane Bending	
JDS	0.32	1.30	Out-of-Plane Bending	

Table 3-5

Comparison of HCLPF and Median Capacities

Second Round Calculations

Component	Range (g)		High/Low Ratio		Median/HCLPF Ratio		
	HCLPF	Median	HCLPF	Median	Median	Range	High/Low Ratio
Flat Bottom Tank	0.28-0.32	0.54-0.83	1.14	1.54	2.13	1.86-2.59	1.39
Cabinet at Grade	0.47-0.71	1.20-1.88	1.51	1.57	2.58	2.33-3.36	1.44
Cabinet on Floor	0.09-0.15	0.30-1.88	1.67	6.27	3.96	3.00-12.53	4.18
Air Tank	0.42-0.53	1.07-1.55	1.26	1.45	2.77	2.23-3.26	1.46
Heat Exchanger	0.39-0.44	1.00-1.18	1.13	1.18	2.59	2.45-2.95	1.20
Block Walls	0.32-0.63	1.30-2.10	1.97	1.62	3.55	3.13-4.06	1.30
		Mean	1.45	2.27			1.83
		Median	1.39	1.55			1.42

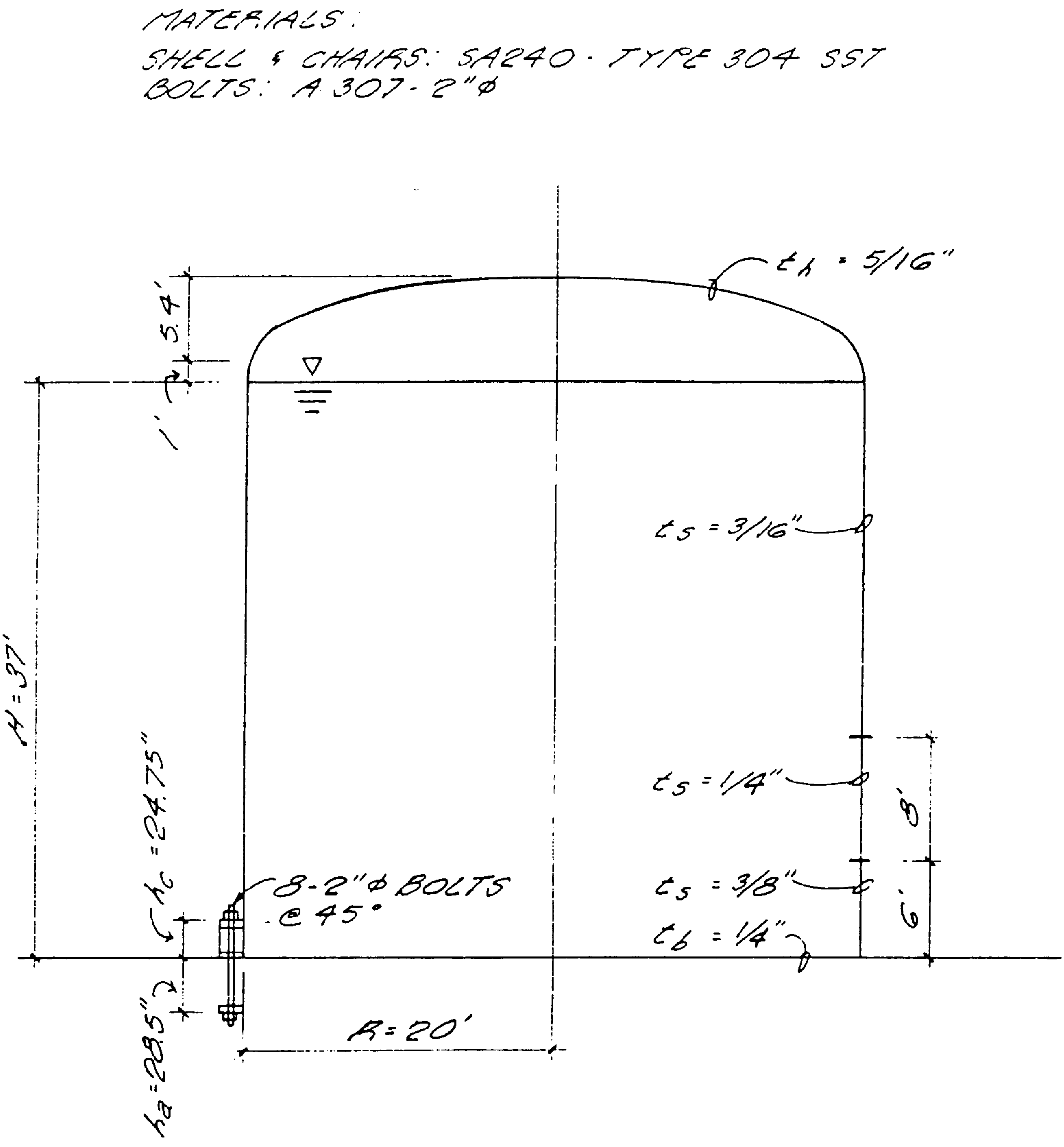


Figure 3-1: Flat Bottom Vertical Water Storage Tank.

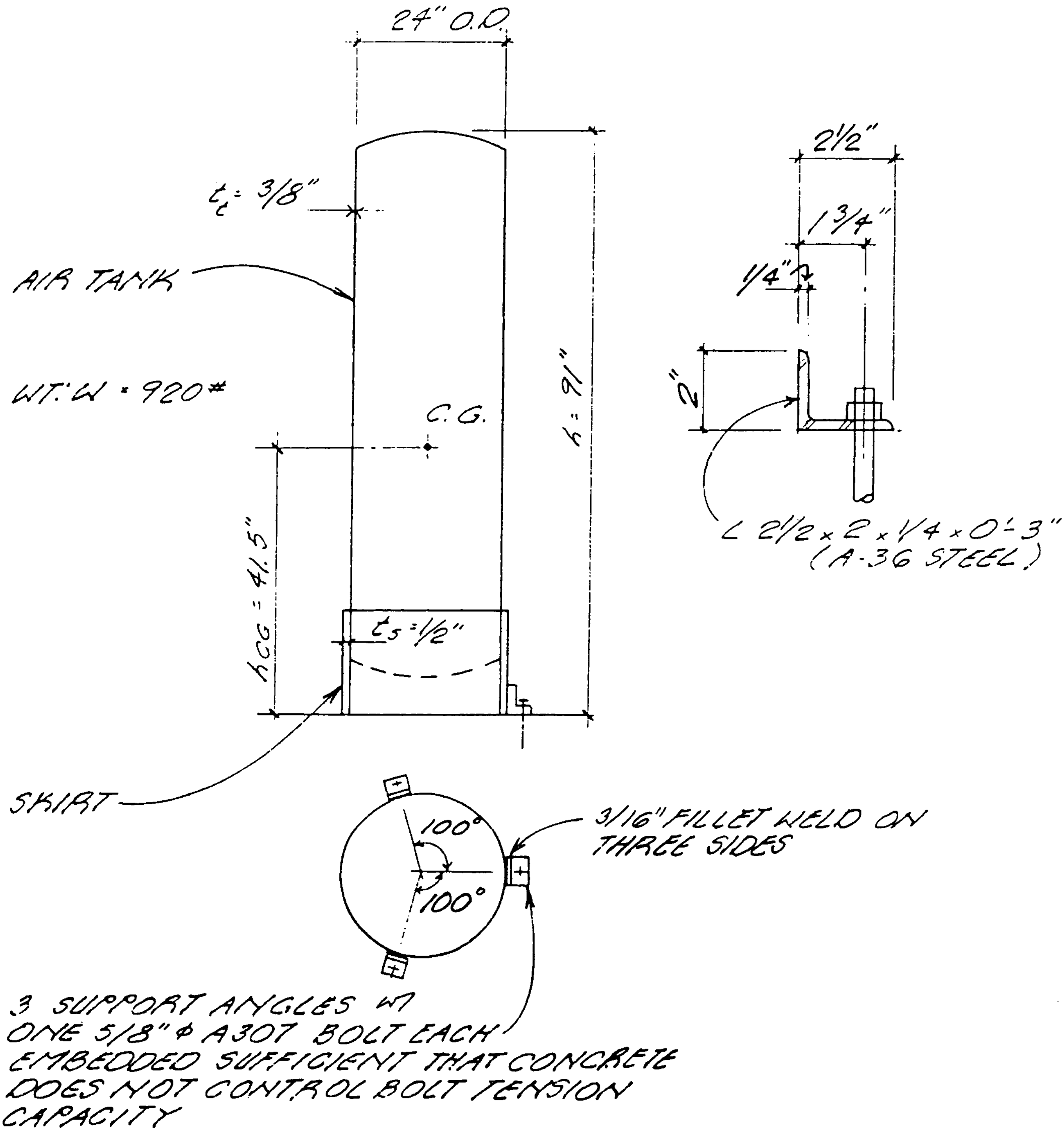


Figure 3-2: Diesel Generator Room Starting Air Tank Supports.

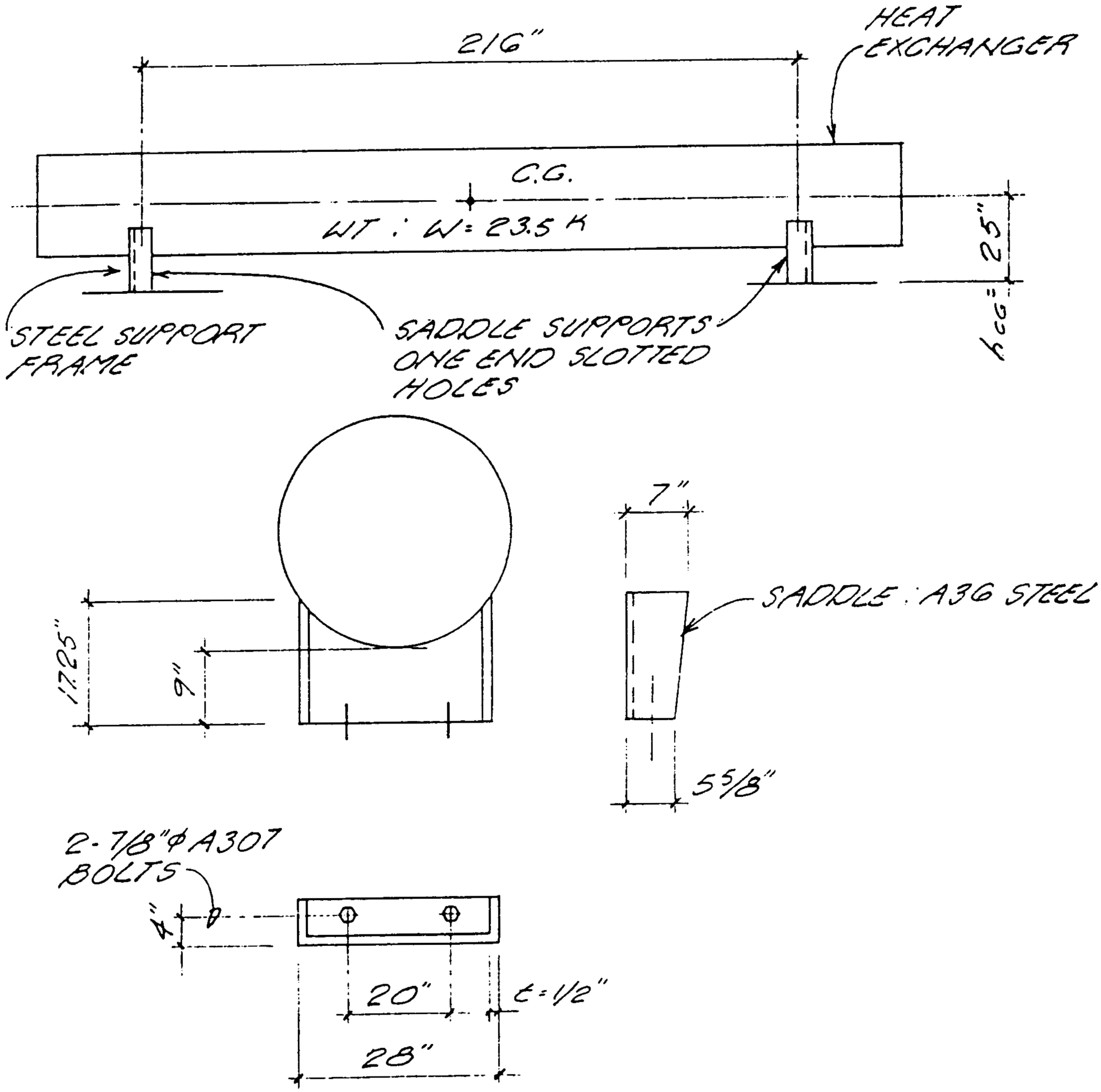
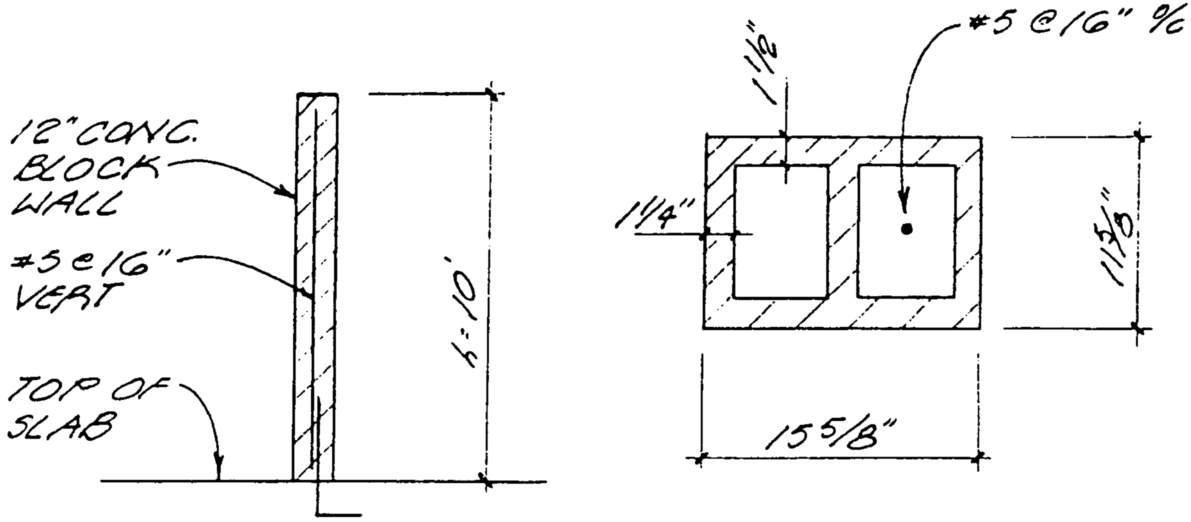


Figure 3-3: Component Cooling Heat Exchanger Supports.



SPECIAL INSPECTION REQUIREMENT, ARE MET.

WALL IS CONSTRUCTED IN RUNNING BOND, CELLS W/
REINF. GROUTED SOLID
BLOCK : 12" HOLLOW UNITS, NORMAL WEIGHT,
ASTM C90 GRADE N, f'_c = 3000 PSI
MORTAR: ASTM C270 TYPE S
VERT. REINF: #5 @ 16" o/c GRADE 60
HORIZ. REINF: EXTRA HEAVY, 3 ROD DUROWAL @ 16" o/c
WALL LONG COMPARED TO HEIGHT : ACTS AS
VERTICAL CANTILEVER
WALL WT: W = 111 PSF

Figure 3-4: Cantilever Reinforced Block Wall.

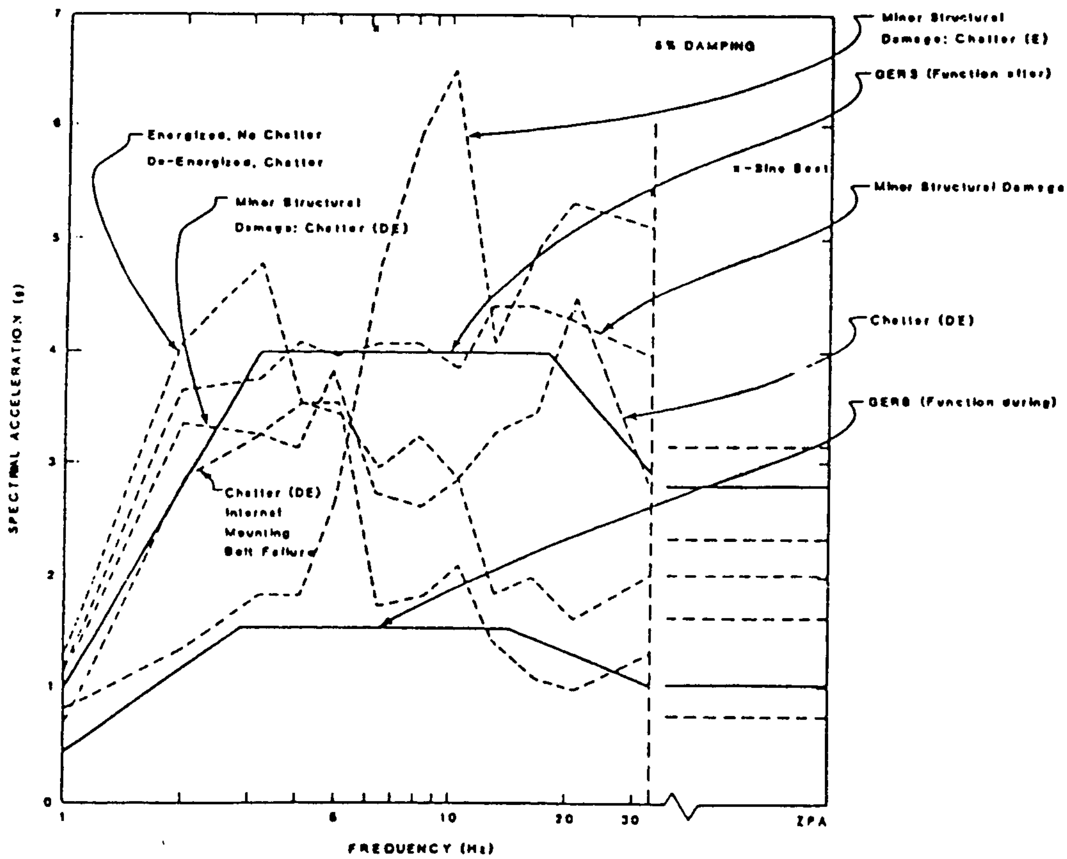


Figure 3-5: Comparison of GERS with Failure Data: Function During and After for MCC (ANCO 1987)

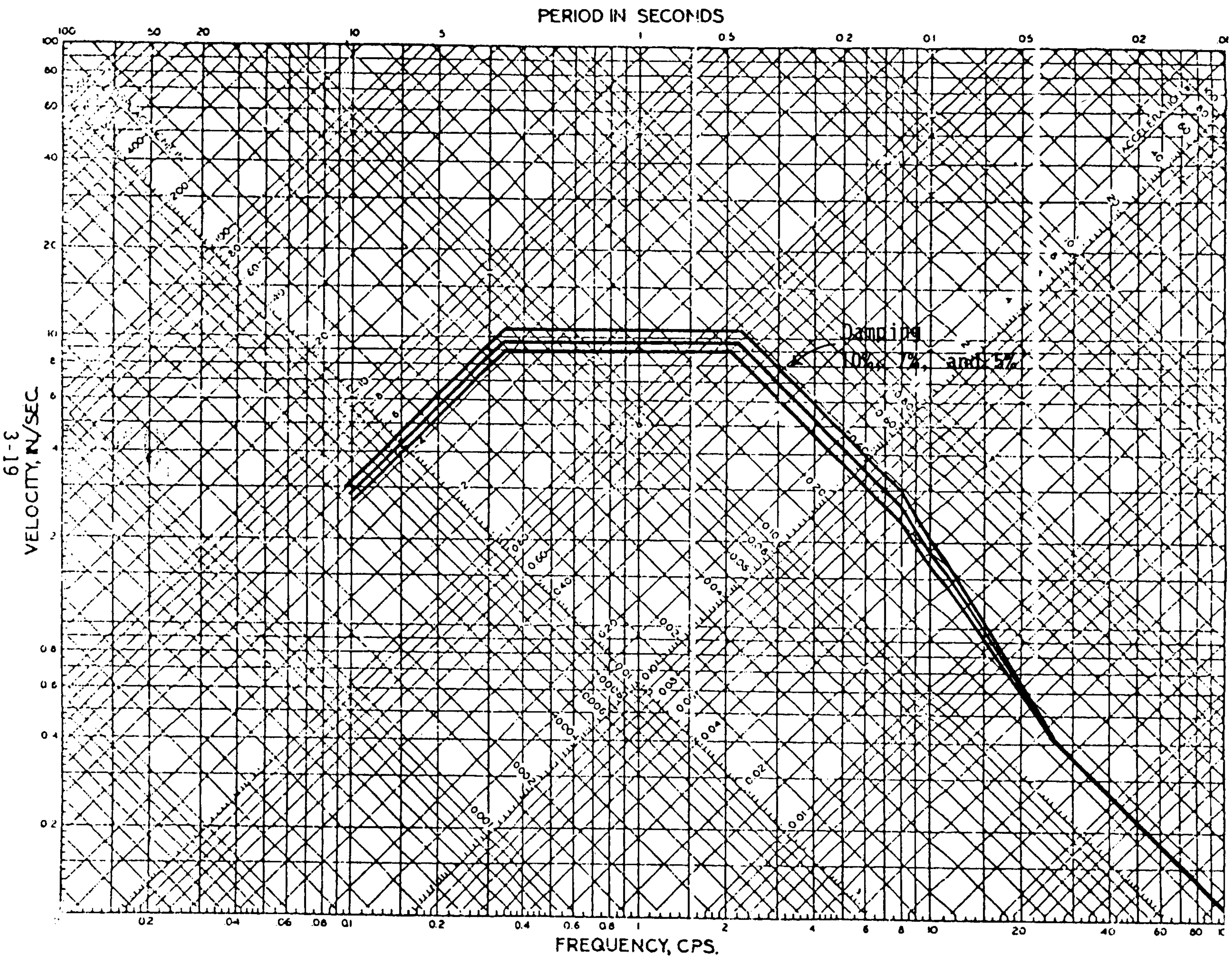
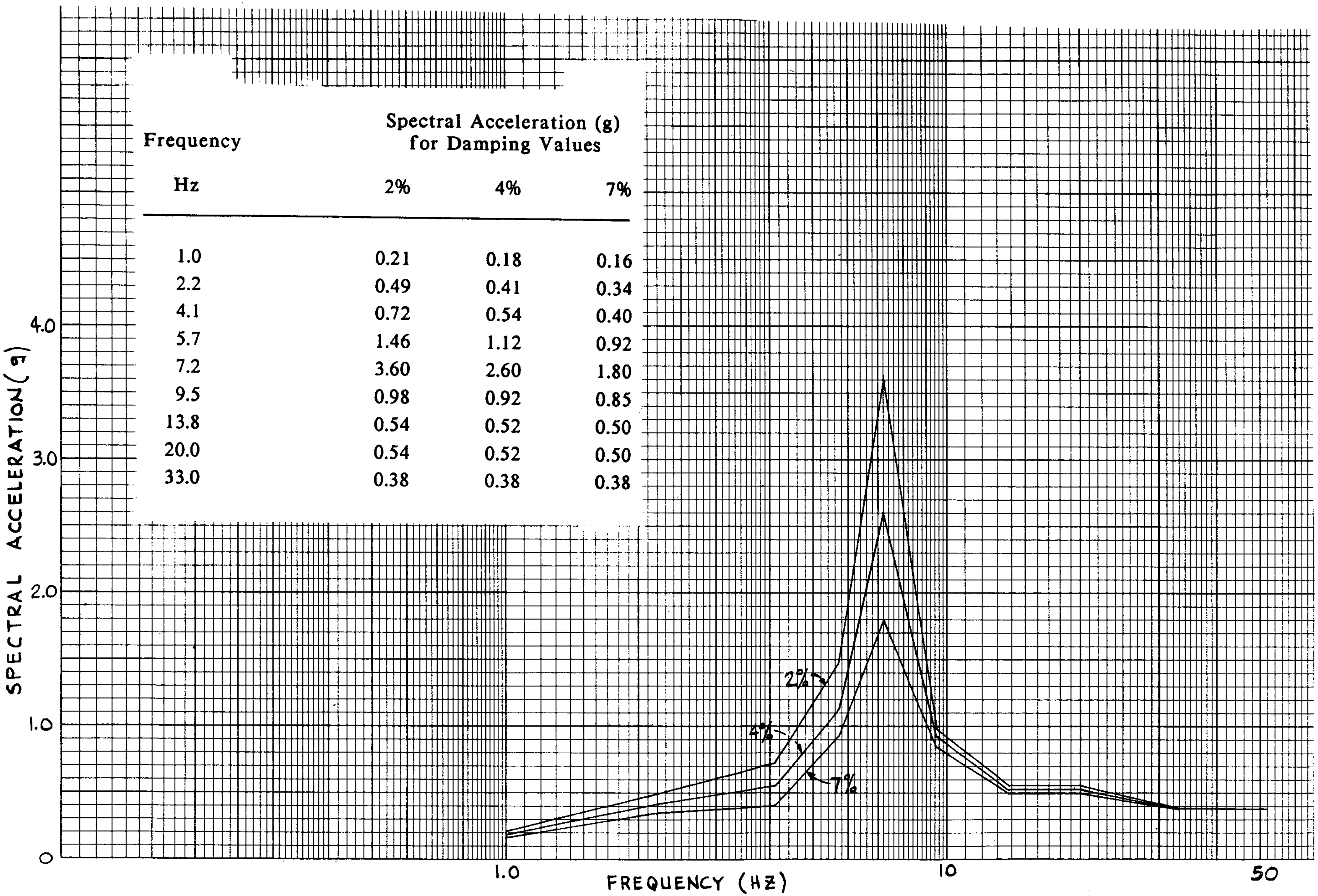
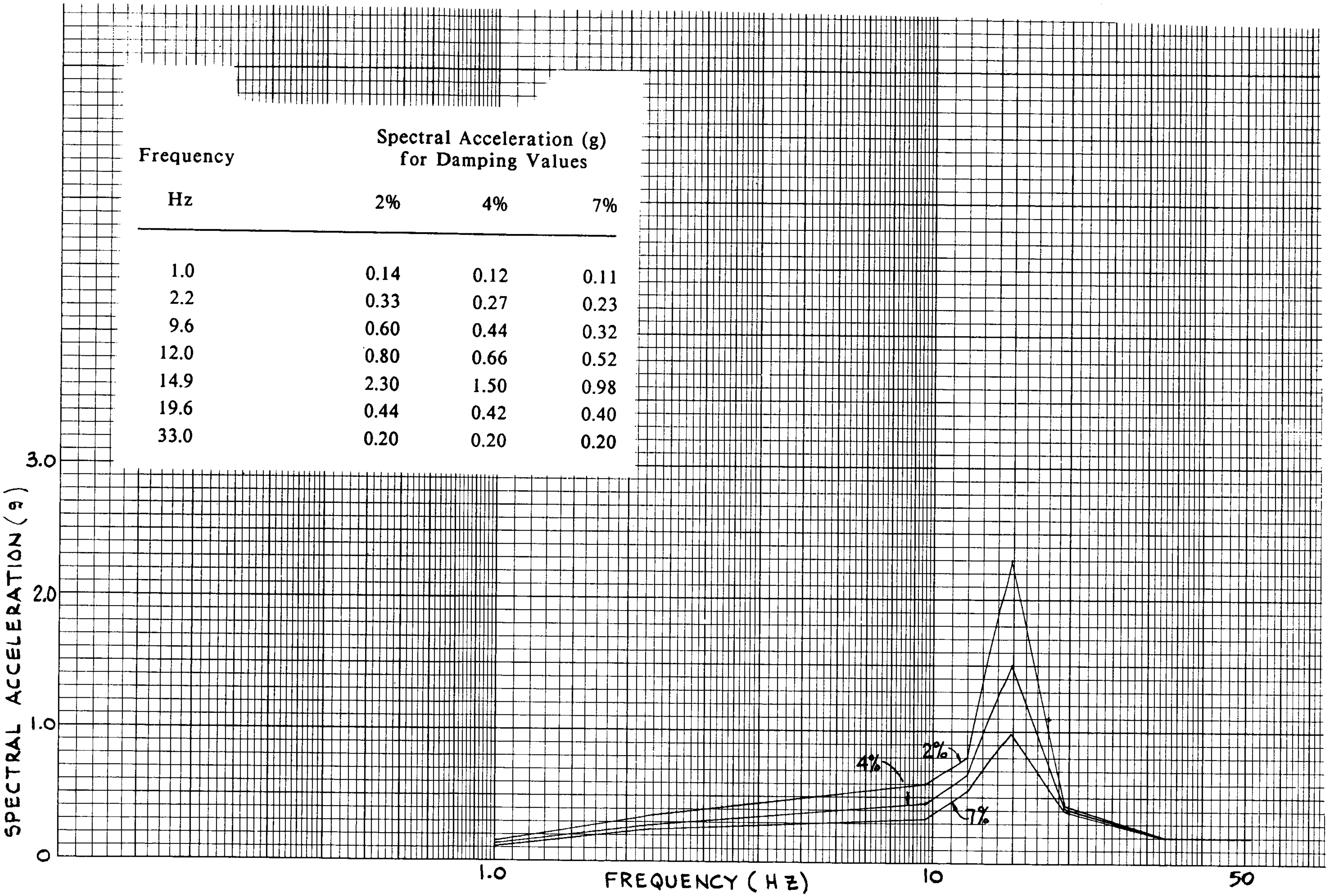


Figure 3-6: Median Ground Response Spectra Anchored to 0.18 pga.



Frequency Hz	Spectral Acceleration (g) for Damping Values 2%	4%	7%
1.0	0.21	0.18	0.16
2.2	0.49	0.41	0.34
4.1	0.72	0.54	0.40
5.7	1.46	1.12	0.92
7.2	3.60	2.60	1.80
9.5	0.98	0.92	0.85
13.8	0.54	0.52	0.50
20.0	0.54	0.52	0.50
33.0	0.38	0.38	0.38

Figure 3-7: Horizontal Floor Spectra



Frequency Hz	Spectral Acceleration (g) for Damping Values 2%	4%	7%
1.0	0.14	0.12	0.11
2.2	0.33	0.27	0.23
9.6	0.60	0.44	0.32
12.0	0.80	0.66	0.52
14.9	2.30	1.50	0.98
19.6	0.44	0.42	0.40
33.0	0.20	0.20	0.20

Figure 3-8: Vertical Floor Spectra

CHAPTER 4
SUMMARY AND CONCLUSIONS

4.1 Summary

This study evaluated the CDFM and FA methods for calculating the HCLPF capacities of components. This was accomplished by performing the HCLPF capacity calculations on a representative set of components using the two methods by four investigators. The components selected were: flat-bottom vertical storage tank, auxiliary contactor for motor starter in an older MCC, starting air tank, component cooling heat exchanger, and cantilevered reinforced block wall. Two different locations in the building (at grade and at a floor high in the building) were studied for the MCC. The details of the components, ground response, and floor response spectra were provided.

In the first round of calculations, Kennedy and Ravindra used both the CDFM and FA methods to obtain the HCLPF capacities. They used the EPRI CDFM method (as discussed in Section 2.2). Reed calculated the capacity using the FA method. Stevenson calculated the HCLPF capacity using a modified SEP deterministic method.

Since Kennedy and Ravindra performed both CDFM and FA calculations following identical approaches, the comparison of their capacity estimates was deemed most appropriate. The ratio of high/low estimates of HCLPF capacities by the CDFM method ranged from 1.00 to 1.29, with a median value of 1.11. For the FA method, this ratio ranged from 1.11 to 1.57, with a median of 1.30.

The range of HCLPF capacities from the four investigators was larger than expected. The ratio of high to low HCLPF capacity estimates ranged from 1.23 to 1.82, with a median of 1.48. The differences in estimated HCLPF capacities between the four investigators was much larger than the differences in results between the CDFM method and FA method. This was primarily due to differences in the models, parameter values, and assumptions used by the investigators.

The differences in median capacity estimates ranged from 1.23 to 2.13, with a median of 1.47. Thus, there was only slightly closer agreement in the estimates of the HCLPF capacities than there was in the estimates of the medians.

Based on a review of the first round calculations and subsequent discussions by the Study Group, the investigators were allowed to revise their calculations and resubmit the HCLPF and median capacity for each component. The four investigators differed from each other by a median ratio of 1.39 to 1.55 in their final estimates of the HCLPF capacity, the median capacity, and the ratio of median/HCLPF capacity.

As before, these differences are mainly due to the differences in models, parameters, and assumptions used by the investigators. Their estimates converged in those cases where the differences were not initially large; they did not converge (and in some cases diverged further) in those cases where the differences were initially large.

4.2 Conclusions from this Study

1. The differences in estimated HCLPF capacities between the four investigators (median high-to-low ratio equals 1.48) are much larger than the differences in results between the CDFM methodology and the FA method.

2. These differences are primarily due to differences in the assumptions and judgments of the four investigators.

3. The differences between the investigators estimating the HCLPF capacities are similar in size to the differences in estimating the median capacities. This is contradictory to the judgment of the NRC Expert Panel on Seismic Margins. The Panel had stated that the median capacity of a component is difficult to calculate and could vary widely between the analysts. The HCLPF capacity was thought to be a more robust quantity which most engineers would agree upon. This has not been borne out by the calculations done by the Study Group. The four investigators differ from each other by a median of the ratio of high-to-low values of about 1.5 to 1.7 for estimates of the HCLPF capacity, the median capacity, and the ratio of median/HCLPF capacities.

4. The differences between the investigators' estimates of the median/HCLPF capacity ratios (median high-to-low ratio equals 1.5) are of the same size as estimates of either HCLPF capacities or median capacities individually. This indicates that differences in estimates of β_R and β_U are also relatively large.

5. Any future effort in reducing the differences in calculated HCLPF capacities should be spent in reducing the differences between the assumptions and judgment of different investigators rather than in trying to reconcile the much smaller differences between the CDFM and FA methods.

6. Although there was as much dispersion in the computed HCLPF capacities as there was in the computed median capacities, there was unanimous agreement that for each of the components all four investigators' HCLPF capacities represented a conservative estimate of the component capacity. Differences were over the degree of conservatism represented and not over whether conservatism existed or not. Thus, for each component, the highest HCLPF is still a conservative capacity estimate. Obviously, the same unanimity cannot exist at the median capacity since each separate estimate cannot represent the 50% failure probability, although all median estimates are within each investigator's 90% confidence band on the median. Thus, unanimity exists that all median estimates are credible medians for each component, and investigator differences simply reflect uncertainty about the median. In the face of uncertainties in both the median and HCLPF capacities, this Study Group

recommends that future seismic margin studies continue to concentrate on estimating the HCLPF capacities since all computed HCLPF estimate were judged to be conservative capacity estimates. In this context, the HCLPF capacity estimate is generally considered to be a more useful capacity estimate for seismic margin reviews than is the median capacity, even though there appears to be as much dispersion in HCLPF capacity estimates as there is in median estimates.

7. The HCLPF Study Group does not recommend any revision to the Expert Panel recommendation that either of the two methods (CDFM or FA) could be used in a seismic margin review for calculating the HCLPF capacities.

8. This study has further confirmed that the estimated median capacity is at least twice the HCLPF capacity of a component. Hence there is no "proverbial cliff" in the seismic capacity in that seismic failure is not imminent if the ground acceleration exceeds the HCLPF capacity of the component.

9. The Study Group believes that the classes of components examined herein are representative and are adequate for the purposes of this comparison study. The differences identified in this study are likely to be larger if (a) a larger group of competent engineers were to make the capacity estimates; (b) investigators are required to make building response calculations independently; (c) the definition of what constitutes failure and (d) what constitutes the limiting failure mode are independently assessed or determined by the investigators.

10. The large differences in the HCLPF capacities estimated by the investigators in this study stem from the differences in failure modes assumed, capacity equations associated with any potential failure mode, different assumed as-built material properties, different estimates of inelastic energy absorption, different damping, and different frequency estimates. Even when the same models are adopted, such differences between the analysts are expected to persist because of variations in parameters. Hence a need exists for peer review of seismic margin studies. The Study Group endorses the recommendations of the Expert Panel on Seismic Margins that a peer review be an integral part of the seismic margin study.

11. It is suggested that pretest predictions of failure modes and levels should be made by independent investigators prior to conducting fragility tests. These predictions should be published prior to the tests. The purpose of this recommendation is to improve the capability of engineers to predict component capacity and to decrease the differences between investigators in predicting component capacities.

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APPENDIX A

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HIGH-CONFIDENCE-LOW-PROBABILITY-OF-FAILURE COMPUTATIONS FOR SELECTED COMPONENTS

By

R. P. Kennedy

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1. Introductory Remarks

Both a Conservative-Deterministic-Failure-Margin (CDFM) and a Fragility Method have been suggested in Ref. (1) and (2) for estimating the High-Confidence-Low-Probability-of-Failure (HCLPF) capacity of components subjected to seismic input. The CDFM method is expanded upon in Ref. (3) and is favored by this writer because it directly leads to a HCLPF capacity estimate using straightforward deterministic computations. The alternate Fragility Method first requires an estimate of the median (50% probability of failure) capacity estimate for the component, and this is often difficult to make. Next one must estimate both the uncertainty in the median estimate and the random variability about this estimate. This uncertainty and random variability are often defined in terms of the logarithmic standard deviations, β_U and β_R, respectively. Then the HCLPF capacity is arbitrarily estimated from:

$$\text{HCLPF} = \breve{A}e^{-1.65(\beta_R + \beta_U)} \qquad (1)$$

where $\breve{A}$ is the median capacity estimate. Thus, by the Fragility Method, the HCLPF capacity is highly dependent upon the judgmental and often highly uncertain estimates of β_R and β_U.

The attached set of calculations compares the HCLPF Seismic Margin Earthquake (SME) capacity of 5 components estimated by the writer using both the CDFM method defined in Ref. (3) and the Fragility Method. Consistent assumptions were made in both methods. The 5 components selected were:

1. Flat-Bottom Vertical Water Storage Tank at grade.
2. Auxiliary Contactor Chatter for Motor Starter in an older Motor-Control-Center (MCC) which has a fundamental frequency of about 6.5 Hz. This estimate is made for both the cabinet high in the structure and on the base slab at grade.
3. Diesel Generator Room Starting Air Tank Supports mounted high in the structure.
4. Component Cooling Heat Exchanger Supports mounted high in the structure.
5. Cantilever Reinforced Block Wall mounted high in the structure.

Figures 1 through 4 schematically illustrate the important seismic capacity aspects of the Storage Tank, Starting Air Tank Supports, Heat Exchanger Supports, and Block Wall, respectively. The attached calculations were made as part of a project for four independent consultants to perform HCLPF SME capacity computations on the same 5 components using the simplified methods each would suggest to be used for production type Seismic Margin Review computations. Each consultant agreed to use the same structural properties for each component; these are illustrated in Figures 1 through 4. For the MCC Auxiliary Contactor Chatter cases, it was agreed to assume that no component-specific seismic qualification test data was available, that the 6.5 Hz frequency estimate was approximate, and that the seismic capacity would be based upon the "Function-during GERS (Generic Equipment Ruggedness Spectrum)" presented in Figure 5 as reproduced from Ref. (4).

2. Ground Motion Aspects

For all HCLPF SME capacity computations, it was agreed to assume that the ground motion for the largest horizontal component is given by a uniform hazard spectrum defined at the 84% non-exceedance probability (NEP) at all frequencies. Furthermore, this uniform hazard spectrum shape was to be defined by the NUREG/CR-0098 (Ref. 5) Median spectrum shape for Rock sites.

The 84% NEP vertical response spectrum was then defined to be equal to 2/3 of the 84% NEP largest horizontal response spectrum.

The above ground motion information is sufficient to enable HCLPF SME capacity computations to be made using the Ref. (3) described CDFM method. However, the Fragility Method requires a further description of the random variability of the ground motion response spectrum. Variability associated with peaks and valleys of actual response spectrum and directional response variability should be included. Each consultant using the Fragility Method was expected to make his own estimate of these sources of response spectrum variability. The lognormal parameters used to define random response spectrum variability for this writer's HCLPF computations using the Fragility Method are given in Table 1. The first four parameter values are the basic assumed parameter values. The remaining parameter values have been derived from these four basic parameter values.

The fact that the Fragility Method requires the incorporation of ground motion response spectrum random variability parameters is one of its weaknesses, since it requires the component capacity evaluator to make an assumption on ground motion variability. However, so long as the SME ground motion is defined at the 84% NEP largest horizontal component levels, this is not a major weakness. The resultant HCLPF SME capacity will be rather insensitive to the assumed parameter variabilities. However, if the SME ground motion is defined at the 50% NEP for the average horizontal component, the resultant HCLPF SME capacity is very sensitive to the assumed response spectrum parameter variabilities. For this reason, this writer strongly recommends that 84% NEP response spectrum be used as the basic ground motion definition for HCLPF SME computations by either the CDFM or Fragility Methods. This recommendation removes the sensitivity of the results to the assumed variability of ground motion response spectra and the capacity evaluator is not the right person to be estimating ground motion response spectra variability. The result of using the 84% NEP largest horizontal component response spectrum to determine the HCLPF SME capacity is that this capacity is conditional on this response spectrum not being exceeded at more than 16% of the frequencies in the frequency range and directions that dominate the component capacity.

The above-described ground response spectrum properties are used for computing the HCLPF SME capacity of the ground mounted Flat-Bottom Storage Tank and the ground mounted MCC Auxiliary Contactor Chatter Case. For the other sample components, a floor spectrum is used as discussed in the next section.

3. Floor Spectrum Aspects

It was agreed that the floor spectra shown in Figures 6 and 7 would be used to represent floor spectra obtained high in a structure from a median-centered building response model subjected to 84% NEP ground response spectra anchored to an SME largest horizontal Peak Ground Acceleration (PGA) of 0.18g. Figures 6 and 7 represent horizontal and vertical floor response, respectively. Since these spectra are assumed to represent unbroadened median response to an 84% NEP ground motion input, they may be used directly without scaling in the CDFM Method defined by Ref. (3) or the Fragility Method for an SME PGA of 0.18g. Linear scaling is used for other SME levels.

Actually, in this writer's opinion, the floor spectra shown in Figures 6 and 7 are not realistic median-centered floor spectra. They have too much resonant amplification to be representative of floor spectra from median-centered response analyses even high in a structure. Their use results in a conservative underestimation of the HCLPF SME capacity for components mounted high in the structure. However, so long as all calculations assume those spectra came from a median-centered response analysis, the above-described problem will not influence the relative comparisons between HCLPF capacities obtained by the CDFM Method versus the Fragility Method or comparisons in capacities obtained from the four independent consultants.

4. Results

Table 2 compares the HCLPF SME PGA capacities obtained by the CDFM Method versus the Fragility Method for each of the five sample components. Table 2 also presents this writer's estimate of the median capacity and the summation ($\beta_R + \beta_U$) used in the Fragility Method. Lastly, Table 2 presents the summation ($\beta_R' + \beta_U'$) necessary so that both the CDFM and Fragility methods produce the identical HCLPF capacity for each component and the ratio between those two summations. Several conclusions are reached:

1. For each case, excellent agreement exists between the HCLPF capacities obtained from the CDFM and Fragility methods.

2. The estimated median capacity is at least a factor of 2 greater than the HCLPF capacity in each case.

3. Considering all of the assumptions which must be made in the Fragility Method to estimate both β_R and β_U, it requires only a negligible change in the summation $(\beta_R + \beta_U)$ as denoted by the ratio $(\beta_R' + \beta_U')/(\beta_R + \beta_U)$ to get HCLPF capacity from the Fragility Method to agree with that from the CDFM Method. Such changes are easily within the uncertainty range on β_R and β_U. Since these estimates are uncertain and the HCLPF capacity from the Fragility Method is sensitive to the estimates of β_R and β_U, the CDFM Method provides a more stable estimate of the HCLPF capacity than does the Fragility Method.

5. Interpretation of Results

Each of the capacity estimates presented in Table 2 is intended to represent component failure capacity estimates. However, the definition of component failure can be somewhat judgmental and therefore warrants further discussion for each component.

The flat-bottom vertical water storage tank capacity estimates are intended to correspond to the onset of development of a significant through-wall crack in the tank shell, thus resulting in a loss of tank contents over a period of less than about 24 hours. This crack would be expected to occur either at the weld between the tank side wall and base-plate or within the lower few feet of the tank side wall.

The auxiliary contactor chatter capacity estimates are intended to correspond to the onset of at least 2 milliseconds of auxiliary contactor chatter. The potential consequences of such chatter would depend upon the electrical circuitry involved and is not addressed.

The diesel generator room starting air tank support capacity is intended to correspond to the loss of lateral constraint for at least one of support angles or tie-down bolts. Prior to such loss of lateral constraint, the top of the air tank would be expected to move laterally less than 4 inches, which would be expected to have no adverse consequences on attached air lines so long as they have flexibility. After loss of lateral constraint, the functional performance of the air tank would be uncertain.

The component cooling heat exchanger support capacity is intended to correspond to breaking of anchor bolts on one of the supports, thus allowing the heat exchanger to slide. With such sliding, the functional performance of the heat exchanger would be uncertain.

The cantilever reinforced block wall capacity is intended to correspond to essentially unconstrained lateral deformation of this wall, such that it would come down unless it hit some support and was held up by that support prior to falling. Prior to reaching this condition, deformations of the top of the wall will be less than 10 inches, so that the Table 2 capacity estimates are judged appropriate unless a component which can be damaged by wall impact exists within 10 inches of the wall.

Of course, capacities of components are influenced by characteristics of ground motion not defined by the response spectral amplitudes. Most important of these are the duration of strong ground motion and the number of strong nonlinear response cycles to which the component is subjected by the ground motion. Since neither were defined, for this exercise I have assumed that the duration of strong ground motion as defined in accordance with Ref. (6) is 3 to 10 seconds, subjecting each component to 3 strong nonlinear response cycles also as defined by Ref. (6). In my judgment, such ground motion characteristics correspond to the ground motion levels listed in Table 2 having come from an earthquake with a local magnitude of about 6.5. For ground motions from earthquakes with local magnitudes less than about 6.0, the capacities given in Table 2 are too conservative. Conversely, for earthquakes with local magnitudes exceeding about 7.0, these capacities may be too liberal.

Several capacities were checked for each component. Only calculations for the controlling capacity for each component are shown in the attached calculation package.

6. Reconciliation of Results

Each of the four consultants who participated in this project independently produced his calculations prior to October 1987. During November 1987, these consultants met to compare results. After that meeting, each consultant was allowed to revise his calculations to see whether such revised calculations would come closer together. All the attached calculations and summary results in Table 2 are from the independently produced work prior to October 1987. This section contains my discussion on reconciliation of my results with those from the other consultants (Reed, Stevenson, and EQE).

6.1 Flat Bottom Tank

Reed and EQE have HCLPF estimates of 0.27 and 0.26g, respectively, which are very close to my estimate of 0.29g, whereas Stevenson has a HCLPF estimate much higher than our three estimates. Similarly, Reed and EQE have median estimates of 0.53 and 0.54g, compared to my estimate of 0.67g, whereas Stevenson has a median estimate much higher than mine. Thus, one might think that it might be easy for me to reconcile on an estimate between that of Reed, EQE, and myself for both the HCLPF and median capacities, with Stevenson being an outlier. However, this is not the case.

As discussed in Ref. (7), a copy of which is included in the attached calculations, my capacity estimates are conservatively biased in two areas. First, I have ignored any benefit from inelastic energy absorption. Second, I have conservatively underestimated the benefits from fluid-holddown forces on the base plate. These conservatisms were intentionally introduced, because I believed that producing defensible estimates of the inelastic energy absorption capacity increase and/or less conservative estimates of the benefits of fluid-holddown would require more sophisticated and much more costly analyses than I considered to be warranted. Therefore, I chose to ignore these additional capacity enhancement benefits. However, these additional factors are real, and ignoring them results in a conservative bias to my calculations.

Based on approximate computations (not attached), I estimate that inclusion of a realistic (but difficult to defend without more sophisticated analyses) estimate of the inelastic energy absorption capacity increase factor would increase my HCLPF capacity by a factor of 1.25 to 0.36g, and my median capacity by a factor of 1.75 to 1.17g. Furthermore, a nonlinear, large-deflection theory base plate uplift analysis to better account for fluid-holddown effects would increase these capacities by an unknown further amount.

Based on these considerations, it is impossible for me to compromise with Reed or EQE on any HCLPF capacity estimate less than 0.29g or median estimate less than 0.67g. On the other hand, it would be very easy for me to reach reconciliation with Stevenson on any HCLPF capacity between 0.29g and 0.40g and any median capacity between 0.67g and 1.3g.

6.2 Auxiliary Contact Chatter

When component-specific fragility or qualification test data is unavailable, Ref. (3) suggests that the Generic Equipment Ruggedness Spectra (GERS) from Ref. (4) be used to make HCLPF capacity estimates. However, when this is done, I have recommended in Ref. (3) that the GERS be divided by 1.3 before being compared with Required Response Spectra (RRS) being input at the base of the component. This recommended reduction factor of 1.3 was based upon two considerations:

1. The GERS were based on only a limited amount of test data and cover broad generic component categories. As such, they do not represent HCLPF capacity estimates. Based upon material presented in Ref. (4) plus material presented to the Senior Seismic Review and Advisory Panel (SSRAP), I have judged that the typical ratio between GERS and HCLPF capacities is about 1.2 in the case of a broad-frequency input RRS.

2. It is my judgment that an RRS developed following the recommendation of Ref. (3) for HCLPF computations lies in the 84% to 90% non-exceedance probability range. As such, additional conservatism of about a factor of 1.1 is necessary to account for variability in the input spectrum when GERS are compared to such RRS to achieve an overall HCLPF capacity estimate.

For cases where chatter of auxiliary contacts in motor control centers is of concern, Ref. (4) has suggested that the "Function During" GERS shown in Figure 5 be factored by 0.87. No basis is given for this recommendation, nor was any basis ever discussed with the SSRAP. I believe this factor is simply an additional conservatism factor in recognition that the GERS do not represent the HCLPF capacity. Since that consideration is already covered by the 1.3 reduction factor recommended in Ref. (3) for HCLPF capacity computations, I do not believe that both the 0.87 factor and the 1.3 factor should be combined as was done by EQE when computing the HCLPF capacity by the CDFM method of Ref. (3). If this 0.87 factor was removed from the EQE HCLPF capacity computations by the CDFM method, their CDFM HCLPF capacities would be identical to mine.

My judgment that the 0.87 factor from Ref. (4) and the 1.3 factor from Ref. (3) should not both be included, is supported by the fragility and qualification test data presented in Ref. (8) for auxiliary contact chatter in motor control centers. Ref. (8) presents results from 51 fragility tests in which auxiliary contacts chattered and 10 qualification tests in which no auxiliary contact chatter occurred. For broad frequency input, the 2% damped spectral acceleration fragility levels ranged from 2.1g to 7.4g for the 51 fragility tests with auxiliary contact chatter, while the 10 qualification tests without auxiliary contact chatter ranged from 2.0g to 5.6g. Assuming the data fits a lognormal distribution, Ref. (8) reports a 2% damped spectral acceleration HCLPF capacity of 1.7g. However, the data does not fit a lognormal distribution; and making the erroneous assumption of a lognormal distribution results in the highest fragility test capacity of 7.4g and the highest qualification test capacity of 5.6g, actually driving down the HCLPF capacity estimate because these high test results increase the estimates of logarithmic standard deviation. If instead one performs a distribution-free one-sided tolerance limit check (which is generally very conservative because it is a distribution-free check which makes no assumptions on the data distribution) on the data, one determines a 2% damped spectral acceleration HCLPF (95% confidence of less than 5% failure probability) capacity of 2.1g. Since the GERS of Ref. (4) are based upon 5% damped spectra, this 2% damped HCLPF capacity of 2.1g must be converted to a 5% damped HCLPF capacity before being compared with the GERS "Function During" level of 1.5g shown in Figure 5. To convert from 2% damped to 5% damped spectra, the 2% damped HCLPF capacity must be divided by about 1.5, which leads to a 5% damped HCLPF capacity of 1.4g. Thus, the ratio of the GERS capacity from Ref. (4) to the HCLPF capacity from the data of Ref. (8) is 1.5/1.4 = 1.07, which is less than the 1.2 ratio of GERS to HCLPF upon which the 1.3 reduction factor recommended in Ref. (3) is based. Thus, it would be very inappropriate to both multiply the GERS by 0.87 and also divide by 1.3 to obtain a HCLPF capacity for auxiliary contact chatter. In fact, the 1.3 reduction factor by itself appears to be too high for this case.

For the case of the motor control center cabinet at grade, the HCLPF estimates from the four consultants range from a low value of 0.39g by EQE to a high value of 0.71g by Stevenson, with my estimate of 0.54g being about midway. Because of the above-discussed considerations, I consider my HCLPF estimate to be somewhat conservatively biased. For this reason, I could accept a consensus HCLPF estimate ranging between 0.54g and 0.64g. Therefore, it is likely to be easier for me to reach a compromise HCLPF estimate with Stevenson than it is with the low HCLPF estimates of EQE.

For the motor control center cabinet high in the structure, the HCLPF estimates range from a low value of 0.07g by EQE to 0.12g by Stevenson, with my estimate of 0.10g being about midway. For the reasons discussed above, my estimate might be slightly conservative so that I could concur with a compromise HCLPF ranging between 0.10g and 0.12g. However, an estimate as low as 0.07g by EQE is totally unacceptable to me.

6.3 Starting Air Tank

For the Starting Air Tank, the HCLPF estimates for the four consultants ranged from 0.39g by Stevenson to 0.53g by EQE using the CDFM, with my estimate of 0.48g being midway. For the median capacity, the estimates of Reed, Stevenson, and myself were tightly bunched from 1.01g to 1.07g, with the median estimate from EQE being much higher at 1.55g.

An important difference in the capacity computations is that neither Stevenson nor I took any credit for an inelastic energy absorption factor increase. Both EQE and Reed did take credit for such an increase using procedures which are really only appropriate when the input consists of broad frequency ground spectra with 5% damped amplified spectral accelerations 2 to 3 times the zero period acceleration. The floor spectrum shown in Figure 6 which is input to this component certainly does not fit into this category. Its frequency content is very narrow, centered at about 7.5 Hz, and the 5% spectral accelerations are amplified by more than a factor of 5. Using an inappropriate approach in my judgment, EQE increased their median capacity by an inelastic energy absorption factor of 2.08 and their CDFM HCLPF capacity by 1.25. Reed increased his median capacity by a much more moderate factor of 1.35 and made essentially no increase in his HCLPF capacity for inelastic energy absorption.

By my calculations, the natural frequency of this starting air tank is about 16 Hz. Considering both the uncertainty at which the floor spectra peaks occur and the uncertainty in the component natural frequency, I recommend that this frequency be shifted down to about 12.8 Hz when entering the unbroadened floor spectrum of Figure 6. The nonlinear analyses of Ref. (6) clearly demonstrate that the shape of the input spectrum to the lower frequency side of the elastic frequency plays a very dominant role on the inelastic energy absorption increase factor as the structure softens due to ductility. In Figure 6, as the component frequency shifts downward from 12.8 Hz, the spectral accelerations rapidly increase. With such a rapid increase in spectral accelerations as the frequency is reduced due to inelasticity, there is essentially no benefit from inelastic behavior since such behavior forces the component right into the power of the input. Unless one performs a series of nonlinear time history analyses, I would not recommend taking any credit for an inelastic energy absorption increase factor for this case even though the failure mode is very ductile. Even with nonlinear time history analyses, it is difficult for me to conceive that this factor could exceed 1.35 for the median capacity or 1.1 for the HCLPF capacity with such a spiked spectrum as shown in Figure 6 lying immediately below the component natural frequency. For this reason, I believe that the median inelastic energy absorption factor of 2.08 used by EQE to get their high median capacity to be totally unrealistic.

For the starting air tank, I can accept a consensus HCLPF capacity anywhere in the range from 0.45g to 0.53g and a median capacity in the range from 1.0g to 1.35g.

6.4 Heat Exchanger

For the heat exchanger, the HCLPF capacity estimates ranged from a low of 0.30 by Stevenson to a high of 0.44 by EQE using the CDFM method, with my estimate of 0.40g being between. The differences result from a number of factors, and no clear cause is apparent to me. I can accept a consensus HCLPF capacity for this heat exchanger anywhere in the range from 0.37g to 0.44g, which covers all but Stevenson's existing estimate.

6.5 Block Wall

My HCLPF capacity estimate for the block wall is 0.62g. The other three consultants had lower HCLPF capacities ranging from a low of 0.38g by Reed to 0.51g by Stevenson. My estimate of the median capacity of 1.94g is similarly high compared to the range of 1.34g to 1.55g estimated by the other consultants. The difference between my results and those of the other consultants primarily lies in my treatment of the inelastic response of this block wall.

Since the block wall is reinforced, its behavior prior to failure will be highly ductile. Therefore, a realistic computation of its capacity requires a realistic assessment of its inelastic behavior. This wall is subjected to the very narrow frequency, highly amplified input spectrum shown in Figure 6. In my judgment, simplified approaches based on broad frequency, less amplified ground spectra, cannot be used to realistically estimate the inelastic response of highly ductile components subjected to a narrow frequency input spectrum such as Figure 6. Based on a large number of nonlinear time-history analyses, Ref. (6) recommends an approach for estimating inelastic response which is appropriate for narrow frequency input spectra such as that in Figure 6. This approach requires the estimation of an effective frequency and effective damping. The spectral acceleration at this effective frequency and damping are then used to estimate inelastic response rather than the spectral acceleration at the elastic frequency and damping. I have used the approach of Ref. (6) to estimate the inelastic response of this block wall. This approach leads to much greater capacity estimates for this ductile wall subjected to the floor spectrum of Figure 6 due to inelastic response than would be obtained using ductility factor correction approaches which are appropriate for broader frequency content ground spectra.

Accounting for uncertainty in the natural frequency of this block wall and uncertainty in the frequency content of the Figure 6 input spectrum, I would conservatively estimate the "so-called elastic" response of this wall using the 5.5 Hz, 7% damped spectral acceleration from Figure 6 of 0.87g. This spectral acceleration value lies high up on the narrow frequency amplified spectrum of Figure 6, and softening of the component due to inelastic behavior will dramatically reduce the input spectral acceleration. In such a case there will be very large benefits from inelastic response. Following the recommendations of Ref. (6), even for the HCLPF capacity I can conservatively estimate that the effective frequency of this wall will not exceed 2.9 Hz with an effective damping of 10%. Using these values, the spectral acceleration from Figure 6 is 0.33g, which is only 38% of the "elastic" spectral acceleration of 0.87g. Conservatively limiting the inelastic energy absorption correction factor to 1.25 as recommended by Ref. (3) rather than using the higher value developed by the procedures of Ref. (6), I obtained an effective spectral acceleration for HCLPF capacity of 0.33/1.25 = 0.26g. In my judgment, this effective spectral acceleration of 0.26g very conservatively represents the inelastic response of this block wall prior to failure when subjected to the input of Figure 6. For comparison, the effective spectral acceleration used by EQE in their CDFM HCLPF capacity computations was 0.36g. This difference of 0.36g versus 0.26g for the effective spectral acceleration more than accounts for the differences in our HCLPF capacities of 0.48g versus 0.62g. Similarly, at the median level, the differences in effective spectral accelerations used fully accounts for the resultant differences in median capacities.

Because I strongly believe that I have used a more realistic approach based on Ref. (6) to estimate the inelastic response of this block wall, I cannot support any consensus HCLPF or median capacity estimate significantly less than those I provided in Table 2, even though my estimates are higher than those of any of the other consultants. I could not support a consensus HCLPF capacity estimate less than 0.57g or a median capacity estimate less than 1.75g for this block wall.

7. References

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TABLE 1

GROUND MOTION RANDOM VARIABILITY RESPONSE PARAMETERS

Parameter	Median	β_R	Comments
(1) F_1: Peak & Valley Variability Factor	1.00	0.20	Basic Assumed Parameter Values
(2) F_2: Horizontal Direction Variability Factor	1.00	0.15	
(3) 84% NEP Vertical / 84% NEP Largest Horizontal	0.67	----	
(4) Vertical / Median Average Horizontal	----	0.34	
			Derived Parameter Values
(5) Largest Horizontal / Median Average Horizontal	1.13	0.22	From (1) & (2)
(6) F_6: 84% NEP Largest Horizontal / Median Average Horizontal	1.41	----	From (5)
(7) F_7: 84% NEP Largest Horizontal / Median Largest Horizontal	1.22	----	From (1)
(8) F_8: Vertical / Median Average Horizontal	0.67	0.34	From (3), (4), & (6)
(9) F_9: Vertical / 84% NEP Largest Horizontal	0.48	0.34	From (6) & (8)

TABLE 2

COMPARISON OF CAPACITY RESULTS FROM CDFM AND FRAGILITY METHODS

Component	CDFM	Fragility Method			Equal HCLPF Capacities	
	HCLPF SME	HCLPF SME	Median SME	$(\beta_R + \beta_U)$	$(\beta_R' + \beta_U')$	$\frac{(\beta_R' + \beta_U')}{(\beta_R + \beta_U)}$
Flat Bottom Storage Tank	0.29g	0.31g	0.67g	0.47	0.51	1.08
Auxiliary Contactor Chatter						
(a) Cabinet At Grade	0.54g	0.59g	1.26g	0.46	0.51	1.12
(b) Cabinet High in Structure	0.10g	0.11g	0.30g	0.60	0.67	1.11
Starting Air Tank	0.48g	0.50g	1.07g	0.46	0.49	1.06
Heat Exchanger	0.40g	0.42g	1.18g	0.62	0.66	1.06
Block Wall	0.62g	0.67g	1.94g	0.64	0.69	1.08

Materials: Shell & Chairs: SA240 - Type 304 SST
Bolts: A307 - 2" ϕ

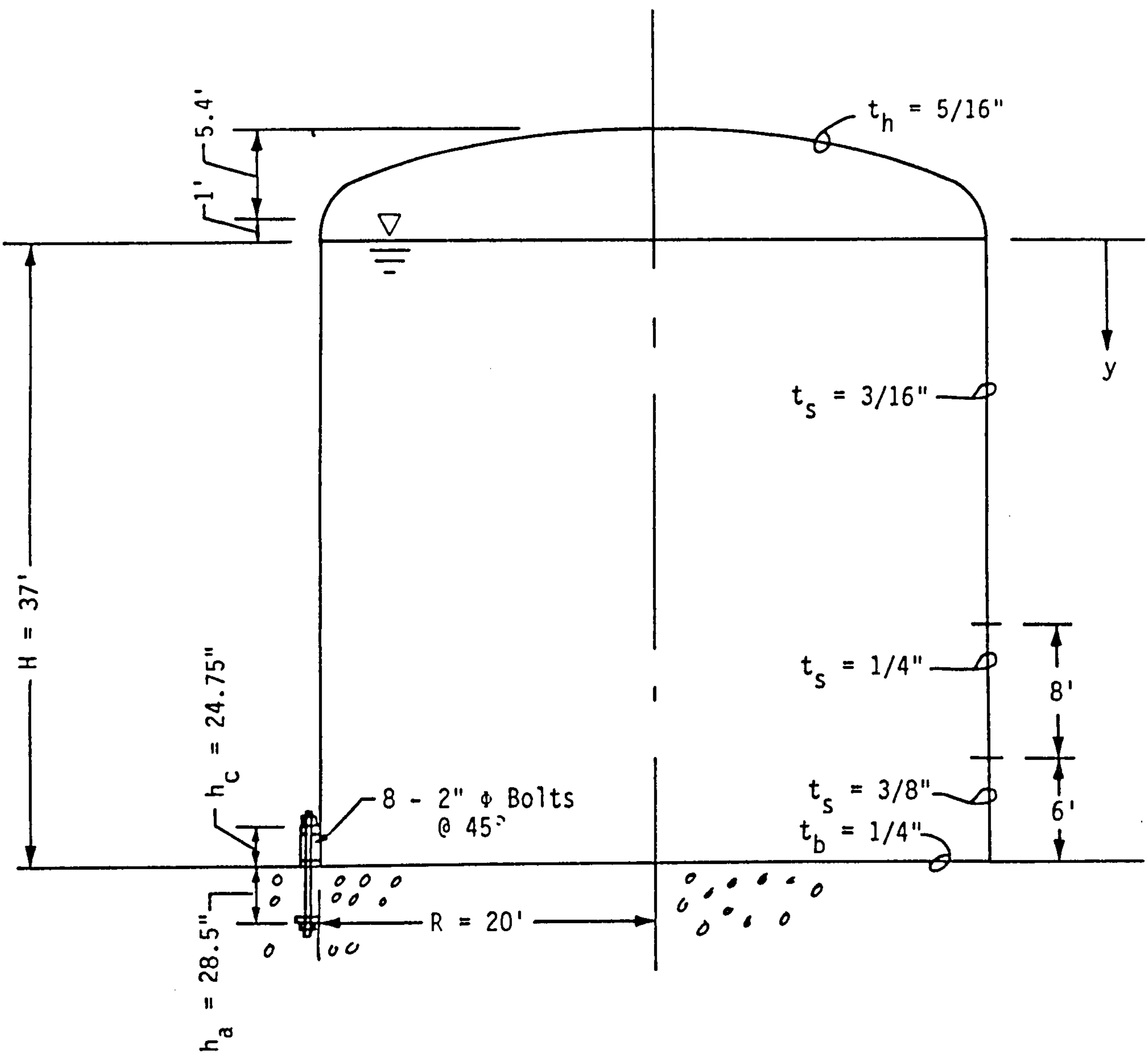

Figure 1. Flat Bottom Vertical Water Storage Tank

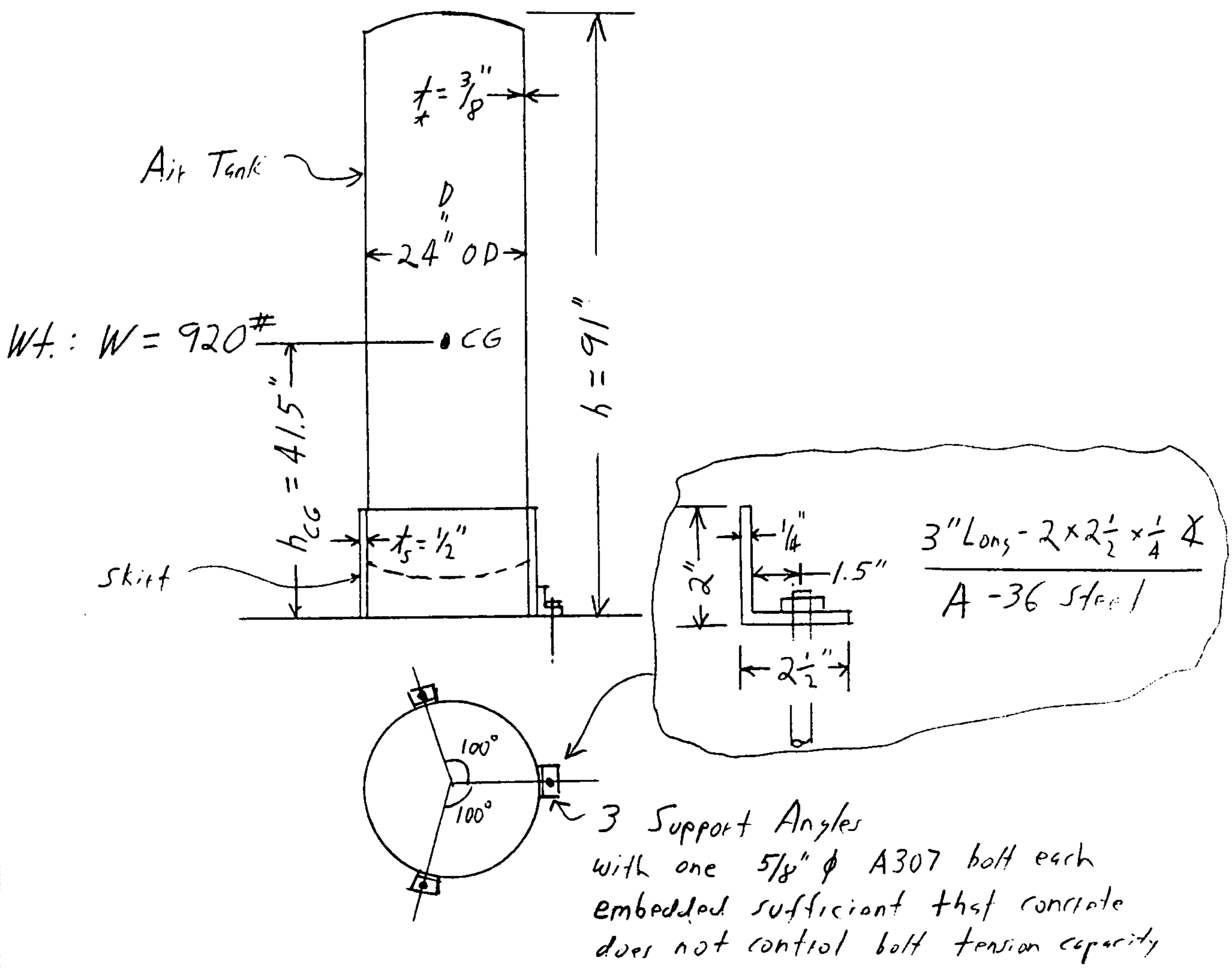

Fig 2: Diesel Generator Room Starting Air Tank Supports

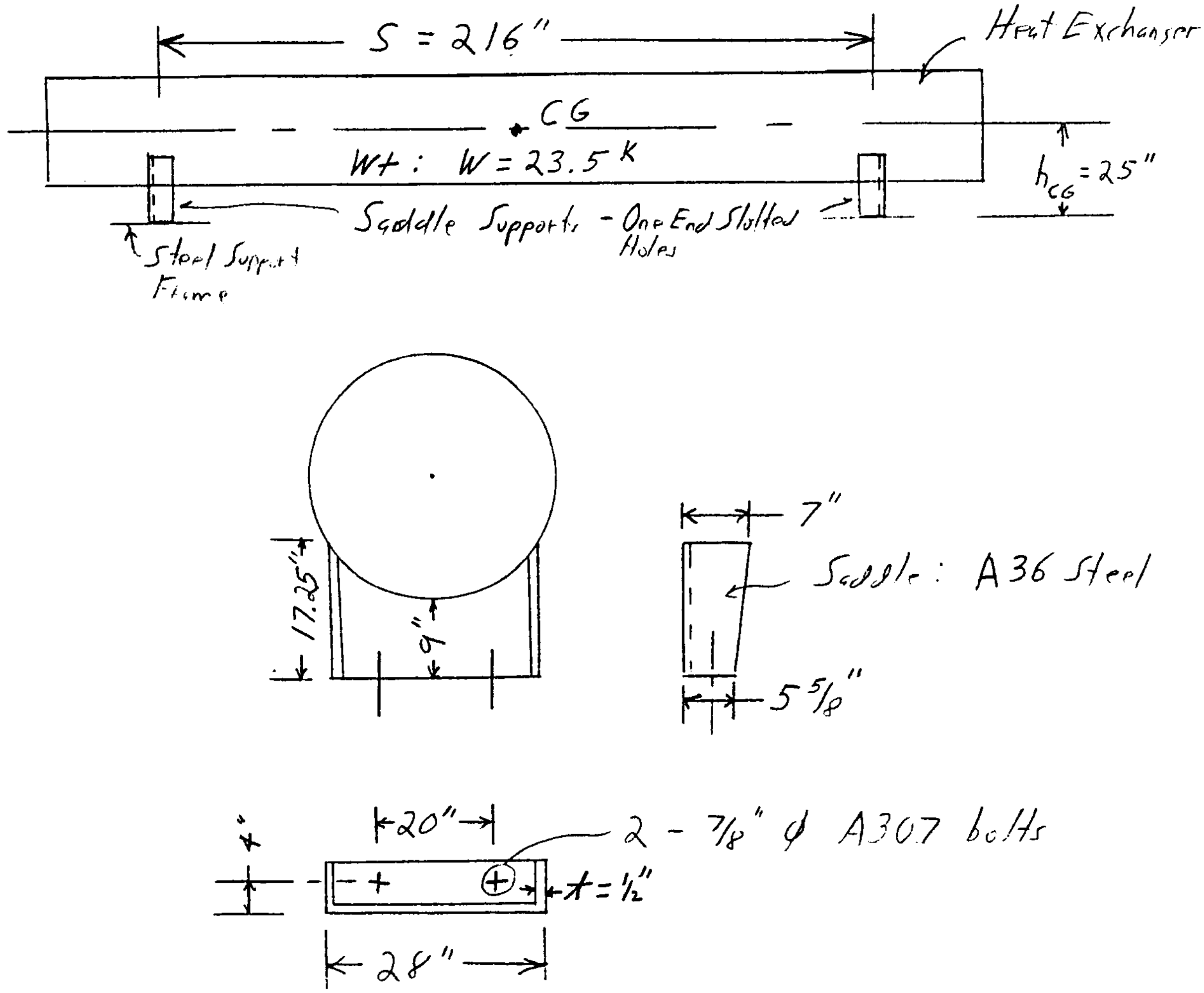

Fig 3: Component Cooling Heat Exchanger Supports

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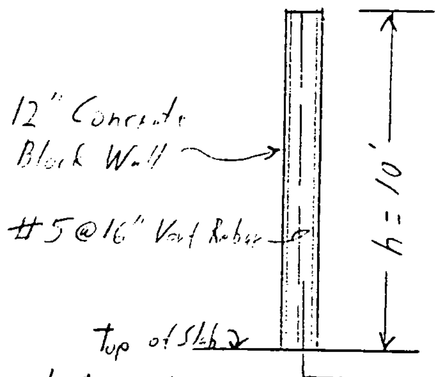
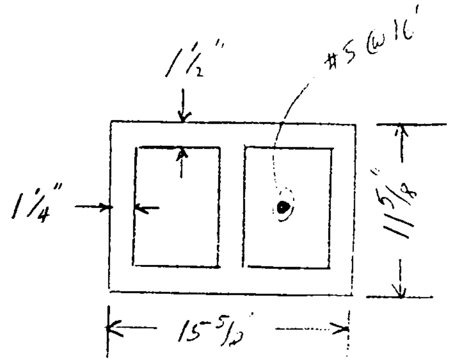




Special Inspection Requirements are Met.

Wall is constructed in running bond, cells w/ rebar grouted solid

Block: 12" hollow units, normal weight, ASTM C90 Grade N, $f'_c = 3000$ psi

Mortar: ASTM C270 Type S

Vert. Rebar: #5 @ 16 o.c., Grade 60

Horiz. Rebar: Extra Heavy, 3 rod Durowal @ 16' o.c.

Wall long compared to height. ∴ Acts as vertical cantilever

Wall Wt: $w = 111$ psf

Fig 4: Cantilever Reinforced Block Wall

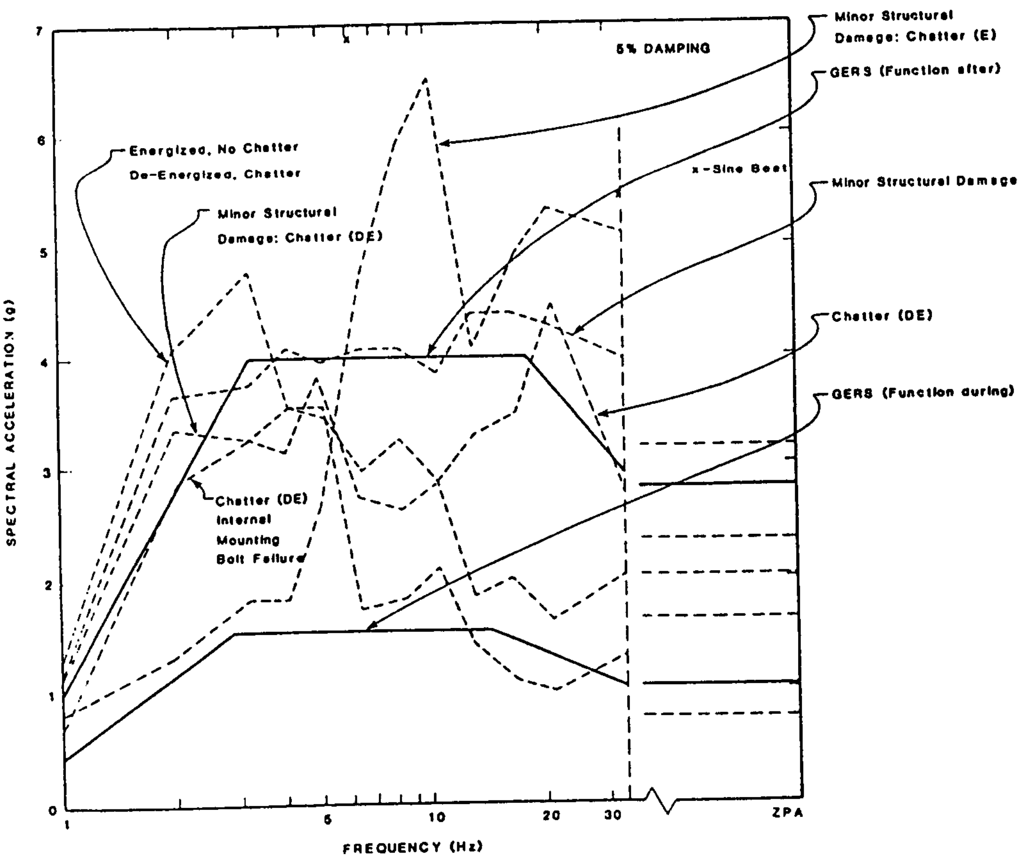
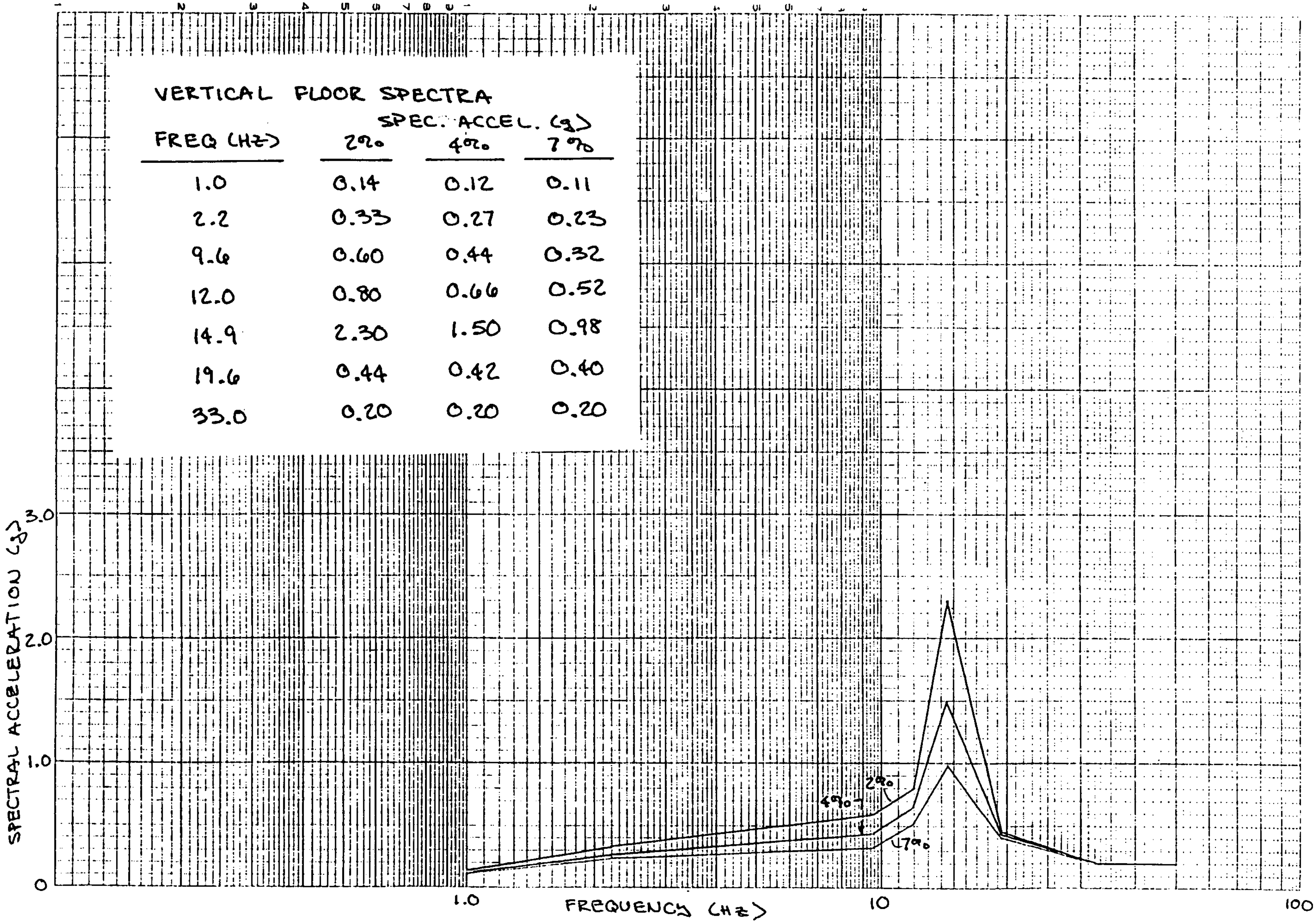


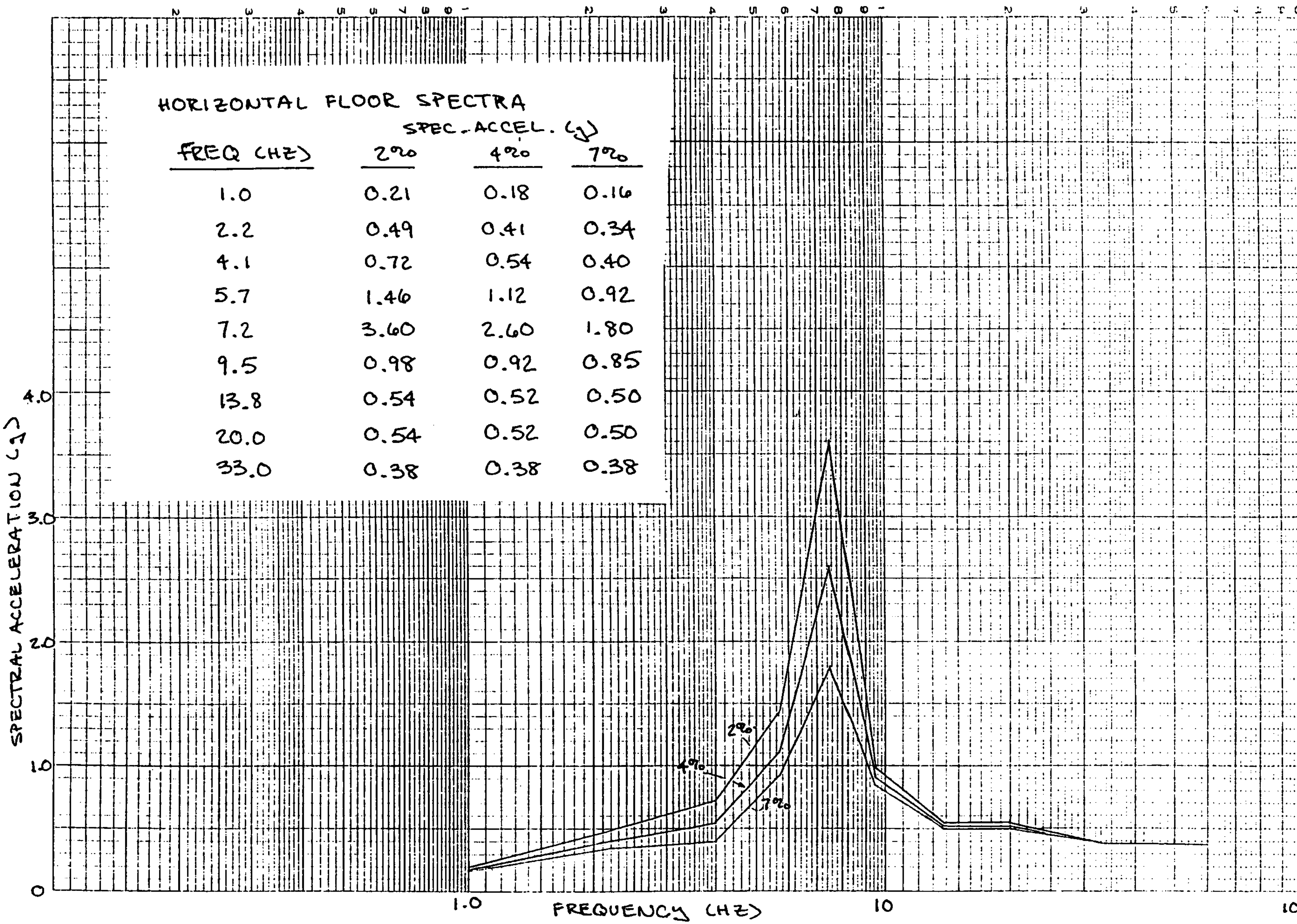
Figure 5. Comparison of GERS with failure data: function during and after for MCC (from Ref. 4).



VERTICAL FLOOR SPECTRA

FREQ (HZ)	SPEC. ACCEL. (g) 2%	4%	7%
1.0	0.14	0.12	0.11
2.2	0.33	0.27	0.23
9.6	0.60	0.44	0.32
12.0	0.80	0.66	0.52
14.9	2.30	1.50	0.98
19.6	0.44	0.42	0.40
33.0	0.20	0.20	0.20

Figure 7. Vertical Floor Spectra



HORIZONTAL FLOOR SPECTRA

FREQ (HZ)	SPEC. ACCEL. (g) 2%	4%	7%
1.0	0.21	0.18	0.16
2.2	0.49	0.41	0.34
4.1	0.72	0.54	0.40
5.7	1.46	1.12	0.92
7.2	3.60	2.60	1.80
9.5	0.98	0.92	0.85
13.8	0.54	0.52	0.50
20.0	0.54	0.52	0.50
33.0	0.38	0.38	0.38

Figure 6. Horizontal Floor Spectra

FLAT-BOTTOM VERTICAL WATER STORAGE TANK

CDFM METHOD*

By

R. P. Kennedy

* Reproduced from Appendix H, Ref. (3), "Flat-Bottom Vertical Water Storage Tank" by R. P. Kennedy, August 1987.

Appendix H
FLAT-BOTTOM VERTICAL FLUID STORAGE TANKS
by
R.P. Kennedy

H.1 Introduction

Flat-bottom vertical fluid storage tanks of the type illustrated in Figure H-1 should be evaluated during a Seismic Margin Assessment (SMA) if they are needed in the success paths being considered. First, examples of failures with loss of contents during strong earthquake shaking exist for such tanks when they have minimal or no anchorage. Secondly, even though such tanks at nuclear power plants have been designed for seismic effects, past designs (those predating about 1977) tend to have less seismic margin than exists for most other seismic designed components.

Many of these tanks were designed using the seismic evaluation procedure of TID-7024 (1). The major problem is that direct application of this method is consistent with the assumption that the combined fluid-tank system in the horizontal impulsive mode is sufficiently rigid to justify the assumption of a rigid tank. For the case of flat bottomed tanks mounted directly on their base, or tanks with very stiff skirt supports, this assumption leads to the usage of a spectral acceleration equal to the zero-period base acceleration. This assumption is unconservative for tanks mounted on the ground or low in structures when the spectral acceleration does not return to the zero period base acceleration at frequencies below about 20 Hz. More recent evaluation techniques (2)(3)(4)(5) have shown that for typical tank designs, the modal frequency for this fundamental horizontal impulsive mode of the tank shell and contained fluid is generally between 2 and 20 Hz. Within this regime, the spectral acceleration is typically significantly greater than the zero period acceleration.

The above described problem with such tanks in the nuclear industry was publicized during the U.S. Nuclear Regulatory Commission Task Action Plan A-40 (2) which provided the basis for Unresolved Safety Issue A-40. One way to resolve this issue is through showing an adequate seismic margin of such tanks by a seismic evaluation as a part of a SMA. This appendix is intended to summarize a reasonable procedure to evaluate the High-Confidence-Low-Probability-of-Failure

(HCLPF) seismic capacity of such tanks following the general Conservative-Deterministic-Failure-Margin (CDFM) approach summarized in Section 2.

The seismic evaluation of these tanks consists of two parts: a seismic response evaluation, and a seismic capacity assessment. The topic of response evaluation has been extensively described in the literature in the last 10 years ((2) through (10)). Therefore, response evaluation will only be summarized herein as it applies to the example tank, illustrated in Figure H-1. The general approach followed will be that given in (2)(5)(10) and the reader is referred to any of these readily available references for further details. Herein, it will be assumed that the example tank is founded on a rock site so that the topic of soil-structure-interaction (SSI) is not germane to this example. When flat-bottomed tanks are founded on soil, the resultant SSI can substantially modify the tank response (5) and such modifications should be considered.

The seismic capacity assessment of minimally anchored tanks such as that illustrated in Figure H-1 has not received as thorough of a treatment in the literature and so will be discussed herein in greater detail. The example tank shown in Figure H-1 will be used as the vehicle for this discussion.

The example tank is representative of tanks often found at low seismic (SSE<0.15g) nuclear power plant sites. The tank radius, R, is 20 feet, while the water height, H, when full, is 37 feet, with the overall tank height to the top of its dome roof being 43.4 feet. Thus, this tank is about twice as high as its radius. Because this tank was designed for a low SSE using the TID-7024 (1) response evaluation approach, this tank was built with only a minimal number of hold-down anchor bolts consisting of eight 2 inch diameter A307 bolts around its circumference. These bolts provide hold-down forces to the tank shell through the top plate of well-designed bolt chairs at a height, h_c, of 24.75 inches above the tank bottom. The bolts are anchored into the concrete foundation through an anchor plate at a depth, h_a, of 28.5 inches. The bolt chairs, their attachment to the tank, and the bolt anchorage are sufficient to develop the full capacity of the bolts. Because this is an atmospheric storage tank (no internal pressure) with a low design SSE, the tank head, side wall, and base plate thicknesses are thin which is typical for these tanks. The tanks shell is SA240-Type 304 stainless steel.

Although this tank had an unconservative TID-7024 (1) seismic response analysis for a low SSE during design, the design capacity assessment was performed very conservatively as is typical of nuclear plant designs and the detailing was good. Therefore, simply from a review of design calculations and a walkdown it is impossible for experienced seismic engineers to assess whether this tank has a sufficiently high seismic margin (HCLPF > 0.30g) such that an evaluation is unnecessary or such a low seismic margin (HCLPF < 0.15g) so that the seismic capability of this tank should be dismissed and an alternate success path selected. A detailed seismic evaluation is warranted.

As noted in Section 2, the first step in a SMA is the selection of a Seismic Margin Earthquake (SME) response spectrum shape anchored to a SME Peak Ground Acceleration (PGA) level. For this example problem, the fourth alternative approach described in Step 1 of Section 2 will be used for this selection. In this alternative, a standard medium spectrum shape which approximates a uniform hazard spectrum shape will simply be specified. For this example, the median NUREG/CR-0098(11) spectrum shape for rock sites will be specified for the horizontal ground motion and the vertical ground motion will be specified as two-thirds of the horizontal motion. When specified in this way, the resultant HCLPF SME statement is conditional on this standard spectrum anchored to the SME level PGA not being exceeded by a future ground motion at more than 16% of the natural frequencies in the frequency range and direction of interest. The resultant SME PGA level will then be determined by the HCLPF seismic evaluation of the tank.

Because capacities to withstand horizontal responses are slightly influenced by vertical responses, a small amount of nonlinearity develops when computing the SME capacity of tanks. Therefore, it is preferable to estimate an SME capacity of the tank, SME_e, and to compute the seismic response, $SEISMIC_e$, for this SME_e. Then the actual SME capacity can be estimated from:

$$SME = \frac{CAPACITY - STATIC}{k \bullet SEISMIC_e} (SME_e) \qquad \text{(H-1)}$$

where CAPACITY is the HCLPF capacity of the tank, STATIC is the portion of this capacity used up by static loads and k is the inelastic energy absorption effective seismic stress correction factor described in Section 2 and discussed in the capacity assessment section of this appendix. If the resultant SME differs substantially from SME_e then iteration of the procedure is necessary because of the slight nonlinearities. So long as SME and SME_e are close, no iteration is necessary. For this example tank, the estimated SME_e will be taken to be 0.27-g. Thus, using (11) for rock sites, the ground motion estimates to be used for the response evaluation will be:

Horizontal PGA: $A_H = SME_e = 0.27g$

Vertical PGA: $A_V = (2/3)A_H = 0.18g$

Horizontal Velocity: $V_H = 36$ in/sec-g$(A_H) = 9.7$ in/sec

Vertical Velocity: $V_V = (2/3)V_H = 6.5$ in/sec

Horizontal Displacement: $D_H = (6V^2{}_H/A_H) = 5.4$ inches

Using the response evaluation and capacity assessment procedures recommended herein together with the above definition for the SME response spectrum shape, the example tank defined by Figure H.1 will be shown to have a HCLPF SME PGA capacity of 0.29g. It should be noted that the HCLPF capacity level is a conservative, essentially lower-bound estimate of the failure capacity. The actual failure capacity is likely to be much higher. In fact, a median failure capacity estimate (50% probability of failure) for this tank is in excess of 0.6g. Therefore, despite this tank having an unconservative TID-7024 (1) response analysis for a low SSE during design, this tank has a substantial seismic margin capability.

H.2 Response Evaluation

The seismic response evaluation should provide estimates of each of the following:

1. The overturning moment in the tank shell immediately above the base plate of the tank. This moment is then compared with base moment capacity as governed by a combination of shell buckling and anchor bolt yielding or failure and generally governs the SME capacity of the tank.

2. The overturning moment applied to the tank foundation through a combination of the tank shell and the base plate. This moment is only needed for tanks founded on soil sites where a foundation failure mode should be investigated and is generally obtained as part of the SSI evaluation. It seldom governs the SME capacity.

3. The base shear beneath the tank base plate. This base shear is compared to the horizontal sliding capacity of the tank. For atmospheric tanks with a radius greater than 15 feet it seldom controls the SME capacity.

4. The combination of the hydrostatic plus hydrodynamic pressures on the tank side wall. It is common design practice to compare these combined

pressures with the membrane hoop capacity of the tank side walls at one-foot above the base and each wall thickness change. Thus, for the example tank these combined pressures are needed at 22-feet, 30-feet, and 36-feet below the top of the water. These combined pressures essentially never govern the SME capacity of a properly designed tank.

5. The average hydrostatic minus hydrodynamic pressure on the base plate of the tank. This pressure is used when evaluating the sliding capacity of the tank. If hold-down forces due to fluid on the base plate are included in the overturning moment capacity estimate, then the minimum value of the hydrostatic minus hydrodynamic pressure near the tank side wall should also be estimated.

6. The fluid slosh height. This slosh height is compared with the freeboard above the top of the fluid to estimate whether roof damage is likely. However, roof damage seldom interferes with the safety function of the tank immediately after an earthquake and is generally not of concern in a SMA.

In estimating each of these response quantities at least two horizontal modes of combined fluid-tank vibration and one vertical mode of fluid vibration should be considered. The two horizontal response modes should include at least one impulsive mode in which the response of the tank shell and roof are coupled together with the portion of the fluid contents which move in unison with the shell and at least the fundamental sloshing (convective) mode of the fluid. As noted previously, the response evaluation will only be summarized herein as it applies to the example tank and will generally follow the approach given in (2)(5)(10).

The first step of a response evaluation is to make a weight takeoff and to determine the hydrostatic fluid pressure, P_{ST}, at capacity evaluation locations along the tankshell and the base. For the example tank, the component weights, W, and their center of gravity heights, X, above the tank base are:

Head:	W_H = 17.2 kips	X_H = 42-ft
Shell:	W_S = 44.9 kips	X_S = 16.4-ft
Bottom:	W_B = 12.8 kips	X_B = 0
Water:	W_W = 2900 kips	X_W = 18.5-ft

It should be noted that the water weight totally dominates over the tank weight being nearly 40 times the total tank weight. As a first order estimate, one could base the entire computed seismic response on the water weight ignoring tank weights. This approach will not be done herein, but tank weight responses will be only approximately computed.

The hydrostatic fluid pressures, P_{ST}, at capacity evaluation locations are given in Table H-1.

H.2.1 Horizontal Impulsive Mode Response

One must first estimate the horizontal impulsive mode natural frequency, f_I. One approach is to use:

$$f_I = \frac{C_{LI}}{2\pi H} \sqrt{E_S / \rho_S} \quad \text{where} \quad C_{LI} = C_{WI} \sqrt{\frac{0.127\rho_S}{\rho_L}} \qquad \text{(H-2)}$$

where E_s is the modulus of elasticity for the tank wall material, $\rho_S$is its mass density, $\rho_L$is the fluid mass density, and C_{WI} is a horizontal impulsive frequency coefficient for water in a steel tank. This coefficient is a function of the ratio (H/R) and the tank wall thickness, t. For the case of roofless tanks with a uniform wall thickness, t, the value of C_{WI} may be obtained from (5) or (8). For variable wall thickness tanks, the variable wall thickness can be approximated by an average value wherein the averaging should be done so as to emphasize the section of the tank for which modal displacements are the largest. For water in steel tanks $(\rho_L/\rho_S) = 0.127$so that $C_{LI} = C_{WI}$ and$\sqrt{E_S/\rho_S} \approx 16{,}200$ ft./sec.

For the example tank, the approximate average thickness is estimated to be 0.22-inch and (H/R) = 1.85. Using Table 7.4 of (5) and these properties, C_{WI} is estimated at 0.085. Thus, f_I is 6.0 Hz with an expected accuracy range of about 5.5 Hz to 6.6 Hz.

Next, one must estimate the horizontal impulsive mode spectral acceleration, S_{A_I}, for frequencies in the range of 5.5 to 6.6 Hz. For such an estimate, one needs an estimate of energy dissipation as expressed by equivalent viscous damping. For CDFM capacity evaluations, Section 2 recommends using a conservative estimate of median damping. For tanks similar to the example tank in which inelastic bolt stretching, some nonlinear tank uplift, and slight "elephant-foot" buckling of the tank shell is expected to occur prior to failure, 5% of critical damping represents a conservative estimate of median damping for this mode. Using the median NUREG/CR-0098 (11) spectrum shape to define the SME spectrum shape, the 5% damped spectral acceleration for frequencies in the range of 5.5 to 6.6 Hz and a SME_e of 0.27g can be given by:

$$S_{A_I} = 2.12\ (0.27g) = 0.57g$$

Reference (2)(5)(10) then recommend that the impulsive mode base shear, V_I, and moment at the base of the tank shell, M_I, be given by:

$$V_I = S_{A_I} \left[W_H + W_S + W_I\right] \qquad \text{(H-3)}$$

$$M_I = S_{A_I} \left[W_H X_H + W_S X_S + W_I X_I\right] \qquad \text{(H-4)}$$

where W_I is the effective impulsive weight of the contained fluid and X_I is its effective height above the tank base. Only this effective impulsive fluid weight is important and the head weight, W_H, and side wall weight, W_S, and their effective heights could be ignored with less than a 5% error.

For tanks with (H/R) ratios greater than 1.5 such as for this example tank, (2)(10) suggest that W_I and X_I can be estimated from:

H/R ≥ 1.5

$$\frac{W_I}{W_W} = 1.0 - 0.436(R/H) \qquad \text{(H-5)}$$

$$\frac{X_I}{H} = 0.50 - 0.188(R/H) \qquad \text{(H-6)}$$

For (H/R) less than 1.5, see (2) or (10) for corresponding equations. Thus, for this example tank with (H/R) = 1.85,

$$W_I = 0.764 W_W$$

$$W_I X_I = 0.304 W_W H$$

Thus, the impulsive mode base shear and moments from Equations (H.3) and (H.4) are:

$$V_I = 1310 \text{ kips} \qquad M_I = 19{,}500 \text{ kip-ft} \qquad \text{(H-7)}$$

Veletsos (5) provides a slightly different formulation for W_I and X_I which leads to the following for this example tank:

$$W_I = 0.724W_W \qquad W_I X_I = 0.304W_W H$$

which will lead to a slightly lower base shear, V_I, and the identical base moment, M_I, as that given above in Equation (H-7). Haroun and Housner (8) provide a slightly different formulation for computing V_I and M_I which leads to the following values for this example tank:

$$V_I = 1150 \text{ kips} \qquad M_I = 20{,}800 \text{ kip-ft}$$

The differences in impulsive response results from each of these three approaches are small and within the underlying accuracy of the computations. Any of these three approaches may be used. For this example, the results given in Equation (H.7) will be used.

Next, the impulsive mode hydrodynamic pressures, P_I, on the tank should be approximately estimated. Reference (2) and (10) suggest that for depths y from the top of the fluid greater than 0.15H, this pressure P_I can be approximated by:

<u>y/H ≥ 0.15</u>

$$P_I = \frac{W_I \; X_I \; S_{A_I}}{1.36RH^2} \qquad \text{(H-8)}$$

with the impulsive pressure varying approximately linearly from zero at the top of the fluid (y = o) to the value for Equation (H.8) at y = 0.15H. For the example tank, at depths greater than 5.6-ft, the impulsive pressure is estimated from Equation (H-8) to be:

$$\underline{y \geq 5.6\text{ft}} \; : \; P_I = 3.5 \text{ psi} \qquad \text{(H-9)}$$

Both Veletsos (5) and Haroun and Housner (8) provide alternate formulations for estimating the impulsive pressure. For this example tank, by (5) the maximum impulsive pressure is estimated to be 3.4 psi, whereas by (8) the average impulsive pressure over the tank height is estimated to be 3.4 psi. Since hydrodynamic pressures seldom govern the determination of the SME capacity, any of these approaches are adequate for estimating the impulsive pressure. The impulsive pressure given by Equation (H-9) has been used in Table H-1 for this example tank.

H.2.2 Horizontal Convective (Sloshing) Mode Response

The fundamental convective mode frequency, f_c, can be estimated from:

$$f_c = \sqrt{\frac{1.50 \text{ ft/sec}^2}{R} \tanh(1.835(H/R))} \qquad \text{(H-10)}$$

Thus, for the example tank, f_c = 0.274 Hz. This convective mode is very lightly damped and it is suggested that a damping ratio of 0.5 percent of critical damping be used when estimating the convective mode spectral acceleration, S_{A_C}. For the SME spectrum shape anchored to the estimated SME_e of 0.27g, at f_c = 0.274 Hz and 0.5 percent damping the convective mode spectral acceleration is:

$$S_{A_c} = 0.084g$$

The convective mode base shear and moment are then given by:

$$V_c = S_{A_c} W_c \qquad \text{(H-11)}$$

$$M_c = S_{A_c} W_c X_c \qquad \text{(H-12)}$$

where W_c is the effective convective mode fluid weight and X_c is its effective height of application above the base.

From (1), (2) and (10), these effective convective mode weights and heights may be estimated from:

$$\frac{W_c}{W_w} = 0.46(R/H) \tanh(1.835(H/R)) \qquad \text{(H-13)}$$

$$\frac{X_c}{H} = 1.0 - \frac{\cosh(1.835(H/R)) - 1.0}{1.835(H/R) \sinh(1.835(H/R))} \qquad \text{(H-14)}$$

For the example tank with (H/R) = 1.85, these equations lead to:

$W_c = 0.248\ W_w$ $\qquad$ $W_cX_c = 0.180\ W_wH$

which result in a convective mode base shear and moment of :

$$V_C = 60 \text{ kips} \qquad M_C = 1600 \text{ kip-ft} \tag{H-15}$$

Note how low these convective mode shears and moments are relative to the impulsive mode values (Eqn (H-7).

The hydrodynamic convective pressure can be estimated from (1), (2) and (10):

$$P_C = \frac{0.267\, W_W\, S_{A_C}}{RH} \quad \frac{\cosh(1.835(\frac{H-y}{R}))}{\cosh(1.835(H/R))} \tag{H-16}$$

Such pressures are generally negligible compared to either the hydrodynamic impulsive pressure, P_I, or the hydrostatic pressure, P_{ST}, except at shallow depths below the fluid surface. For instance, at the first critical section (y = 22 ft) listed in Table H-1 for the example tank, the convective pressure, P_C, is only 0.1 psi and is less at greater depths.

Lastly, the fundamental mode fluid slosh height, h_s, can be approximated by (1), (2), (5), and (10):

$$h_s = 0.837R(S_{A_C}/g) \tag{H-17}$$

which leads to the following slosh height for the example tank:

$$h_s = 1.41 \text{ ft} \tag{H-18}$$

H.2.3 Vertical Fluid Mode Response

Hydrodynamic pressures due to the fundamental vertical fluid mode should be estimated at critical locations on the tank shell. The fundamental frequency of this vertical response mode is heavily influenced by the breathing flexibility of the tank shell and is typically only slightly greater than the horizontal impulsive mode frequency. Thus, the vertical fluid response mode typically lies in the highly amplified spectral acceleration response regime so that the use of the vertical PGA to compute the hydrodynamic pressures due to vertical response as recommended by (2) is generally inappropriate and unconservative.

One approach to compute the fundamental frequency of the vertical fluid response mode is to use:

$$f_v = \frac{C_{LV}}{2\pi H} \sqrt{E_s/\rho_s} \quad \text{where} \quad C_{LV} = C_{WV} \sqrt{\frac{0.127\rho_s}{\rho_L}} \tag{H-19}$$

which is the same equation form as that used to compute the horizontal impulsive mode response. The coefficient C_{WV} for water in a rigidly supported steel tank of uniform thickness, t, can be obtained from Table 1 of (6) as a function of the (t/R) and (H/R) ratios. For variable wall thickness tanks, an effective uniform wall thickness must be estimated similarly as was done for the horizontal impulsive modal frequency. Assuming a (t/R) ratio of 0.00092 and (H/R) = 1.85 for the example tank, C_{WV} is estimated to be 0.091 for which Equation (H-19) provides a frequency f_v of 6.4 Hz with an estimate accuracy range of about 5.9 to 7.0 Hz. The frequency of the vertical fluid response mode can also be estimated using Eqn C3500-13 of (10) which for the above example tank properties leads to a frequency f_v estimate of 6.6 Hz which is in the same range.

The hydrodynamic vertical fluid response mode pressure for a tank on a rigid foundation can be estimated from (7):

$$P_v = 0.8\ \rho_L H\ (S_{A_v})\ \cos\left(\frac{\pi}{2}\ \frac{H-y}{H}\right) \qquad \text{(H-20)}$$

which is more accurate than the linear varying pressure defined by Eqn 3500-7 of (10). An effective damping value together with the vertical mode frequency f_v must be used to estimate the vertical spectral acceleration S_{A_v}. A flexible foundation greatly reduces the vertical fluid mode hydrodynamic pressures below that computed for a rigid foundation (5)(7). One way to approximate this influence is through the use of increased damping. Even for tanks on a rock site there will be some foundation flexibility. To partially account for this effect, it is recommended that for a CDFM evaluation 5% of critical damping be used when estimating the vertical spectral acceleration S_{A_v} for tanks on rock sites.

For the previously defined SME spectrum shape anchored to a SME_e horizontal PGA of 0.27g (corresponding vertical PGA of 0.18g), the 5% damped S_{A_v} for a frequency range of 5.9 to 7.0 Hz is:

$$S_{A_v} = 2.12\ (0.18g) = 0.38g$$

Using this spectral accelerations, the hydrodynamic vertical fluid response mode pressures, P_v, computed by Equation (H-20) for the various capacity evaluation locations are presented in Table H-1 for the example tank. Note that the hydrodynamic pressures due to the vertical fluid response mode, P_v, exceed those due to the horizontal response mode at all locations.

H.2.4 Combined Responses

The combined horizontal seismic responses for base shear (V_{SH}) base moment (M_{SH}), and horizontal seismic hydrodynamic pressures (P_{SH}) can be obtained by the SRSS combination the corresponding horizontal impulsive and convective responses (2), (5)(10). Thus, for the example tank the impulsive responses (Eqn (H-7)) and the convective responses (Eqn (H-15)) produce the following combined horizontal seismic responses:

$$V_{SH} = 1310 \text{ kips} \qquad M_{SH} = 19{,}600 \text{ kip ft} \qquad \text{(H-21)}$$

Table H-1 presents the combined horizontal seismic hydrodynamic pressures, P_{SH}, at the capacity evaluation locations. Note that for this example tank, the combined horizontal seismic responses are essentially equal to the impulsive mode responses and the influence of the convective mode is negligible.

For the purposes of the membrane hoop stress capacity check, it is necessary to have an estimate of the maximum seismic hydrodynamic pressures, P_{SM}, which can be obtained by the SRSS combination of the horizontal seismic pressures, P_{SH}, and the vertical fluid response hydrodynamic pressures P_v. For the example tank, Table H-1 presents the maximum seismic hydrodynamic pressures, P_{SM}, at capacity evaluation locations.

For the purposes of estimating the compressive buckling capacity of the tank shell, it is necessary to have an estimate of the expected maximum and minimum fluid pressures acting against the tank shell near its base at the location of maximum axial compression during the time of maximum base moment. Those expected maximum and minimum compression zone pressures, P_{C+} and P_{C-}, at the time of maximum base moment can be estimated from:

$$P_{C+} = P_{ST} + P_{SH} + 0.4P_v$$

$$P_{C-} = P_{ST} + P_{SH} - 0.4P_v \qquad \text{(H-22)}$$

where the 0.4 factor on P_V is to account for the probable vertical mode hydrodynamic vertical pressure at the time of maximum base moment.

Similarly, for the purposes of estimating the expected minimum fluid hold-down forces in the zone of maximum tank wall axial tension, one needs an estimate of the minimum tension zone fluid pressure, P_{T-}, at the time of maximum moment as given by:

$$P_{T-} = P_{ST} - P_{SH} - 0.4P_v \tag{H-23}$$

For the sliding capacity evaluation one needs the expected minimum average fluid pressure on the base plate, P_a, at the time of maximum base shear as given by:

$$P_a = P_{ST} - 0.4\ P_v \tag{H-24}$$

Using the base plate hydrostatic and hydrodynamic pressures given in Table H-1, the following combined pressures are computed for the base plate from Eqns (H-22) through (H-24):

<u>Base Plate</u>

$$\begin{aligned} P_{C+} &= 16.0 + 3.5 + (0.4)(4.9) = 21.5 \text{ psi} \\ P_{C-} &= 16.0 + 3.5 - (0.4)(4.9) = 17.5 \text{ psi} \\ P_{T-} &= 16.0 - 3.5 - (0.4)(4.9) = 10.5 \text{ psi} \\ P_a &= 16.0 - (0.4)(4.9) = 14.0 \text{ psi} \end{aligned} \tag{H-25}$$

Lastly, one also needs an estimate of the expected minimum total effective weight, W_{Te}, of the tank shell acting on the base at the time of maximum moment and base shear:

$$W_{Te} = (W_H + W_S)(1 - 0.4\ (A_v/g)) \tag{H-26}$$

which for the example tank problem is:

$$W_{Te} = 57.6 \text{ kips}$$

H.3 Capacity Assessment

Generally the SME capacity of a minimally anchored flat bottom tank such as that shown in Figure H-1 is governed by the seismic overturning moment capacity at its base, M_{SC}, compared to the applied overturning moment seismic response, M_{SH}. In turn, this moment capacity depends upon the axial compressive buckling capacity of the tank shell (C_m), the tensile hold-down capacity of the anchor bolts including their anchorage and attachment to the tank (T_{BC}), and the hold-down capacity of fluid pressure acting on the tank base plate (T_e). Thus, each of these capacities must be estimated prior to estimating the overturning moment capacity.

Although unlikely for larger radius tanks, the SME capacity is sometimes governed by the sliding shear capacity at the tank base, V_{SC}, compared to the seismic base shear response, V_{SH}. Ever though it does not appear that any butt welded steel tank has ever failed due to seismic induced membrane hoop stresses due to combined hydrostatic plus hydrodynamic fluid pressures, the SME capacity of this failure mode should also be checked. Such a check requires the computation of the pressure capacity, P_{CA}, of the tank shell for comparison with the combined hydrostatic, P_{ST}, and maximum seismic hydrodynamic, P_{SM}, pressures.

Some assessment of the possibility and consequences of fluid sloshing against the tank roof should be made. For soil sites, foundation failure modes should also be checked. Lastly, the possibility of failure of piping or their attachment to the tank should be assessed.

Each of these topics will be further discussed in this subsection.

H.3.1 Compressive Buckling Capacity of the Tank Shell

The most likely way for tank shells to buckle is in "elephant-foot" buckling near the base of the tank shell. The tank shell is subjected to a biaxial stress state consisting of hoop tension and vertical (axial) compression. In addition, radial deformations under internal pressure which are prevented at the base due to membrane tension in the base plate introduce eccentricity and bending stresses in the axial plane which further induce the tendency to "elephant-foot" buckle. The onset of such "elephant-foot" buckles can be estimated using elastic-plastic collapse theory (12)(13)(14). However, it should be noted that the initiating of "elephant-foot" buckles does not directly correspond to failure of a tank. Many tanks have continued to perform their function of containing fluid even after developing substantial "elephant-foot" buckles. However, no simple capability to predict tank performance after the development of "elephant-foot" buckles exists. Therefore, for a CDFM approach to estimating the HCLPF SME capacity of tanks, the onset of "elephant-foot" buckling will be judged to represent the limit to the compressive buckling capacity of the tank shell, C_m. However, because such buckling is not failure, no significant conservatism needs to be intentionally introduced when estimating C_m.

The "elephant-foot" buckling axial stress capacity, σ_p, of the tank shell can be accurately (no intentional conservatism) estimated by (12)(13)(14):

$$\sigma_p = \frac{0.6\ E_S}{(R/t_s)}\left[1 - \left(\frac{P}{\sigma_{ye}}\frac{R}{t_s}\right)^2\right]\left[1 - \frac{1}{1.12 + S_1^{1.5}}\right]\left[\frac{S_1 + (\sigma_{ye}/36\ \text{ksi})}{S_1 + 1}\right] \qquad \text{(H-27)}$$

where $S_1 = (R/t_s)/400)$

and t_s is the sidewall thickness near the shell base, P is the tank internal pressure near its base, and σ_{ye} is the effective yield stress of the tank shell material. For HCLPF capacity computations it is suggested that a slight conservatism be introduced by specifying C_m in terms of 0.9 σ_p. Thus:

$$C_m = 0.9\ \sigma_p t_s \qquad \text{(H-28)}$$

Furthermore, P should be set equal to P_{C+} which represents the maximum combined pressure against the tank wall at the time of maximum moment. Lastly, for a tank shell material such as SA 240-Type 304 stainless steel with no specific yield point, it is uncertain what stress to use for σ_{ye}. This material shows no flat yield plateau and continues to show increasing stress with increased strain until its minimum ultimate stress capacity of 75 ksi is reached. For a CDFM capacity evaluation it seems reasonable to set σ_{ye} at the ASME Code (15) seismic design limit for primary local membrane plus primary bending which is $2.4S_M$ or 45 ksi for this material. The potential uncertainty range for σ_{ye} is estimated to be from 30 ksi to 60 ksi with it likely to exceed 45 ksi.

For the example tank, $P=P_{C+}$ = 21.5 psi, t_s = 0.375 inch, E = 27.7 x 10^3 ksi, (R/t_s) = 640, S_1 = 1.6, $(P/\sigma_{ye}) = 0.48 \times 10^{-3}$, and $(\sigma_{ye}/36\ \text{ksi}) = 1.25$. Thus, from Eqn (H-27), σ_p = 17.6 ksi. When one considers the potential range on σ_{ye} of 30 to 60 ksi, then the resultant range on σ_p is 13.1 ksi to 21.1 ksi. The influence of this uncertainty range on the SME capacity will be subsequently assessed. Using Eqn (H-28), the compressive capacity of the shell is:

$$C_m = 0.9\ (17.6\ \text{ksi})(0.375\ \text{in}) = 5.92\ \text{Kip/in} \quad \text{(H-29)}$$

Although unlikely to govern for overall seismic response of fluid containing tanks, one should also check the buckling capacity of supported cylindrical shells under combined axial bending and internal pressure. The axial bending induced buckling stress, σ_{CB}, for such a case can be conservatively (essentially lower bound) estimated from (16):

$$\sigma_{CB} = (0.6\gamma + \Delta\gamma)\frac{E_s}{(R/t_s)} \quad \text{(H-30)}$$

where $\phi = \frac{1}{16}\sqrt{R/t_s}$

$$\gamma = 1 - 0.73(1 - e^{-\phi})$$

and $\Delta\gamma$ is an increase factor for internal pressure as given by Figure 6 of (16). The minimum compression zone pressure at the base of the tank shell, P_{C-}, corresponding to the time of maximum moment should be used with Figure 6 of (16) when estimating $\Delta\gamma$. Equation (H-30) is appropriate for σ_{CB} so long as σ_{CB} is less than the yield stress, σ_Y. Otherwise, see (16). Since σ_{CB} is conservatively estimated, it may be directly used with no reduction for estimating $C_m = \sigma_{CB} t_s$.

For the example tank, $P_{C-} = 17.5$ psi, $\gamma = 0.419$, and $\Delta\gamma = 0.15$. Thus, $\sigma_{CB} = 17.4$ ksi which exceeds $0.9\sigma_p$ so this buckling mode does not govern.

H.3.2 Bolt Hold-Down Capacity

The bolt hold-down capacity, T_{BC}, is governed by the weakest of the following elements:

1. Bolt tensile capacity
2. Anchorage of bolt into concrete foundation
3. Capacity of the top plate of bolt chairs to transfer bolt loads to the vertical chair gussets.
4. Attachment of the top plate and vertical chair gussets to the tank shell.
5. Capability of tank shell to withstand concentrated loads imposed on it by bolt chairs.

To simplify this already long tank evaluation example, it will be assumed that the bolt tensile capacity is the weakest bolt hold-down link for the example tank. However, in an actual application, each of these five capacity elements need to be checked.

Type A307 anchor bolts are the most common low strength anchor bolt material used to anchor tanks and other heavy equipment. For a CDFM capacity evaluation, their capacity can be estimated based upon the Part 2 provisions of the AISC Code (17). The example tank has 2-inch diameter bolts which have a nominal cross-sectional area, A_{nom}, of 3.14 square inches. Based upon Part 2 of the AISC Code for A307 bolts the tension capacity, T_{BC}, is:

$$T_{BC} = 1.7(20 \text{ ksi})(3.14 \text{in}^2) = 107 \text{ kips} \qquad \text{(H-31)}$$

H.3.3 Fluid Hold-Down Forces

For tanks with minimum anchorage, hold-down forces resulting from fluid pressure acting on the tank bottom will contribute significantly to the overturning moment capacity, M_{SC}, of the tank. The situation in the region of axial tension in the tank shell is illustrated in Figure H-2 for small uplift displacements, δ_e. At point "o" away from the tank sidewall, the tank bottom is in full contact with the foundation and the displacements, rotation, and moment in the tank bottom is zero. However, at the intersection of the tank bottom and side wall at point "1", the tank bottom has uplifted δ_e and rotated α_e. The length of the uplift zone is ℓ and the fluid pressure, P, on the tank bottom and side wall resists this uplift. This uplift is accompanied by the development of a tension, T_e, and moment, M_e, in the side shell at the intersection with the tank bottom. This tension, T_e, acts as a fluid hold-down force on the tank shell. For a given uplift height, δ_e, the hold-down tension, T_e, that develops is both a function of the bending stiffness of the tank shell which is a function of its thickness, t_s, and radius, R, and the bending stiffness of the base plate which is a function of its thickness, t_b.

For a tank shell restrained against radial displacement at point "1" by the base plate, the relationship between M_e and α_e can be obtained from pages 276 through 278 of Flugge (18) to be:

$$M_e = K_S \alpha_e + M_F \qquad \text{(H-32)}$$

where $K_S = \frac{2K\kappa}{R}$

$$\frac{M_F}{P} = \frac{Rt_s}{\sqrt{12(1-\nu^2)}} \left[1 - \frac{R}{H\kappa}\right]$$

$$K = \frac{E_s\ t_s^{\ 3}}{12(1-\nu^2)}$$

$$\kappa = \left[(R/t_s)\sqrt{3(1-\nu^2)}\right]^{1/2}$$

One can show for the base plate that:

$$\alpha_e = \frac{P\ell^3}{12E_sI_b} - \frac{M_e\ell}{2E_SI_b} \qquad \text{(H.33)}$$

$$\delta_e = \frac{P\ell^4}{24E_sI_b} - \frac{M_e\ell^2}{6E_sI_b}$$

$$T_e = \frac{P\ell}{2} + \frac{M_e}{\ell}$$

where $I_b = \frac{t_b^{\ 3}}{12(1-\nu^2)}$

Combining Equations (H-32) and (H-33), one obtains:

$$\frac{E_sI_b\delta_e}{P} = \left[\frac{\ell^4}{24} - \left(\frac{1}{F}\right)\left(\frac{K_s\ell^5}{72E_sI_b} + \frac{M_F}{P}\frac{\ell^2}{6}\right)\right] \qquad \text{(H-34)}$$

$$\frac{T_e}{P} = \left[\frac{\ell}{2} + \left(\frac{1}{F}\right)\left(\frac{K_s\ \ell^2}{12E_sI_b} + \frac{M_F}{P\ell}\right)\right] \qquad \text{(H-35)}$$

$$\frac{M_e}{P} = \left(\frac{1}{F}\right)\left(\frac{K_s\ \ell^3}{12E_sI_b} + \frac{M_F}{P}\right) \qquad \text{(H-36)}$$

$$\frac{M_+}{P} = \frac{\ell^2}{8} - \frac{(M_e/P)}{2} + \frac{(M_e/P)^2}{2\ell^2} \qquad \text{(H-37)}$$

where $F = \left[1 + \frac{K_s\ell}{2E_sI_b}\right]$

Using Equations (H-34) through (H-37), one can determine the uplift height (δ_e), tank shell hold-down tension (T_e), end moment (M_e), and maximum positive moment (M_+) in the base plate as a function of the uplift length, ℓ, and fluid pressure, P. From this information the relationship between δ_e and T_e is obtained. This small displacement theory solution is only strictly applicable under the following conditions:

1. $(\ell/R) \le 0.15$. The solution ignores the stiffening of the base plate from hoop behavior and thus conservatively overpredicts the displacement δ_e corresponding to a given T_e as the ratio (ℓ/R) becomes larger.

2. $(\delta_e/t_b) \le 0.6$. This solution is based upon small displacement theory and conservatively ignores the beneficial influence of large displacement membrane theory together with membrane tensions in the base plate to reduce δ_e corresponding to a given T_e. For unanchored tanks it has been shown (19)(20) that large displacement membrane theory greatly increase the fluid hold-down forces, T_e. Thus, for unanchored tanks, ignoring large displacement membrane theory is likely to lead to excessive conservatism. For anchored tanks the uplift heights δ_e are not expected to be so great and only moderate conservatism is expected to result from ignoring large displacement membrane effects. Unfortunately no simple solution exists for considering such membrane effects and so one must either accept this source of conservatism for anchored tanks at this time or make judgmental corrections to the completed fluid hold-down forces following guidance from (19) and (20).

3. $(M_e/M_{p_b}) \le 0.9$; $(M_e/M_{p_s}) \le 0.9$; and $(M_+/M_{p_b}) \le 0.9$ where M_{p_b} and M_{p_s} are the plastic moment capacity of the base plate and shell sidewalls, respectively. The previous solution is an elastic solution and becomes unconservative if these conditions are not met. An alternate solution with plastic hinges at locations where these conditions are not met is easily formulated following the same approach as was used herein but is judged to be unwarranted because violation of these conditions is highly unlikely in a CDFM evaluation.

Only this third condition leads to unconservative estimates of the hold-down force T_e corresponding to a given uplift displacement δ_e. The first two conditions can be violated so long as one is willing to accept the resulting conservative underestimation of the fluid hold-down force T_e for a given uplift displacement δ_e

Since the hold-down force T_e increases with increasing fluid pressure, P, one should conservatively substitute for P the minimum tension zone fluid pressure P_{T-}, expected at the time of maximum moment. For the example tank problem, P = P_{T-} = 10.5 psi. Using this fluid pressure and Equations (H-34) through (H-37), the relationship between T_e, δ_e, and ℓ has been estimated for the example tank. Figure H-3 shows the relationship between T_e and δ_e while Figure H-4 relates T_e and ℓ. Note that even with zero uplift there is a hold-down tension T_e of 58-lbs/inch resulting from base rotation of the side wall due to the outward acting fluid pressure thereon. At an uplift δ_e of 0.15 inches which corresponds to $0.6t_b$ or the approximate limit of small displacement theory, the hold-down tension has increased to 98-lbs/inch. Beyond this point, small displacement theory will become increasingly conservative. The relationship shown in Figure H-3 between T_e and δ_e can be reasonably and slightly conservatively approximated by the linear expression:

$$Te \approx T_{eo} + T_{e1}\delta_e \qquad \text{(H-38)}$$

where for the example tank: T_{eo} = 60 lbs/inch, and T_{e1} = 270 lbs/inch^2. This approximation will be used in the overturning moment capacity evaluation. To partially account for membrane tension effects, this equation for T_e will be linearly extrapolated beyond the displacement δ_e of 0.15 inch for which it is directly applicable.

H.3.4 Overturning Moment Capacity

With an estimate of the compressive capacity of the tank shell (C_m), the anchor bolt hold-down capacity (T_{BC}), and the relationship between fluid hold-down force and uplift displacement, it is possible to estimate the overturning moment capacity (M_{sc}) of the tank making several conservative, but reasonable assumptions:

1. The bottom of the tank shell is assumed to rigidly rotate vertically (plane sections remain plane).

2. The cross-section of the tank at the top of the top plate of the bolt chairs (h_c above the base in Figure H.1) is assumed to remain

horizontal so that all vertical tank distortions needed to result in base uplift and mobilization of the anchor bolts must be accommodated over the height, h_c.

3. The tank shell remains linear elastic until the compressive buckling capacity is reached at the point of maximum compression and reaching this limit defines the overturning moment capacity. The maximum compressive shortening of the tank shell which takes place over the length h_c between the tank base and the top plate of the bolt chains is conservatively underestimated to be:

$$\delta_c = \frac{C_m \, h_c}{E_s \, t_s} \qquad \text{(H-39)}$$

Figure H-5 schematically illustrates the vertical loadings applied to the tank shell and its vertical rotational distortion resulting from these assumptions. At any angle α, the vertical uplift, δ_{e_α}, is then:

$$\delta_{e_\alpha} = \delta_c \left(\frac{\cos\alpha - \cos\beta}{1 + \cos\beta}\right) \qquad \text{(H-40)}$$

Then, if the anchor bolts are assumed to be anchored at a depth h_a below the tank base as is shown in Figure H-1, and chair distortions are considered negligible, then the anchor bolt tension, T_{B_i}, in anchor bolt "i" corresponding to a maximum tank shell compressive distortion, δ_c, below the top of the bolt chair is:

$$T_{B_i} = T_{BP} + d_{e_i} \left(\frac{A_B \quad E_B}{h_a + h_c}\right) \leq T_{BC} \qquad \text{(H-41)}$$

or by combining with Equation H-40:

$$T_{B_i} = T_{BP} + K_B \left(\frac{\cos\alpha_i - \cos\beta}{1 + \cos\beta}\right) \leq T_{BC} \qquad \text{(H-42)}$$

where $$K_B = \frac{\delta_c \, A_B \, E_B}{(h_a + h_c)} \qquad \text{(H-43)}$$

and α_i is the angle α at bolt "i", T_{BP} is any bolt pretension, A_B is the bolt area, and E_B is the bolt modulus of elasticity. The fluid hold-down force, T_{e_α}, at angle α obtained by combining Equations H-38 and H-40 is:

$$T_{e_\alpha} = T_{eo} + \Delta T_e \left(\frac{\cos\alpha - \cos\beta}{1 + \cos\beta}\right) \qquad \text{(H-44)}$$

where $$\Delta T_e = T_{e1} \, \delta_c \qquad \text{(H-45)}$$

For the force distribution shown in Figure H-5, the tank maximum shell compression, C_m', and overturning moment capacity, M_{SC} can be obtained by setting the sum of all vertical forces to W_{Te} and summing moments about the centerline of the circular tank cross-section. Thus:

$$C_m' = \left(\frac{W_{Te} + \sum_{i=1}^{n} T_{Bi}}{2R} + T_{eo}\,\beta\right) C_1 + \Delta T_e C_3 \tag{H-46}$$

$$M_{SC} = C_m' C_2 R^2 + \sum_{i=1}^{n} (T_{B_i} R \cos\alpha_i) + T_{eo} R^2 (2 \sin\beta) + \Delta T_e C_4 R^2 \tag{H-47}$$

where

$$C_1 = \frac{1 + \cos\beta}{\sin\beta + (\pi-\beta)\cos\beta}$$

$$C_2 = \frac{\sin\beta \cos\beta + \pi-\beta}{1 + \cos\beta}$$

$$C_3 = \frac{\sin\beta - \beta\cos\beta}{\sin\beta + (\pi-\beta)\cos\beta} \tag{H-48}$$

$$C_4 = \frac{\beta - \sin\beta\cos\beta}{1 + \cos\beta}$$

First, a trial angle β is selected and Equation H-46 is used to obtain C_m' which is compared to the shell compressive capacity C_m from Equation H-28. The angle β is varied until $C_m' = C_m$. Then Equation H-47 is used with this β to determine the tank overturning moment capacity.

The example tank problem has the following properties:

C_m=5.92 kips/in	T_{BC}=107 kips	
T_{eo}=0.060 kips/in	T_{e1}=0.270 kips/in^2	
W_{Te}=57.6 kips	A_B=3.14 inch^2	E_B=29x10^3 ksi
R=20 ft=240 inches	h_c=24.75 inches	h_a=28.5 inches
t_s=0.375 inch	E_s=27.7x10^3 ksi	

Then, from Equation (H-39) δ_c = 0.0141 inch; from Equation (H-43) K_B = 24.5 kips; and from Equation (H-45) ΔT_e = 0.0038 kips/inch . Since any bolt pretension, T_{BP}, is unreliable after a number of years, it will be conservatively assumed that T_{BP} is zero. Now it is possible to compute the overturning moment capacity, M_{SC}, using Equations (H-47) and (H-48) as shown in Table H-2.

Thus,

$$M_{SC} = 20,800 \text{ kip-ft}$$
$$\Sigma T_B = 635.7 \text{ kips} \qquad \text{(H-49)}$$
$$\beta = 2.70$$

One should check whether the largest bolt elongation ($\alpha = 0$) is acceptable using Equation (H-40) to determine the elongation. For this solution:

$$\delta_{eo} = 0.28 \text{ inches}$$

Certainly these bolts which have an overall length of 53.25 inches can accommodate a 0.28 inch or 0.53% elongation. The ability of the connection between the bottom plate and the tank side wall to withstand the distortions associated with this uplift height, δ_{eo}, should also be considered. However, a well-designed detail at this location should be capable of easily withstanding more than 0.3 inch uplift. If the elongation in considered too great, then neither δ_c nor the compressive buckling capacity can develop in the compressive zone. Then, a maximum δ_{eo} should be defined and the quantities δ_c and C_m should be back calculated using Equations (H-40) and (H-39), respectively, for any given trial β. These back calculated δ_c and C_m values should then be used to solve for M_{SC}. However, seldom will the resultant δ_{eo} be excessive so that this additional refinement is seldom necessary.

The previous solution was for the case when α=0 is aligned with one of the bolts. This case will nearly always govern. However, one should also check the case where α=0 lies midway between bolts (i.e., for the example problem rotate the α=0 line by 22.5 degrees). For the example problem, M_{SC}=21,000 kip-ft for this alternate case and thus does not govern.

As noted in Subsection H.3.1, the HCLPF buckling capacity of the tank shell has some uncertainty. This capacity could possibly (but highly unlikely) range as low as 4.42 kips/in, rather than the 5.92 kips/in used in the above calculations. With C_m of 4.42 kips/in, the overturning capacity M_{SC} would be reduced to 19500 kip-ft or 94% of that for C_m=5.92 kips/in. Thus, M_{SC} is rather insensitive to the estimate of C_m for this tank and the uncertainty issue previously raised for estimating C_m turns out to be relatively unimportant.

Given the estimate of M_{SC}=20,800 kip-ft and the previously estimated overturning response M_{SH}=19,600 kip-ft for an SME_e of 027g, it is now possible to estimate the SME level from Equation H-1. However for this estimate, one must have an estimate of the inelastic energy absorption reduction factor, k, to apply to linear computed seismic response. Certainly, this combined bolt yielding and tank shell buckling failure mode for overturning moment is not brittle so that a k value less than unity should be appropriate in a HCLPF capacity evaluation. However, within the existing state of knowledge it is very difficult to make an appropriate estimate of k for this failure mode. Therefore, it is conservatively recommended that k be taken as unity at this time for this failure mode. Future research into tank failure capacities will likely lend to a less conservative recommendation in the future. With this recommendation, from Equation H-1:

$$SME_M = \frac{20,800 \text{ kip-ft}}{19,600 \text{ kip-ft}} (0.27g) = 0.29g \qquad \text{(H-50)}$$

based upon the overturning moment capacity.

H.3.5 Sliding Capacity

As noted previously, one should also check the sliding shear capacity, V_{SC}, with the seismic base shear response, V_{SH}. Since the base shear, V_{SH} and the base overturning moment, M_{SH}, are primarily due to the fluid horizontal impulsive mode of response, they both are maximum at the same time. Thus, the sliding shear capacity is:

$$V_{SC} = (COF) \left[W_{V_e} + (\Sigma\ T_B)\right] \qquad \text{(H-51)}$$

where $$W_{V_e} = W_{T_e} + P_a\ (\pi R^2) \qquad \text{(H-52)}$$

and (COF) is the coefficient of friction between the tank base and its foundation. It should be noted that the effective tank shell weight, W_{T_e}, used for the overturning moment calculations did not include the effective weight of the tank base plate. Neither does the seismic base shear response, V_{SH}, include the response forces associated with this base plate. One could add the effective base plate weight to W_{T_e} in Equation H-52, but then would also have to add its response contribution to V_{SH}. For any tank where the base plate weight is small compared to the fluid weight, this refinement is unnecessary in that the SME for base shear capacity will be uneffected.

Most large diameter flat bottomed tanks such as the example tank have a slight cone to their bottom plate so that contained fluid will always drain away from the center and toward the drain pipe at the edge. This cone is generally created by a variable thickness sand cushion between the tank bottom plate and its foundation. Furthermore, the tank bottom is generally made up of slightly overlapped fillet welded individual plates. Thus, the surface between the bottom plate and the sand cushion contains a series of rough steps. Under these conditions, it is reasonably conservative to estimate:

$$(COF) \geq 0.7 \qquad \text{(H-53)}$$

However, under other base plate details, a conservative (COF) might be less.

For the example tank, W_{T_e} = 57.6 kips, P_a = 14.0 psi, and (ΣT_B) = 635.7 kips have been previously computed. Thus, from Equations (H-51) through (H-53):

$$W_{V_e} = 2600 \text{ kips}$$

$$V_{SC} = 2260 \text{ kips} \qquad \text{(H-54)}$$

For base shear sliding, it is recommended that the inelastic energy absorption reduction factor k be taken as unity. Combining the above V_{SC} with the previously computed base shear response, V_{SH} = 1310 kips for SME_e = 0.27g, from Equation (H-1) one obtains:

$$SME_V = \frac{2260 \text{ kips}}{1310 \text{ kips}} (0.27g) = 0.47g \qquad \text{(H-55)}$$

based on base shear. Since SME_V substantially exceeds SME_M, base shear does not govern the seismic margin capacity of this tank. This situation is nearly always the case.

H.3.6 Fluid Pressure Capacity

It is recommended that the CDFM hoop membrane stress capacity of the tank shell, σ_a, be taken as the ASME Code (15) seismic design limit for primary stress of $2.0S_M$ or 37.5 ksi for SA240-Type 304 stainless steel. Using this stress limit, the pressure capacities, P_{CA}, at capacity evaluation locations of the example tank are given in Table H-1. A hoop membrane stress failure mode of a steel tank with full penetration butt-welded joints is certainly ductile. For this case an inelastic energy absorption seismic response reduction factor of k=0.8 can be easily justified for HCLPF capacity evaluations. Using the capacity pressure, P_{CA}, the hydrostatic pressures, P_{ST}, and the maximum seismic induced hydrodynamic pressures, P_{SM}, listed in Table H.1 together with k=0.8, the SME_p associated with fluid pressure can be computed using Equation (H-1). Table H-1 presents these computed SME_p at several capacity evaluation locations for the example tank. Note that the lowest computed SME_p is 1.3g which is many times greater than SME_M so that it does not govern. Even if k=1.0 had been used, the lowest SME_p would have been 1.0g from which the same conclusion would be reached. Actually, the fluid pressure capacity never seems to govern the seismic capacity either by the CDFM capacity calculation procedure or from seismic experience for normal designed flat bottomed steel tanks with butt-welded side plates. Therefore, pressure computations are included more for completeness than for SME capacity evaluations.

H.3.7 Other Capacity Checks

For the example tank, the fluid slosh height, h_s, was estimated to be 1.41 ft. for SME_e = 0.27g. Based on Figure H-1, this slosh height would have to exceed about 3.4 ft before any significant roof damage might be expected. Thus from Equation (H-1), $SME_S \geq 0.65g$ and will not govern. Even if roof damage might be expected, such damage is unlikely to impair the ability of the tank to contain fluid for at least a few days after an earthquake.

For tanks on soil sites, one should also check the SME capacity of the tank foundation and this check sometimes governs.

Lastly, the possibility of piping failure or the failure of nozzles where such piping is attached to the tank should be checked. Such failures will likely lead to loss of tank contents. In fact, a significant fraction of the cases of seismic induced loss of tank contents have been due to such failures when the piping contained poor seismic details. A check of seismic details of piping and their attachment to such tanks should be made during the seismic walkdown. A SME evaluation of piping and nozzles is only necessary when potentially poor seismic details are observed. Otherwise, this failure mode can be screened out during the walkdown. The issues involved are:

1. Are heavy pipe valves or long piping runs being supported through the piping nozzles off of either the tank side walls or the bottom plate,

or are they independently supported? If heavy valves or long piping runs are being supported off the tank, then the ability of the nozzles and the tank side wall or bottom plate to withstand the imposed seismic induced inertial forces should be checked. Methods outlined in Welding Research Bulletin 107 (21) may be used to compute local stresses in the tank shell, whereas the strength acceptance criteria for vessels contained in Section 6 can be used for stress capacity. If heavy valves or long piping runs are independently supported as is the normal case, then for SME levels up to 0.5g these inertial checks at piping attachment points should generally be unnecessary based upon seismic experience and judgment.

2. Is there sufficient piping flexibility to accommodate relative seismic anchor movements (SAM) between where the piping is supported from the tank shell and where it is independently supported? Almost any type of flexibility loop in the pipe between the tank and independent piping supports should be sufficient for SME levels up to 0.5g so no evaluation should generally be necessary. However, if a straight run of pipe exists between where the pipe is independently rigidly supported and the tank shell, the piping nozzle and tank shell should be evaluated for their ability to withstand the expected relative SAM.

H.4 Discussion of Seismic Capacity

For the example tank, the HCLPF SME PGA capacity of 0.29g was governed by overturning moment which is nearly always the case. Again, it should be noted that this HCLPF statement is conditional on the SME response spectrum anchored to this SME PGA of 0.29g not being exceeded by a future ground motion at more than 16% of the natural frequencies within the frequency range and direction of interest. The overturning moment response and thus the SME capacity is primarily governed by the horizontal impulsive response mode with an estimated frequency of about 6 Hz. Thus, the frequency range of interest is about 5 to 7 Hz. For a circular tank, the direction of interest is the direction of largest horizontal ground motion. Thus, the HCLPF statement is conditional on the SME response spectrum anchored to a PGA of 0.29g not being exceeded at more than 16% of the natural frequencies between 5.0 and 7.0 Hz in the direction of largest horizontal ground motion.

Within the frequency range of 5.0 to 7.0 Hz, the SME 5% damped response spectrum used for this example tank had a spectral amplification factor of 2.12. Therefore, the SME 5% damped spectral acceleration becomes (2.12) (0.29g) = 0.61g. Therefore, rather than defining the HCLPF SME capacity of this tank in terms of its PGA of 0.29g, an improved HCLPF SME capacity statement for this tank would be as follows:

> "The HCLPF SME capacity of this example tank is a 5% damped, 84% non-exceedance probability spectral acceleration between 5.0 and 7.0 Hz of 0.61g in the direction of largest horizontal ground motion."

Again, it should be noted that this capacity is a HCLPF capacity and not the median capacity which is expected to be more than twice as great.

H.5 References

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Table H-1.

Hydrostatic and Hydrodynamic Pressures and SME Capacity
at Capacity Evaluation Locations

Wall Section	y	Individual Pressures (psi) P_{ST}	P_I	P_c	P_v	Combined Pressures (psi) Horz. Seismic P_{SH}	Maximum Seismic P_{SM}	Capacity Pressures (psi) P_{CA}	Capacity SME_P (g)
3/16"	22'	9.5	3.5	0.1	3.9	±3.5	±5.3	29.3	1.3
1/4"	30'	13.0	3.5	--	4.7	±3.5	±5.8	39.1	1.5
3/8"	36'	15.6	3.5	--	4.9	±3.5	±6.0	58.6	2.4
Base	37'	16.0	3.5	--	4.9	±3.5	±6.0	--	--

Table H-2.

Computation of Overturning Moment Capacity, M_{SC}

Trial β	$(1+\cos\beta)$	C_1	C_2	C_3	C_4	T_{B1} kips	$2T_{B2}$ kips	$2T_{B3}$ kips	$2T_{B4}$ kips	ΣT_B kips	C_M' kip/in	M_{SC} kip-ft
2.80	0.0578	4.40	.449	286	53.9	107	214	214	199.4	734.4	9.09	23,100
2.75	0.0757	3.84	.513	186	41.0	107	214	214	140.7	675.7	7.21	21,900
2.70	0.0959	3.41	.575	102	32.2	107	214	214	100.7	635.7	5.87	20,800

Materials: Shell & Chairs: SA240 - Type 304 SST
Bolts: A307 - 2" ¢

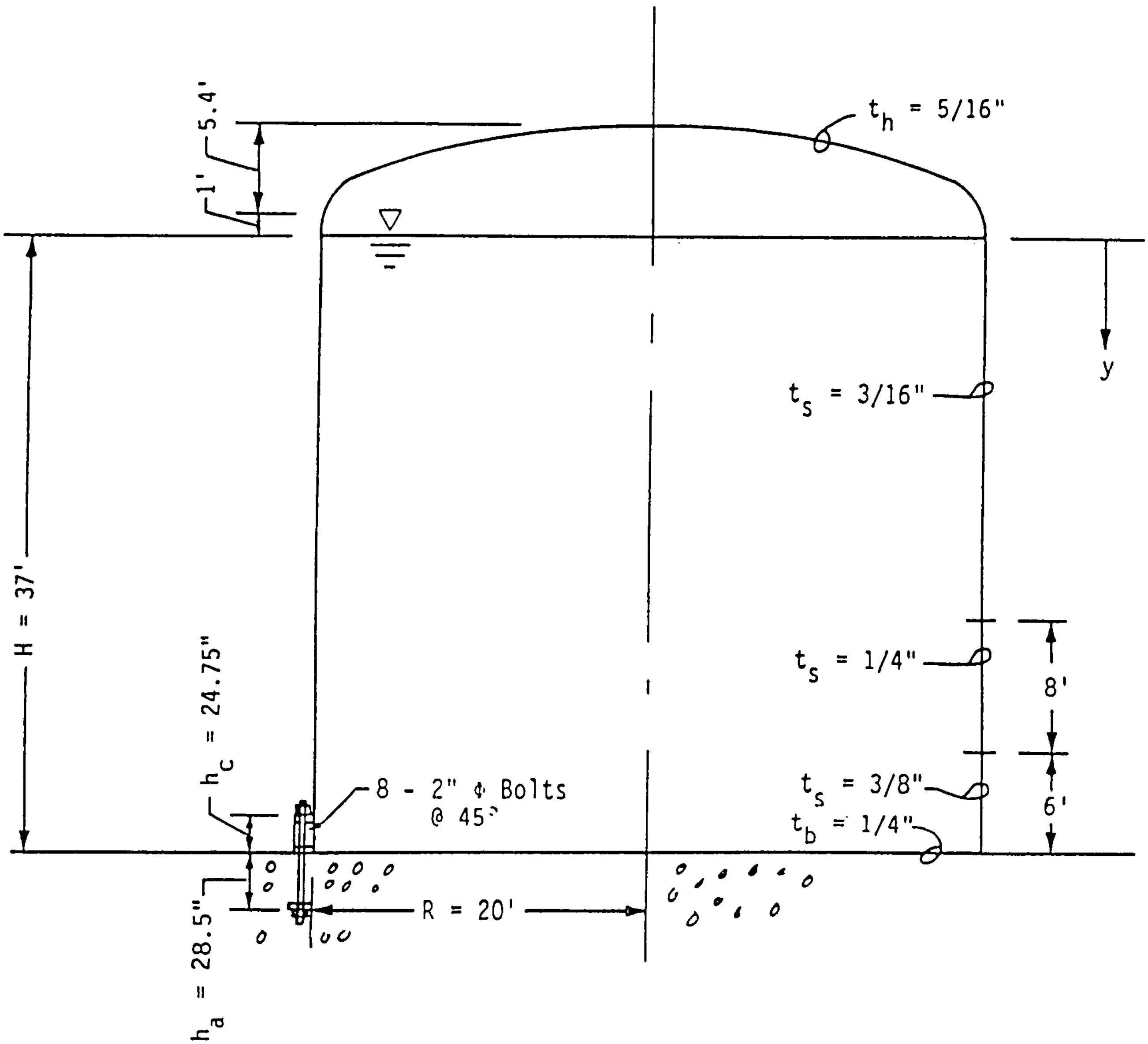

Figure H-1. Example Tank

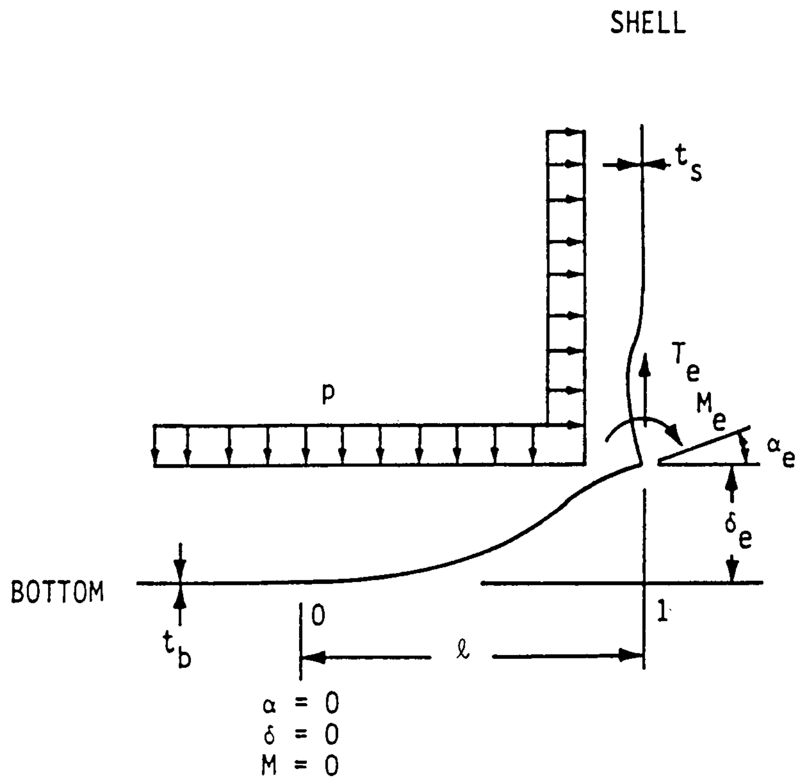

Figure H-2. Schematic Illustration of Tank Bottom Behavior Near Tensile Region of Tank Shell

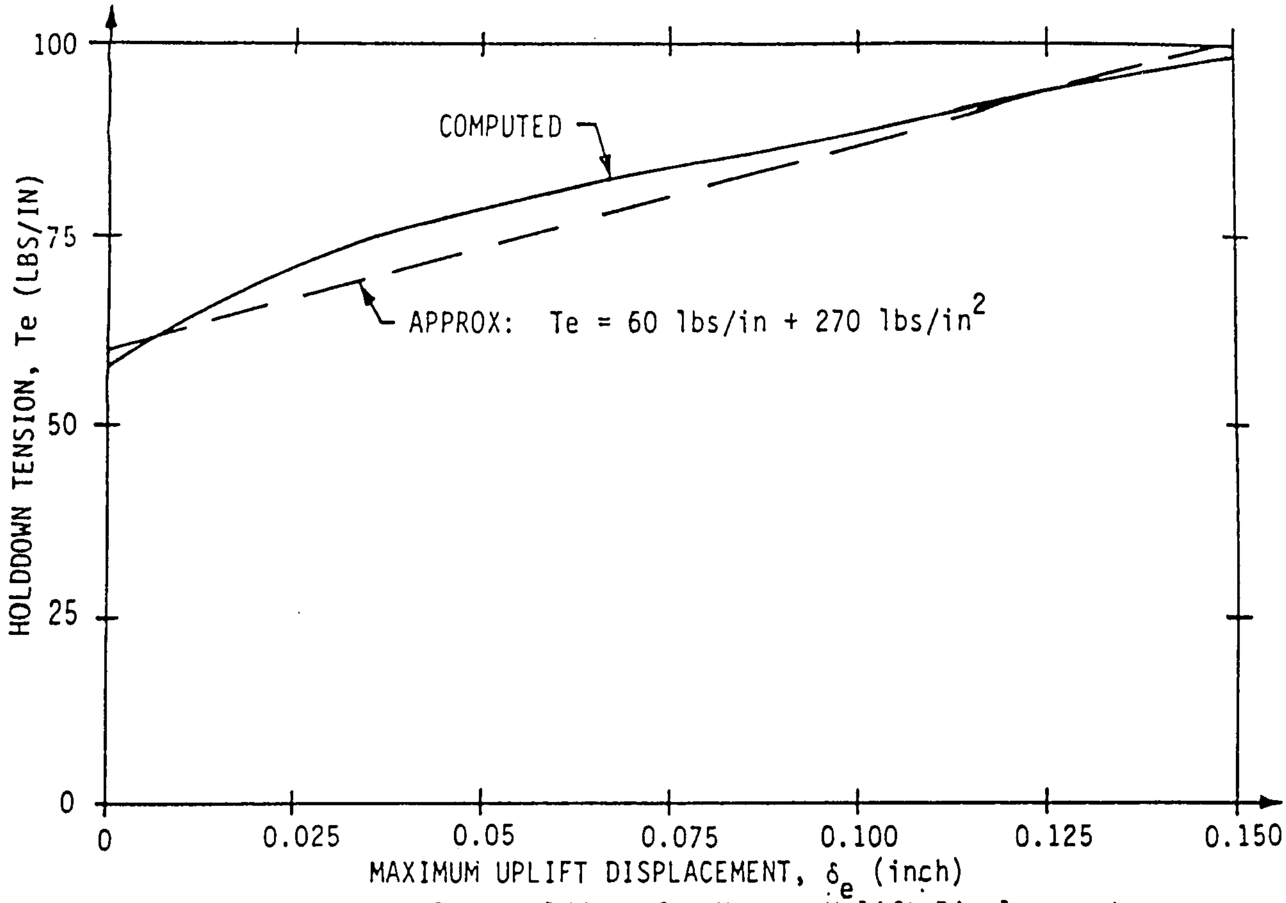

Figure H-3. Fluid Holddown for Versus Uplift Displacement for Example Tank

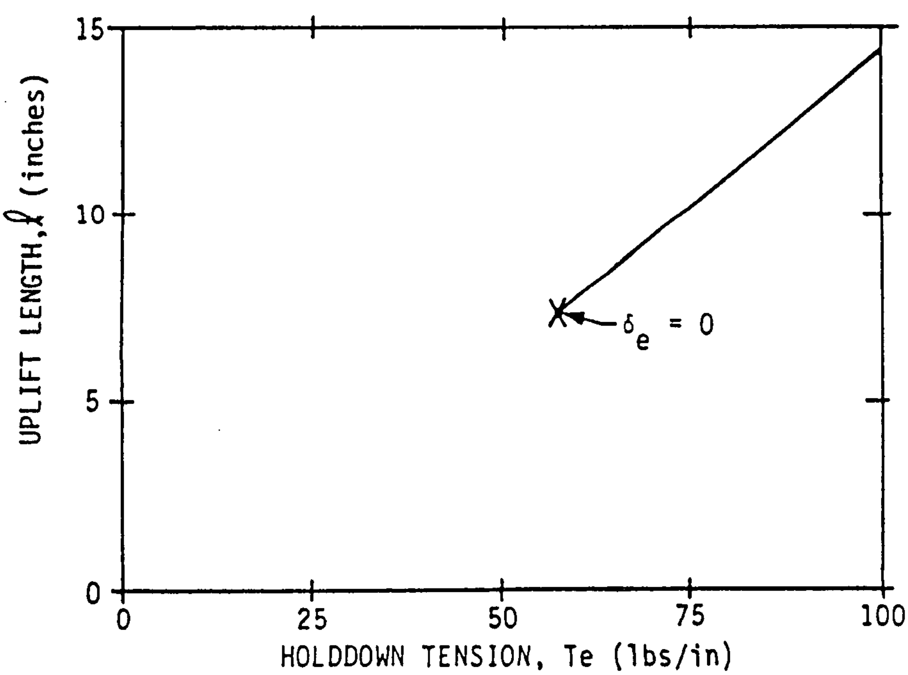

Figure H-4. Tank Bottom Uplift Length Versus Holddown Tension for Example Tank

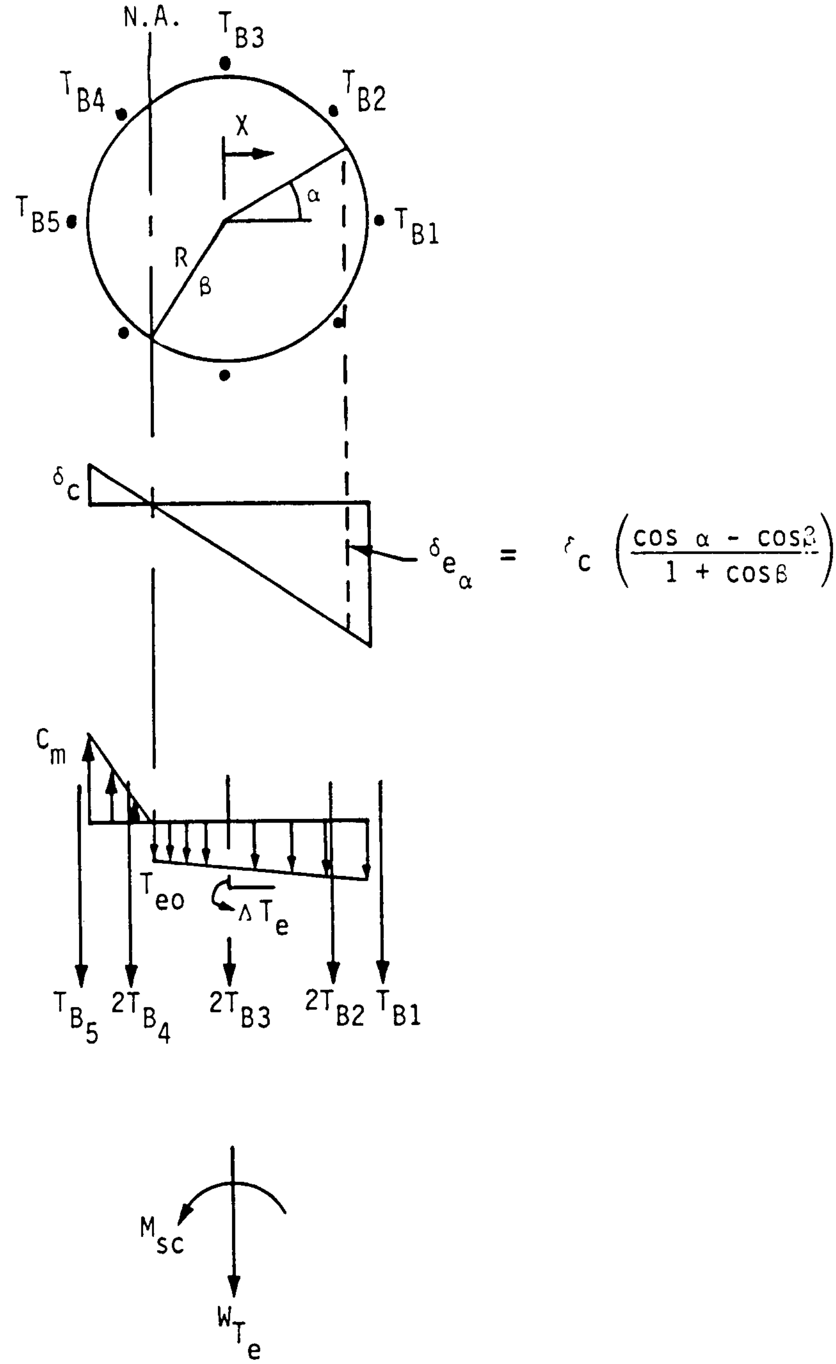

Figure H-5. Vertical Loading on Tank Shell at Base

FLAT-BOTTOM VERTICAL WATER STORAGE TANK

FRAGILITY METHOD

By

R. P. Kennedy

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1) Tank Shown in Fig. 1

2) Capacity controlled by overturning moment (See Appendix H CDFM Calculations) due to largest horizontal component

3) Estimated Median Capacity $\check{SME}_e = 0.6g$ (84% NEP)

$$\check{SME} = \frac{\text{Capacity}}{\text{Response}_e}(SME_e)$$

Response$_e$ for $SME_e = 0.6g$

$$M_e = M_{CDFM_e} \cdot \left(\frac{0.6g}{SME_{CDFM_e}}\right) \cdot \frac{F_D \, F_F}{F_{GMH}}$$

Quantity	Median	β_R	β_U	Comment
SME_{CDFM_e}	0.27g	—		Appendix H, Pg H.4
M_{CDFM_e}	19600 k'	—		Appendix H, Eqn H-21
F_{GMH}	1.22	0.20	—	Note 1
F_D	1.0	—	0.10	Note 2
F_F	1.0	—	0	Note 3
M_e	35,700 k'	0.20	0.10	

Note 1 : F_{GMH} = Horizontal Ground Motion Response Factor

Median Tank Response is controlled by median largest horizontal spectral response at tank impulsive mode natural frequency while SME spectrum defined as 84% NEP largest horiz. spectral response

$$\therefore \check{F}_{GMH} = F_7 = 1.22 \quad \text{(Table 1)}$$

Must include peak & valley variability

$$\therefore \beta_{R_{GMH}} = \beta_{R_1} = 0.20 \quad \text{(Table 1)}$$

Note 2 : F_D = Damping Uncertainty Factor

Overturning Moment Controlled by Horiz. Impulsive Mode at median frequency, $\check{f}_1 = 6.0$ Hz (Appendix H) using NUREG/CR0098 Median Amplification Factors.

CDFM : Damping = 5% $AF_{6.0,5\%} = 2.12$ } $\check{F}_D = 1.0$
Median : Damping = 5%
$-1.5\beta_U$: Damping = 3% $AF_{6.0,3\%} = 2.46$

$$\beta_{D_U} = \frac{\ln(2.46/2.12)}{1.5} = 0.10$$

Note 3 : F_F = Freq. Shift Effect

$\pm 1\beta$ freq. range = 5.5 Hz to 6.6 Hz

Within this range AF does not change for CR0098 Spectrum

$$\therefore \check{F}_F = 1.0 \qquad \beta_{F_U} = 0$$

Other Response Quantities Required for Capacity Evaluation for $SME_e = 0.6g$ (84% NEP Largest Horiz)

Pressures On Base

Static Pressure : $P_{ST} = 16.0$ psi (Pg H.30, Appendix H)

Impulsive Pressure : $P_I = P_{I_{CDFM}} \cdot \left(\frac{0.6g}{0.27g}\right)\left(\frac{F_D \; F_F}{F_{GMH}}\right)$

$P_{I_{CDFM}}$: (pg H.30) → 3.5 psi; 0.27g: SME_{CDFM_e}; $F_D F_F / F_{GMH}$: same as M_e

Vert. Pressure : $P_V = P_{V_{CDFM}} \left(\frac{0.6g}{0.27g}\right)\left(\frac{F_D \; F_F}{F_{GMV}}\right)$

$P_{V_{CDFM}}$ = (pg H.30) → 4.9 psi

① F_D & F_V same as for M_e
② F_{GMV} is for vertical

← Ratio Vert/Horiz Used in CDFM Calcs

$$\check{F}_{GMV} = \frac{\check{F}_6}{\check{F}_8}(0.67) = \frac{1.41}{0.67}(0.67) = 1.41$$

$$\beta_{R_{GMV}} = \beta_{R_8} = 0.34$$

} From Table 1

F_{EC} = Earthquake Directional Component Combination Factor

$\check{F}_{EC} = 0.4$ $\quad \beta_{R_{EC}} = 0.39$

Max Comp. Zone Pressure : $P_{C+} = P_{ST} + P_I + F_{EC} \cdot P_V$
Min Comp Zone Pressure : $P_{C-} = P_{ST} + P_I - F_{EC} P_V$
Min Tension Zone Pressure : $P_{T-} = P_{ST} - P_I - F_{EC} P_V$

} Eqn H.22 & H.23 (Appendix H)

Effective Holddown Wt

$$W_{T_e} = 62.1^k \left[1 - F_{EC} \cdot F_9 \cdot SME_e\right]$$

$62.1^k = W_H + W_S$; F_9 ← Table 1; SME_e = 0.6g

Eqn H.26 App. H

$\check{F}_9 = 0.48$ $\quad \beta_{R_9} = 0.34$

Parameter	Median	β_R	β_U	β_C	Comments
P_{ST}	16.0 psi	—	—	—	Basic Values
P_I	6.4 psi	0.20	0.10	0.22	
P_V	7.8 psi	0.34	0.10	0.42	
P_{C+}	26.1 psi	0.09	0.04	0.10	Best Fit Lognormal Distribution From Simulation Using Eqns from Previous Page & β_U dependent
P_{C-}	18.8 psi	0.13	0.02	0.13	
P_{T-}	5.3 psi	0.38	0.18	0.42	
W_{Te}	53.6^k	0.08	0.02	0.09	

Capacity Computations

Compressive Capacity Tank Shell

Controlled by Elephant Foot Buckling (Eqn H.27, App. H)

$$C_m = F_E\left(\frac{0.6 E_s t_s}{(R/t_s)}\right)\left[1-\left(\frac{P_{C+}\,R}{\sigma_{ye}\,t_s}\right)^2\right]\left[1-\frac{1}{1.12+S_I^{1.5}}\right]\left[\frac{S_I+\frac{\sigma_{ye}}{36^{ksi}}}{S_I+1}\right]$$

$$S_I = \left(\frac{R}{400 t_s}\right) = \left(\frac{240''}{400(3/8'')}\right) = 1.6$$

$$E = 27.7\times10^3\,ksi \qquad t_s = 3/8'' \qquad R = 240''$$

$$\therefore\; C_m = 9.74^{k/in}\, F_E\left[1-\left(\frac{640\,P_{C+}}{\sigma_{ye}}\right)^2\right]\left[0.682\right]\left[\frac{1.6+\frac{\sigma_{ye}}{36^{ksi}}}{2.6}\right]$$

F_E = Eqn. Error Variable $\quad \check{F}_E = 1.0 \quad \beta_{U_E} = 0.11$

σ_{ye} = Effective Yield Stress for SA240-Type 304 SS which has no abrupt yield stress. Will lie between σ_y at 0.2% offset and σ_u but expected to be closer to σ_u

Parameter	Median	β_R	β_U	β_C	Comments
σ_Y	37.0 ksi	—	0.13	0.13	Expected Properties Type 304 SS
σ_U	84.0 ksi	—	0.07	0.07	Expected Properties Type 304 SS
σ_{ye}	68.0 ksi	—	0.25	0.25	Estimated By Judgement from Stress/Strain Curve with Above Properties (local plastic bending)
P_{C+}	26.1 psi	0.09	0.04	0.10	pg 4
F_E	1.0	—	0.11	0.11	pg 4
C_m	8.36 k/in	0.01	0.20	0.20	Derived using Pg 4 Eqn. & above parameters by simulation. Best fit lognormal distribution

Anchor Bolt Tension Capacity − 2"φ A307

$A_{nom} = 3.14 \text{ in}^2 \qquad A_{stress} \approx 0.788\, A_{nom} = 2.48 \text{ in}^2$

Median Eqn : $\check{T}_{BC} = 0.9\, \check{\sigma}_U\, A_{stress}$

10% Prob : $T_{BC_{10}} = \sigma_Y\, A_{nom}$

$$\beta_{U_{Eqn}} = \frac{1}{1.28} \ln\left[\frac{0.9\, \sigma_{UC} A_{stress}}{\sigma_{YC} A_{nom}}\right]$$

Parameter	Min Code	Median	β_U	Comments
σ_Y	36 ksi	44.0 ksi	0.12	Expected Properties A307 Bolts
σ_U	58 ksi	64.0 ksi	0.06	Expected Properties A307 Bolts
Eqn. Error		1.0	0.11	above equation
T_{BC}		143 K	0.13	

RPK STRUCTURAL MECHANICS CONSULTING

COMMENTS Fragility Method

Fluid Holddown Forces

$$T_{eo} = T_{eo_{CDFM}} \left(\frac{P_{T-}}{P_{T-CDFM}} \right) \quad \leftarrow \text{pg 4}$$

$$T_{e1} = 0.27 \frac{k}{in^2}$$

From App H, Section H.3.3

$T_{eo_{CDFM}} = 0.06$ k/in Pg H.20

$P_{T-CDFM} = 10.5$ psi Eqn H.25

$$\check{T}_{eo} = 0.06 \left(\frac{5.3}{10.5} \right) = 0.03 \frac{k}{in}$$

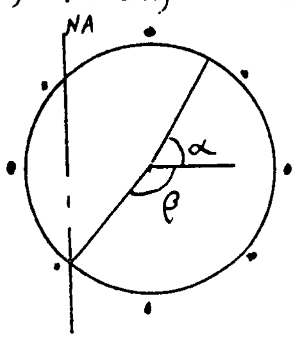
$\beta_{U_{T_{eo}}} = \beta_{U_{P_{T-}}} = 0.18$

$\beta_{R_{T_{eo}}} = \beta_{R_{P_{T-}}} = 0.38$

Overturning Moment Capacity : M_{SC}

1) CDFM Formulation for M_{SC} is estimated to be at 95% Confidence level and is too conservative for Median Capacity Estimate

2) Following Formulation is considered to be median centered.

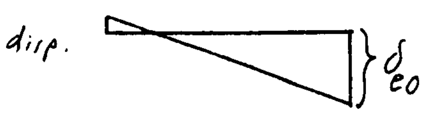


$$\delta e_{\alpha} = \delta_{eo} \left(\frac{\cos\alpha - \cos\beta}{1 - \cos\beta} \right)$$

$$\Delta T_e = \delta_{eo} T_{e1}$$

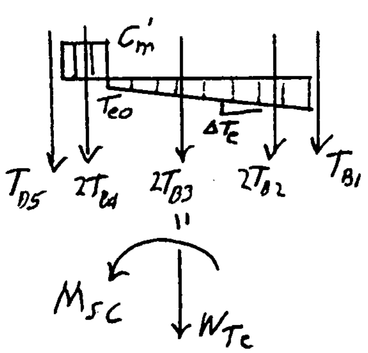
$$T_{B_i} = T_{BP} + K_B' \left(\frac{\cos\alpha - \cos\beta}{1 - \cos\beta} \right) \leq T_{BC} \qquad K_B' = \frac{\delta_{eo} A_B E_B}{(h_c + h_a)}$$

T_{BP} ← Pretension, if any



$$B_1 = \left[\frac{1}{(\pi - \beta)} \right] \qquad B_3 = \left(\frac{\sin\beta - \beta\cos\beta}{1 - \cos\beta} \right)$$

$$B_2 = 2 \sin\beta \qquad B_4 = \left(\frac{\beta - \sin\beta\cos\beta}{1 - \cos\beta} \right)$$



$$C_m' = B_1 \left[\frac{W_{Te} + \Sigma T_{B_i}}{2R} + T_{eo}\beta + \Delta T_e B_3 \right]$$

$$M_{SC} = C_m' R^2 B_2 + \Sigma (T_{B_i} R \cos\alpha_i) + T_{eo} R^2 B_2 + \Delta T_e R^2 B_4$$

trial & error: assume β, find C_m'
vary β until $C_m' = C_m$ (pg 5)

Find $\check{M}_{SC}$ by using all median properties in previous formulation

5% Strain Over Unanchored Length

$$\check{\delta}_{e0} = 0.05(h_c + h_a) = 0.05(24.75'' + 28.5'') = 2.66''$$

$$\check{K}_B' = 0.05(A_B)(E_B) = 0.05(3.14''^2)(29\times10^3 \text{ ksi}) = 4550^k$$

$\check{C}_m = 8.36 \text{ k/''}$ $\quad \check{T}_{Bc} = 143^k$ $\quad \check{T}_{e0} = 0.03 \frac{k}{''}$ $\quad \Delta \check{T}_e = 0.72 \frac{k}{''}$

$\check{W}_{T_e} = 53.6^k$ $\quad R = 20' = 240''$ $\quad T_{Bp} = 0$ (No Pretension)

β	$(1-\cos\beta)$	B_1	B_2	B_3	B_4	T_{B1} (k)	$2T_{B2}$ (k)	$2T_{B3}$ (k)	$2T_{B4}$ (k)	ΣT_B (k)	C_m' (k/'')	M_{SC} (k')
2.75	1.924	2.55	.763	1.519	1.613	143	286	286	286	1001	8.60	40,000
2.70	1.904	2.26	.855	1.506	1.621	"	"	"	"	"	7.60	39,800
2.65	1.882	2.03	.944	1.492	1.629	143	286	286	286	1001	6.80	39,400

$\therefore \check{M}_{SC} = 40{,}000$ $\qquad \check{SME} = \frac{40000}{35700}(0.60) = 0.67g$

Find M_{SC} Variabilities

① Set C_m to $-\beta_U$ Value: $C_{m_{-\beta_c}} = 8.36 \frac{k}{''} e^{-0.20} = 6.84$

All else unchanged

$M_{SC_{①}} = 39400^{k'}$ $\qquad \therefore \beta_{U_{C_m}} = \ln\left(\frac{40{,}000}{39400}\right) = 0.02$

Insensitive to C_m Variability

② Set T_{BC} to $-\beta_U$ Value, all else median: $T_{BC} = 143e^{-0.13} = 126^k$

β	$1-\cos\beta$	B_1	B_2	B_3	B_4	T_{B1}	$2T_{B2}$	$2T_{B3}$	$2T_{B4}$	ΣT_B	C_m'	M_{SC}
2.75	1.924	2.55	.763	1.519	1.613	126	252	252	252	882	7.97	37,400
2.80	1.942	2.93	.670	1.531	1.604	126	252	252	252	882	9.19	37,700

$M_{SC_{②}} = 37500$ $\qquad \therefore \beta_{U_{TB}} = \ln\left(\frac{40{,}000}{37500}\right) = 0.06$

Slightly sensitive to T_B

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③ Vary Fluid Holddown & Effective Wt Together since these parameters are not independent. All else median

$T_{eo} = 0.03e^{-0.42} = 0.02^k/_{''}$ $\quad W_{T_e} = 53.6^k e^{-0.09} = 49.0^k$

β	C_m'	M_{SC}
2.75	8.51	39,700
2.70	7.52	39,400

$M_{SC_{③}} = 39700$

$\beta_{R_{T_e}} = \ln\left(\frac{40000}{39700}\right) = 0.01$ Negligible

---

④ Investigate Equation Error

CDFM Formulation (Section H.3.4, App H) Assumed to be at 95% Confidence

Use median properties in CDFM Formulation

$\check{C}_m = 8.36^k/_{''} \qquad T_{BC} = 143^k \qquad T_{eo} = 0.03^k/_{''}$

$\check{\delta}_c = \frac{C_m h_c}{E_s t_s} = \frac{8.36(24.75)}{27.7\times10^3(0.375)} = 0.0199''$

$\Delta\check{T}_e = \check{T}_{e1}\check{\delta}_c = 0.27^k/_{''}(0.0199'') = 0.0054^k/_{''}$

$\check{K}_B = \frac{\delta_c A_B E_B}{(h_c + h_s)} = \frac{0.0199(3.14)(29\times10^3)}{(24.75+28.5)} = 34.0^k$

$\check{W}_{T_e} = 53.6' \qquad R = 20' = 240'' \qquad T_{BP} = 0$

β	$(1+\cos\beta)$	C_1	C_2	C_3	C_4	T_{B1}	$2T_{B2}$	$2T_{B3}$	$2T_{B4}$	ΣT_B	C_m'	M_{SC}
2.70	.0959	3.41	.575	102	32.2	143	286	286	139.7	854.7	7.28	26,000
2.75	.0757	3.84	.513	186	41.0	143	286	286	195.2	910.2	9.03	27,600

$M_{SC_{④}} = 27000$

$$\beta_{U_{EQN}} = \frac{1}{1.65}\ln\left(\frac{40000}{27000}\right) = 0.24$$

Equation is biggest source of uncertainty. Much greater uncertainty than that associated with structural strength

Variabilities & HCLPF Computation

Parameter	β_R	β_U	Comments
Response Me	0.20	0.10	Pg 1 of Calcs
Capacity			
Equation	—	0.24	pg 8
C_m Effect	—	0.02	pg 7
T_B Effect	—	0.06	pg 7
T_e Effect	0.01	—	pg 8
Total	0.20	0.27	

$$HCLPF = \check{SME}\, e^{-1.65(\beta_R + \beta_U)}$$

(SME ← pg 7)

$$HCLPF = 0.67 e^{-1.65(0.47)} = 0.31g$$

Excellent agreement with prior CDFM value from Appendix H of 0.29g. Therefore fragility method has produced reasonable results.

AUXILIARY CONTACTOR CHATTER IN MOTOR CONTROL CENTER

CDFM METHOD

By

R. P. Kennedy

TITLE MCC Aux Contactor Chatter

BY RPK DATE 9/2/87

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RPK STRUCTURAL MECHANICS CONSULTING

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COMMENTS ____ CDFM Method

First do for Cabinet At Grade

From Ref (3)

$$SME = \frac{TRS_{5\%}}{1.3\ \overline{AF}} \qquad \overline{AF} = \text{Aver. } S_{a_{5\%}} \text{ Over 20\% Frequency Bandwidth}$$

$TRS = GERS = 1.5g$ Ref (4) & Fig 5

$\overline{AF}_{5\%} = 2.12$ Ref (5) NUREG/CR 0098 ground 6.5 Hz

$\therefore$ CDFM at grade $SME = \frac{1.5g}{1.3(2.12)} = 0.54g$

Next Cabinet Mounted High in Structure (Fig 6 & 7 Floor Spectra)

① Find Average $S_{A_{5\%}}$ for Horiz. Floor Spectrum (Fig 6) over 20% Frequency Bandwidth Centered On Resonant Peak (6.7 - 8.2 Hz)

$\overline{S}_{a_{4\%}} = 2.28g \qquad \overline{S}_{a_{7\%}} = 1.61g \qquad \therefore \overline{S}_{a_{5\%}} = 1.99g$

② These $\overline{S}_a$ are for $SME_e = 0.18g$ since that was level used to generate floor spectrum.

$$\therefore \overline{AF}_{5\%} = \frac{\overline{S}_{a_{5\%}}}{0.18g} = \frac{1.99g}{0.18g} = 11.1$$

$\therefore$ CDFM high in struct. $SME = \frac{1.5g}{1.3(11.1)} = 0.10g$

AUXILIARY CONTACTOR CHATTER IN MOTOR CONTROL CENTER

FRAGILITY METHOD

By

R. P. Kennedy

TITLE MCC Aux. Contactor Chatter | PAGE 1 OF 3 Job No.
BY RPK DATE 9/25/87 | RPK STRUCTURAL MECHANICS CONSULTING | COMMENTS
CHKD. BY ____ DATE __/__/__ | Fragility Method

General Notes on Conservatism & Variability of GERS Data

① GERS are based upon broad spectrum inputs which are amplified through different cabinets in differing amounts and at differing frequencies so large variability in aux contactor chatter vs GERS

② GERS are not at HCLPF level. They represent a lower bound of a limited number of tests. Estimate aux contactor chatter GERS are 84% Confidence of 90% NEP for broad freq. input

③ GERS have greater conservatism and greater variability when subjected to narrow frequency input such as Figure 6 floor spectrum than they have for broad frequency input.

④ Cabinets amplify input at many frequencies in range of 5 to 15 Hz that are of prime interest for aux. contactor chatter.

⑤ Based upon a review of limited fragility data for aux. contactor chatter, the following estimates are made for the ratio of CAPACITY / GERS:

CAPACITY / GERS	Median	β_R	β_U	HCLPF
Broad Frequency Input Spectrum	1.45	0.11	0.23	0.83
Narrow Freq. Input Spectrum	1.74	0.11	0.25	0.96

Cabinet At GRADE

Parameter	Med	β_R	β_U	Comments
CAP/GERS	1.45	0.11	0.23	Broad Freq. Input
GERS/5% AF	0.71g	—	—	$= \frac{1.50g}{2.12}$ ← CR 0098
F_{GMH} - Ground Motion	1.22	0.20	—	Pg 1 of Flat Bottom Tank Fragility Calc. From F_1 and F_7 Table 1
Total SME	1.26g	0.23	0.23	

At Grade

$$HCLPF = \check{SME}\, e^{-1.65(\beta_R + \beta_U)}$$

$$HCLPF = 1.26g\, e^{-1.65(0.46)} = 0.59g$$

close agreement with CDFM estimate of 0.54g

Cabinet High in Structure

Must incorporate structure response variability factors due to ① F_F: Structure & Equipt. Freq. Variability (Combined $\beta_R \approx 0.35$ estimated)
② F_D: Structure Damping Uncertainty ($\beta_U \approx 0.35$)
③ F_C: Structure Modal & Earthquake Component Combination
④ F_M: Structure Modelling Variability

A) Generally these parameters would be estimated based upon a detailed review of the structure response analysis. However, this information is not available. Therefore, average values will be used based on a review of similar estimates from several higher quality PRA studies (heavily influenced by special studies performed on Diablo Canyon)

B) Above estimates will be made for case of equipment very close to resonant frequency of structure

C) Peak $S_{a_{5\%}} = 2.25g$ (↖Fig 6) for 0.18g Input ∴ $AF_{Peak} = \frac{2.25g}{0.18g} = 12.5$

$GERS/AF_P = 1.50g/12.5 = 0.12g$

High In Structure (Cont.)

Parameter	Median	β_R	β_U	Comment
CAP/GERS	1.74	0.11	0.25	Narrow Freq. Input Spectrum
GERS/AF_p	0.12g	—	—	pg 2
F_{GMH}	1.22	0.20	—	Ground Motion Parameter When SME defined as 84% NEP largest component response and failure governed by expected largest component response. See Pg 1 of Flat Bottom Tank Fragility Calcs. Obtained from F_1 and F_7 of Table 1
F_F	1.18	—	0.08	Frequency Variation Shifts Off of resonance but equipment vulnerable to many frequencies. Probable S_A is only about 85% of peak.
F_D	1.0	—	0.16	Median Centered Struct. Analysis but have damping uncertainty. Value given is for near resonance high in structure from variability studies
F_C	1.0	0.12	—	Values are for high in representative structure with good dynamic modelling
F_M	1.0	—	0.15	(see above)
SME	0.30g	0.26	0.34	

High In Structure

$$HCLPF = 0.30g\, e^{-1.65(0.60)} = 0.11g$$

Very close to CDFM estimate of 0.10g so fragility estimate appears to be reasonable

DIESEL GENERATOR ROOM STARTING AIR TANK SUPPORTS

CDFM METHOD

By

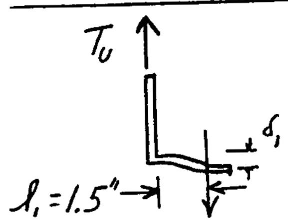
R. P. Kennedy

Air Tank Shown in Figure 2 is Mounted High In Structure With Floor Spectra Input Shown In Figs 6 & 7 for $SME_e = 0.18g$

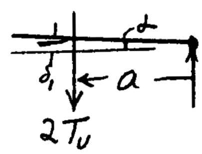
① Seismic Capacity is Controlled by Combined Base Angle Bending and Anchor Bolt Failure Mode. Only this failure mode is computed in these example CDFM calculations.

② Anchor Bolts are sufficiently embedded in concrete so that capacity is governed by steel and not by concrete

Horizontal Natural Frequency – Controlled by bending flexibility of base angles.



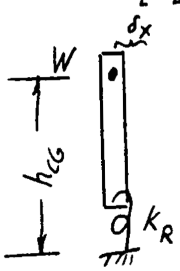
$$\delta_1 = \frac{T_U l_1^3}{3 E_L I_L}$$



$$\alpha = \frac{\delta_1}{a}$$

$$a = R_B (1 - \cos 100^\circ)$$

$$M = 2 T_U a$$



$$M = W \cdot h_{CG} = K_R \alpha$$

$$\delta_x = \frac{W h_{CG}^2}{K_R} \qquad \therefore K = \frac{K_R}{h_{CG}^2}$$

$$K_R = \frac{M}{\alpha} = \frac{(2a^2)(3 I_L E_L)}{l_1^3}$$

$$\therefore K = \frac{(2a^2)(3 E_L I_L)}{l_1^3 h_{CG}^2} \qquad f_H = \frac{1}{2\pi}\sqrt{\frac{Kg}{W}}$$

$W = 0.92^K$ $h_{CG} = 41.5''$ $l_1 = 1.5''$ $R_B \approx 12.25''$ $a \approx 14.4''$

$E_L = 29 \times 10^3$ ksi $I_L = \frac{3.0''(0.25'')^3}{12} = 3.91 \times 10^{-3}\, in^4$

$$\therefore K = \frac{(2(14.4'')^2)(3(29\times10^3 ksi)(3.91\times10^{-3} in^4))}{(1.5'')^3 (41.5'')^2} = 24.2\ ^K/_{in}$$

$$f_H = \frac{1}{2\pi}\sqrt{\frac{24.2\ ^K/_{in}\ (386\ ^{in}/_{sec^2})}{0.92^K}} = 16.0\ Hz$$

Vertical Frequency $- f_V > 33^{Hz}$

Determine Response Amplification Factors over Ground SME

Horiz: Damping = 5%

Fig 6 Floor Spectra is Unbroadened. To account for both structure and equipment frequency uncertainty, enter Fig 6 with a reduced frequency, f_H'

$$f_H' = 0.80(f_H) = 0.80(16.0^{Hz}) = 12.8\ Hz$$

$$S_{A_{12.8^{Hz},5\%}} = 0.60g$$

$$AF_H = \frac{0.60}{0.18 \leftarrow SME_e} = 3.33$$

Vert:

$$AF_V = \frac{0.20}{0.18} = 1.11$$

Base Moment Response

$$M_R = W \cdot h_{CG} \cdot AF_H \cdot SME = 0.92^K(41.5'')(3.33)SME = 127^{K''}SME$$

Effective Wt Base

$$W_E = W[1 - 0.4\,AF_V\,SME] = 0.92^K[1 - 0.4(1.11)SME]$$

$$W_E = 0.92^K[1 - 0.44\,SME]$$

TITLE Starting Air Tank Supports
BY RPK DATE 9/12/87
CHKD. BY ____ DATE / /

RPK STRUCTURAL MECHANICS CONSULTING

COMMENTS CDFM Method

Capacity Calcs

Bolts – 5/8" A307 Use 1.7 * Part 1 AISC Capacity (Eighth Ed)

$A_{nom} = 0.3068"$ V' = Shear on bolt to combine with tension

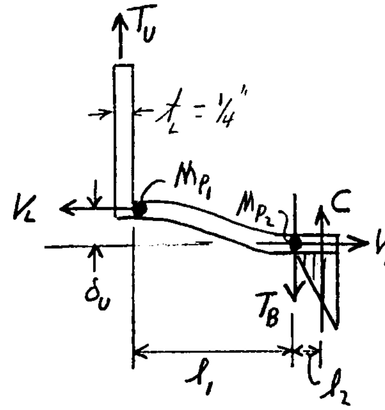
$$T_B = A_{nom}\left[1.7(26.0^{ksi}) - \frac{1.8V'}{A_{nom}}\right] \le T_{BC}$$

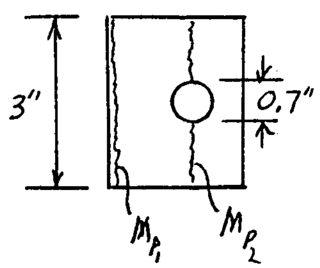
$$T_{BC} = 1.7(6.1^k) = 10.4^k$$

$$T_B = 13.6^k - 1.8\,V'$$

Use Whichever Less

Base Angle Will Yield & Go into membrain action with prying before bolt capacities are reached.





for CDFM

M_p = 90% Full Plastic Moment

$$M_{P_1} = 0.9\,\frac{36^{ksi}(3")(1/4")^2}{4} = 1.52^{k"}$$

$$M_{P_2} = 0.9\,\frac{36^{ksi}(2.3")(1/4")^2}{4} = 1.16^{k"}$$

$l_1 = 1.5"$ $l_2 \approx 0.5"$

$$T_U = T_B - C$$

$$C = M_{P_2}/l_2 = 1.16^{k"}/0.5" = 2.3^k$$

$$T_U = 11.3^k - 3.06\,V_L \le 8.1^k \qquad (1)$$

Also

$$T_U\,l_1 = (M_{P_1} + M_{P_2}) + V_L\,\delta_U = 2.68^{k"} + V_L\,\delta_U$$

$$T_U = 1.79^k + V_L\,\delta_U/1.5" \qquad (2)$$

Combine (1) & (2) to find T_U vs δ_U

$$T_U = \frac{11.3^k + \frac{4.82^{k"}}{\delta_U}}{\left[1 + \frac{2.7}{\delta_U}\right]}$$

δ_U	T_U
0	1.79^k
0.1"	2.13
0.2"	2.44
0.3"	2.74
0.4"	3.01^k

← for CDFM Calcs it is prudent to take no credit for δ_U

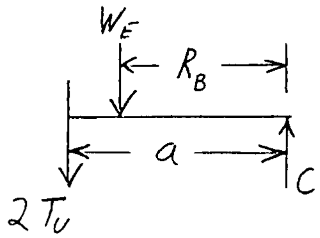
$\therefore T_U = 1.79^k$ ← controlled by angle plastic hinges

TITLE Starting Air Tank Supports
BY RPK DATE 9/12/87
CHKD. BY ____ DATE __/__/__

RPK STRUCTURAL MECHANICS CONSULTING

 Job No. ____
COMMENTS CDFM Method

Moment Capacity



$M_C = 2T_U\,a + W_E\,R_B = M_R$

$T_U = 1.79^k$ (pg 3) $W_E = 0.92^k[1 - 0.44\,SME]$ (pg 2)

$M_R = 127^{k''} \cdot SME$ (pg 2)

$a = 14.4''$ $R_B = 12.25''$ (pg 1)

$$SME = \frac{2(1.79^k)(14.4'') + 0.92^k(12.25'')}{127^{k''} + 0.92^k(12.25'')(0.44)}$$

SME = 0.48 g CDFM Method

DIESEL GENERATOR ROOM STARTING AIR TANK SUPPORTS

FRAGILITY METHOD

By

R. P. Kennedy

Fragility Calcs Follow Same Format As Previous CDFM Calcs

Response Factors

Horiz:

Frequency: CDFM Calcs produced a low estimate of natural frequency. Rocking in opposite direction is stiffer. Considering both uncertainty in structure & equipment frequency, the $\pm 1\beta_U$ frequency range to use in entering the unbroadened floor spectrum plot (Fig 6) is estimated to be:

$$f_{H \pm 1\beta_U} = 12.5 \text{ to } 24 \text{ Hz}$$

Equipt. Damping: 5% = Median; 7% = $+1\beta_U$; 4% = $0.66\beta_U$

From Fig 6: S_{A_H}

	12.5 Hz	24 Hz
4%	0.63 g	0.50 g
7%	0.60 g	0.48 g

$$\beta_{U_{ED}} = \frac{\ln\left(\frac{0.63}{0.60}\right)}{1.66} = 0.03$$

$$\beta_{U_F} = \frac{\ln(0.63/0.50)}{2} = 0.12$$

$$\check{S}_{A_H} = 0.55 g \qquad \beta_{U_{S_{AH}}} = \sqrt{0.12^2 + 0.03^2} = 0.12$$

$$\check{AF}'_H = \frac{S_{A_H}}{0.18g \leftarrow_{SME}} = \frac{0.55}{0.18} = 3.06$$

Vert: (Rigid)

Express AF'_V in terms of vert ground acc (0.12g)

(Fig 7) $$\check{AF}'_V = \frac{0.20}{0.12} = 1.67 \qquad \beta_U = 0$$

Must Add in Structural Response Variability As to how it will effect floor spectra away from resonant peak. Use estimates for high in structure away from resonance obtained from Diablo Canyon Variability Sensitivity Study.

Struct. Response Parameters	Median	β_R	β_U
Struct Damping	1.0	—	0.11
Modal & EQ Component Combination	1.0	0.10	—
Modelling Variability	1.0	—	0.15
F_{SR}	1.0	0.10	0.19

Combined Horiz Response AF_H

Parameter	Median	β_R	β_U	Comments
$1/F_{GMH}$	0.82	0.20	—	Pg 1 - Flat Bottom Tank Fragility Calc. (1/1.22) =
AF_H'	3.06	—	0.12	
F_{SR}	1.0	0.10	0.19	
AF_H	2.51	0.22	0.22	← Apply to SME

Combined Vert Response AF_V

Parameter	Median	β_R	β_U	Comments
Vert/SME	0.48	0.34	—	F_q from Table 1 of Text
AF_V'	1.67	—	0	
F_{SR}	1.0	0.10	0.19	
AF_V	0.80	0.35	0.19	← Apply to SME

TITLE Starting Air Tank Supports | BY RPK DATE 9/25/87 | CHKD. BY ____ DATE __/__/__

RPK STRUCTURAL MECHANICS CONSULTING

COMMENTS Fragility Method

Base Moment Response $W = 0.92^k$ $h_{CG} = 41.5''$

$$M_R = W \cdot h_{CG} \cdot AF_H \cdot SME = 38.2^{k''} \cdot AF_H \cdot SME$$

Eff. Base Wt.

$$W_E = W\left[1 - \overbrace{F_{VC} \cdot AF_V}^{F_V} \cdot SME\right] = 0.92^k\left[1 - F_V \cdot SME\right]$$

Parameter	Median	β_R	β_U	Comments
F_{VC}	0.40	0.39	—	Vertical Component Combination Factor
AF_V	0.80	0.35	0.19	
F_V	0.32	0.53	0.19	

Bolt Capacity 5/8" A307 $A_{nom} = 0.3068''^2$ $A_{stress} = 0.2260''^2$

$$T_{BC} = F_B \cdot T_{BC_{CDFM}} = 0.9 \cdot \sigma_U \cdot A_{stress}$$

$$\therefore F_B = \frac{0.9 \sigma_U A_{stress}}{T_{BC_{CDFM}}}$$ $T_{BC_{CDFM}} = 10.4^k$ (pg 3 CDFM Calc)

10% Prob: $T_{BC} = \sigma_y \cdot A_{nom}$ $\therefore \beta_{EQN} = \frac{1}{1.28} \ln\left[\frac{0.9\, \sigma_{UC}\, A_{stress}}{\sigma_{YC}\, A_{nom}}\right]$

Parameter	Median	β_U	
σ_{YC}	36^{ksi}	—	
σ_{UC}	58^{ksi}	—	
σ_U	64.0^{ksi}	0.06	
Eqn Error	1.0	0.05	From β_{EQN}
F_B	1.25	0.08	← Also Applies to Interaction Eqn

Angle Bending

Plastic Moment Factor

$$M_p = F_{M_p} \cdot M_{P_{CDFM}}$$

Parameter	Median	β_U	
σ_y / σ_{yc}	1.22	0.12	A36 Steel
EQN	1.11	0.06	$(1/0.9) = 1.11$
F_{M_p}	1.35	0.13	

Find Uplift Capacity T_U versus Uplift δ_U (see pg 3 CDFM Calcs)

$T_U = T_B - C$: $\therefore T_U = 13.6^k \cdot F_B - 1.8 V_L F_B - 2.3^k F_{M_p}$

From Moment Balance: $T_U = 1.79^k F_{M_p} + V_L \delta_U / 1.5''$

} combine to find T_U vs δ_U

$$T_U = \frac{13.6 F_B - 2.3 F_{M_p} + \frac{4.82 F_{M_p} F_B}{\delta_U}}{\left[1 + \frac{2.7 F_B}{\delta_U}\right]}$$

① Use $\check{F}_B = 1.25$ $\check{F}_{M_p} = 1.35$

δ_U	T_U
0	2.42^k
0.25"	3.20^k
0.50"	3.89^k
0.75"	4.49^k
1.0"	5.03^k

$\check{\delta}_U = 0.3''$ Estimate

$\therefore \check{T}_U = 3.34^k$ $\beta_{U_\delta} = 0.14$

$T_{U_{95\%}} = 2.65^k$

② Find Influence of F_{MP} Variability

$\check{\delta}_U = 0.3''$ $\check{F}_B = 1.25$ $F_{M_{P-\beta}} = 1.35 e^{-0.13} = 1.19$

$\therefore T_{U_{②}} = 3.12$

$$\beta_{M_p} = \ln\left(\frac{3.34}{3.12}\right) = 0.07$$

③ Find Influence of F_B Variability

$\check{\delta}_U = 0.3''$ $\check{F}_{M_p} = 1.35$ $F_{B_{-e}} = 1.25e^{-0.08} = 1.15$

$\therefore T_{U_{③}} = 3.30$ $\beta_{U_B} = \ln\left(\frac{3.34}{3.30}\right) = 0.01$

Parameter	Median	β_U
T_U	3.33	—
β_δ	—	0.14
β_{M_p}	—	0.07
β_B	—	0.01
T_U	3.33	0.16

Moment Capacity (pg 4 CDFM Calcs)

$M_C = 28.8'' T_U + 11.3^{k''}[1 - F_V \cdot SME]$

$M_R = 38.2^{k''} \cdot AF_H \cdot SME$ (pg 3)

$M_C = M_R$

$$\therefore SME = \frac{28.8'' T_U + 11.3^{k''}}{38.2^{k''} AF_H + 11.3^{k''} F_V}$$

Parameter	Median	β_R	β_U	Comments
T_U	3.34^k	—	0.16	
AF_H	2.51	0.22	0.22	
F_V	0.32	0.53	0.19	
SME	1.07g	0.21	0.25	From Simulation, Best Fit lognormal distribution

$HCLPF = 1.07g\, e^{-1.65(0.46)} = 0.50g$ — Very close to CDFM Value of 0.48g so fragility results reasonable

COMPONENT COOLING HEAT EXCHANGER SUPPORTS

CDFM METHOD

By

R. P. Kennedy

TITLE Heat Exchanger Supports
BY RPK DATE 9/3/87
CHKD. BY ____ DATE __/__/__

RPK STRUCTURAL MECHANICS CONSULTING

PAGE 1 OF 2 Job No. ____
COMMENTS CDFM Method

Heat Exchanger Shown in Fig 3 is Mounted High in Structure With Floor Spectra Input Shown in Figures 6 & 7 for $SME_c = 0.18g$

① Seismic Capacity is Controlled by Longitudinal Shear on bolts and Only this failure mode capacity is shown in these example CDFM calculations

② Heat Exchanger is Bolted to Steel Support frame which is assumed to be rigid

Horizontal & Vertical Frequencies Exceed 33 Hz

Determine Horiz & Vertical Response Amplification Factor

$$S_{A_H} = AF_H \cdot SME \qquad S_{A_V} = AF_V \cdot SME$$

$$AF_H = \frac{0.38}{0.18} = 2.11 \text{ (Fig. 6)} \qquad AF_V = \frac{0.20}{0.18} = 1.11 \text{ (Fig. 7)}$$

Determine Base Response Per Saddle

Longitudinal (one saddle has slotted bolt holes)

$$V_{long} = W \cdot AF_H \cdot SME = 23.5^k(2.11) \cdot SME = 49.6^k \cdot SME$$

Lateral

$$V_{lat} = \frac{W \cdot AF_H \cdot SME}{2} = 24.8^k \cdot SME$$

$$M_{lat} = V_{lat} \cdot h_{CG} = 24.8^k(25'') \cdot SME = 620^{k''} \cdot SME$$

Vector Sum Shear Per Bolt

Combine 100% Long With 40% Lat & 40% Vert

$$V_{RB} = \frac{1}{2}\sqrt{(V_{long})^2 + (0.4V_{lat})^2} = 25.3^k \cdot SME$$

(1/2 ← two bolts)

TITLE Heat Exchanger Supports | BY RPK | DATE 9/13/87 | CHKD. BY ___ DATE __/__/__

RPK STRUCTURAL MECHANICS CONSULTING

COMMENTS: CDFM Method

Vertical

$$W_e = \frac{W}{2}\left[1 - 0.4\,AF_v\,SME\right] - V_{long}\left(\frac{h_{cG}}{s}\right)$$

$$W_e = \frac{23.5^k}{2}\left[1 - 0.4(1.11)SME\right] - 49.6^k\left(\frac{25''}{216''}\right)SME$$

$$W_e = 11.75^k - 10.96^k\,SME$$

Capacity

First, assume bolt capacity controlled by shear. Will subsequently check combined shear and tension

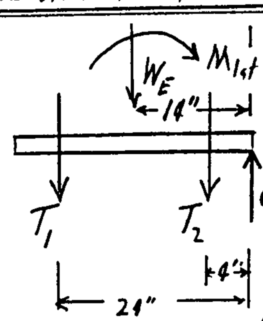
V_C = 1.7 * AISC Part 1 Capacity (Eighth Edition)

for 7/8"ϕ - A307 bolts

$$V_c = 1.7(6.0^k) = 10.2^k$$

$$\therefore V_c = V_{RB} : \quad SME = \frac{10.2^k}{25.3^k} = 0.40$$ CDFM Capacity

Check Tension Interaction for SME = 0.40g



$$T_1(24'') + T_2(4'') + W_E(14'') = M_{lat}$$

$$T_2 = \left(\frac{4}{24}\right)T_1$$

$$\therefore T_1 = \frac{M_{lat} - W_E(14'')}{24'' + \frac{(4'')^2}{24''}} = \frac{M_{lat} - 14''W_E}{24.67''}$$

40% lateral at time of max shear

$$M_{lat} = 620^{k''}(0.40g)(0.40) = 99.2^{k''} \quad W_E = 11.75^k - 10.96^k(0.40) = 7.37^k$$

$$T_1 = \frac{99.2^{k''} - 7.37^k(14'')}{24.67''} < 0 \quad \text{No Tension}$$

With Full Shear V_c Present: $T_C = 1.7(26^{ksi})(0.6'') - 1.8(10.2^k) = 8.16^k$ } doesn't control

Should Also Generally Check 100% Lateral + 40% Long + 40% Vert. However Won't

COMPONENT COOLING HEAT EXCHANGER SUPPORTS

FRAGILITY METHOD

By

R. P. Kennedy

TITLE Heat Exchanger Support | PAGE 1 OF 2 Job No.
BY RPK DATE 9/26/87 | **RPK STRUCTURAL MECHANICS CONSULTING** | COMMENTS
CHKD. BY ____ DATE __/__/__ | Fragility Method

① SME Capacity of Heat Exchanger Is Not Governed By Largest Horiz Response Component. It is Governed By Longitudinal Response Component. SME is defined as 84% NEP Largest Horiz Response.

$$\frac{\text{Long Resp}}{\text{84\% Largest Horiz}} = \frac{F_1 \cdot F_2}{F_6} \qquad \frac{\text{Lat Response}}{\text{84\% Largest Horiz}} = \frac{F_1}{F_2 F_6}$$

$$\frac{\text{Vert Resp}}{\text{84\% Largest Horiz}} = F_9$$

where F_1, F_2, F_6, and F_9 are all defined in Table 1 of introductory text

② Must Also Include Structural Response Variability for High in Structure Away From Resonance. Use F_{SR} derived on page 2 of fragility method calcs for starting air tank supports.

③ Since freq > 33 Hz, $AF_H = 2.11$ used in CDFM Calcs without variability

④ ∴ Vector Sum Shear Per Bolt (see pg 1 of CDFM Calcs)

$$V_{RP} = \frac{49.6^k}{2}\left(\frac{F_1 F_{SR}}{F_6}\right) SME \underbrace{\sqrt{(F_2)^2 + \left(\frac{F_{EC}}{2F_2}\right)^2}}_{F_X}$$

(2 ← two bolts)

where F_{EC} is eq component combination variable

Parameter	Median	β_R	β_U	Comments
F_1	1.0	0.20	—	From Table 1
F_2	1.0	0.15	—	
F_6	1.41	—	—	
F_{SR}	1.0	0.10	0.19	Pg 2 Starting Air Tank Supports
F_{EC}	0.40	0.39	—	
F_X	1.02	0.14	—	Best fit lognormal distribution from simulation
V_{RP}/SME	17.9^k	0.26	0.19	

Bolt Capacity (assume treads are in shear plane)

7/8" - A307

$V_C = 0.55 \, A_{Nom} \, \sigma_U$ $\sigma_{UC} = 58^{ksi}$ $\sigma_{YC} = 36^{ksi}$ $A_{nom} = 0.6$

Eqn Error: $\beta_{EQN} = \frac{1}{1.65} \ln\left(\frac{\sigma_{UC}}{\sigma_{YC}}\right) = 0.29$

Parameter	Median	β_U
σ_U	64^{ksi}	0.06
EQN	1.0	0.29
V_C	21.1^K	0.30

$SME = V_C / (V_{RP}/SME)$

Parameter	Median	β_R	β_U
(V_{RP}/SME)	17.9^K	0.26	0.19
V_C	21.1^K	—	0.30
SME	1.18g *	0.26	0.36

$HCLPF = 1.18 \, e^{-1.65(0.62)} = 0.42g$

Very close to CDFM value of 0.40g so fragility results are reasonable

* In a fragility study, one should check other failure modes to insure they do not result in a lesser median capacity. The failure mode which was checked controls the HCLPF capacity but possibly not the median.

CANTILEVER REINFORCED BLOCK WALL

CDFM METHOD

By

R. P. Kennedy

TITLE Block Wall — BY RPK DATE 9/13/87 — CHKD. BY ____ DATE _/_/_

RPK STRUCTURAL MECHANICS CONSULTING

COMMENTS: CDFM Method

Cantilever Block Wall Shown in Fig 4 is Mounted High in Structure & Subjected to Floor Spectra Input Shown in Fig 6 & 7 for $SME_e = 0.18g$

① Seismic Capacity is Governed by Out-of-Plane Flexural Behavior about wall base and this is the only failure mode checked in these calcs.

② Wall is evaluated using strength design method.

Concrete Block Properties

$f_c' = 3000\,psi$ Type S Mortar

∴ From ACI 531-79: $\begin{cases} f_m' = 1700\,psi \text{ (masonry comp strength)} \\ E_m = 1.7\times10^6\,psi \end{cases}$

Tensile Strength: $f_R = 2.5\sqrt{f_m'} = 103\,psi$ {Ref: Amrhein, "Reinforced Masonry Engineering Handbook," Fourth Edition, pg 117

Gross Section: $I_g = 1340\,in^4/ft$ $S_g = 231\,in^3/ft$

Cracked Section: $I_{CR} = 86\,in^4/ft$

Weight: $w = 111\,psf$ Ht: $h = 10^{ft}$

Natural Freq Gross Section: $f_g = 12\,Hz$

Cracking Moment Capacity

$$M_{CR} = f_R S_g = 0.103\,ksi\,(231\,in^3/ft) = 23.8^{k\,in}/1$$

Ultimate Moment Amrhein, Pg 115

Special Inspection

$$M_U = \phi A_s f_y \left[d - \frac{a}{2}\right] \qquad a = \frac{A_s f_y}{0.85 f'_m b} \qquad \phi = 0.80$$

$$A_s = 0.233''^2/' \qquad f_y = 60^{ksi} \qquad d = 5.81''$$

$$\therefore \quad a = \frac{0.233(60)}{0.85(1.7)(12''/')} = 0.81''$$

$$M_U = 0.8(0.233)(60)\left[5.81 - \frac{0.81}{2}\right] = 60.4^{k''/'}$$

Determine I_{eff} & Elastic freq. f

$$I_{eff} = \left(\frac{M_{CR}}{M_U}\right)^3 I_g + \left[1 - \left(\frac{M_{CR}}{M_U}\right)^3\right] I_{CR} \qquad \left(\frac{M_{CR}}{M_U}\right)^3 = \left(\frac{23.8}{60.4}\right)^3 = 0.0612$$

$$I_{eff} = (0.0612)(1340''^4/ft) + (0.9388)(86''^4/ft) = 163''^4/'$$

$$f = f_g \sqrt{I_{eff}/I_g} = 12 Hz \sqrt{\frac{163}{1340}} = 4.2^{Hz}$$

However wall will go substantially inelastic before it fails and response will be governed by an effective inelastic frequency f_e'. NUREG/CR-3805, Vol. 1, May 1984 (Kennedy, et. al.) will be used to estimate f_e' and inelastic spectral response

① Estimate Inelastic Base Rotation Capacity During EQ

Non-Cyclic Base Rotation Capacity r_U given by ACI 349, App. C

$$r_U = (0.0065)\left(\frac{0.85d}{a}\right) = 0.040 \text{ radians}$$

However, under 3 to 5 strong response cycles from EQ should limit r_U to lesser value.

$$r_U' = \tfrac{2}{3} r_U = \tfrac{2}{3}(0.040) = 0.026 \text{ radians}$$

② Determine Elastic & Inelastic deformation of wall CG.

$$\text{Elastic:} \quad \delta_{e_{CG}} = \frac{w h^4}{22.6 E I_{eff}} = \frac{0.111^{k}/_{12} (10')^4 (144^{in^2}/_{ft^2})}{22.6 (1.7\times10^3 ksi)(163^{in^4}/_{1})} = 0.026'$$

$$\text{Inelastic:} \quad \delta_{CG} = \delta_{e_{CG}} + r_u'\left(\frac{h}{2}\right) = 0.026' + 0.026(5') = 0.156'$$

③ Determine Secant & Eff. Frequencies & Eff. Damping

$$f_s = f\sqrt{\delta_{e_{CG}}/\delta_{CG}} = 4.2\,Hz\sqrt{\frac{0.026}{0.156}} = 1.7\,Hz$$

$$f_e' = 0.15 f + 0.85 f_s = 0.15(4.2) + 0.85(1.7) = 2.1\,Hz$$

$$\beta_e' \approx 10\%$$

④ Determine Eff S_A

$$S_{A_{eff}} = K \cdot S_{A\,(f_e',\,\beta_e')}$$

where K is an inelastic response reduction factor.

$$K = \left[f_s / f_e' \right]^2 = 0.67$$

However Ref (3) limits $K \geq 0.8$ for the CDFM Method unless a nonlinear response analysis is performed.

⑤ Account for Freq Uncertainty

The Fig 6 Floor Spectra is unbroadened and must be frequency shifted to account for structure frequency uncertainty. Furthermore, f_e' is uncertain & should be freq. shifted to account for uncertainty. To account for both, use highest spectral acceleration within range 0.7 to 1.4 times f_e'

$$f_e = 1.5\,Hz \text{ to } 2.9\,Hz$$

⑥ Determine AF_H

$$S_{A(2.9Hz,\ 10\%)} = 0.33g \quad \text{from Fig 6 for } SME_e = 0.18g$$

$$S_{A_{eff}} = 0.8(0.33g) = 0.26g$$

$$AF_H = \frac{S_{A_{eff}}}{0.18g} = \frac{0.26g}{0.18g} = 1.47$$

⑦ Determine Base Moment, M_R

$$M_R = \frac{wh^2}{2} AF_H \cdot SME = \frac{0.111^{KSF}(10')^2(12''/1')}{2}(1.47)SME = 97.9^{K''/1} \cdot SME$$

⑧ Determine SME

$$SME = \frac{M_U}{(M_R/SME)} = \frac{60.4^{K''/1}}{97.9^{K''/1}} = 0.62g$$

CDFM

CANTILEVER REINFORCED BLOCK WALL

FRAGILITY METHOD

By

R. P. Kennedy

TITLE Block Wall | PAGE 1 OF 3 Job No. ____
BY RPK DATE 9/26/87 | RPK STRUCTURAL MECHANICS CONSULTING | COMMENTS ____
CHKD. BY ____ DATE __/__/__ | Fragility Method

Moment Capacity M_U

$$M_U = F_\phi \cdot A_s f_y \left[d - F_a \frac{a}{2}\right] \qquad a = \frac{A_s f_y}{0.85 f_m' b} \qquad A_s = 0.233''^2/'$$

Parameter	Med	β_U	Comment
f_y	67 ksi	0.08	Grade 60 Rebar
f_m'	2.2 ksi	0.13	
d	5.81"	0.05	
F_a	0.75	0.18	Keller & Suter, 1982 for Masonry
F_ϕ	1.0	0.11	
a	0.81"	0.15	derived from above
$[d - F_a \frac{a}{2}]$	5.51"	0.05	" " "
M_U	77.0 k"/'	0.14	

Rotational Capacity

$$r_U' = F_S F_{r_U} (0.0065)\left(\frac{0.85d}{a}\right)$$

Parameter	Med	β_U	Comments
F_S	0.80	0.12	Seismic Cyclic Reduction Factor
F_{r_U}	1.40	0.32	Eqn Variability Factor
d	5.81"	0.05	
a	0.81"	0.15	
r_U'	0.044	0.38	

TITLE Block Walls		PAGE 2 OF 3 Job No.
BY RPK DATE 9/26/87	RPK STRUCTURAL MECHANICS CONSULTING	COMMENTS
CHKD. BY DATE / /		Fragility Method

Elastic & Inelastic Deformation Uncertainty & Freq. Unc.
(based on values & equations used in CDFM evaluation)

Parameter	Med	β_U		Comments
Elastic $\delta_{e_{CG}}$	0.026'	0.50		Pg 3 CDFM Calcs.
Inelastic δ_{CG}	0.251'	0.34		Eqn on Pg 3 CDFM Calcs with f_U' from fragility calc pg 2 by simulation
f_s	1.35 Hz	0.17		Eqn on Pg 3 CDFM Calcs
f	4.2 Hz	0.25		" " " "
f_e'	1.80 Hz	0.14		Assume f_s & f are independent. Obtain by simulation using Eqn on Pg 3 of CDFM method
β_e'	11.5%	0.17		
K	0.563	0.22		f_s & f_e' are closely dependent
f_e	1.80 Hz	0.29		Broaden f_e' uncertainty to account for struct. frequency uncertainty when using unbroadened floor spectra. $\sqrt{(.25)^2+(.14)^2} = 0.29$
$S_{A f_e, \beta_e'}$	0.27	0.15		From Fig 6 considering f_e and β_e' uncertainty
$S_{A_{eff}}$	0.15	0.27		Eqn Pg 3 CDFM Calcs.
$AF_H = \frac{S_{A_{eff}}}{0.18g}$	0.84	0.27		
			β_R	
Random Scatter	1.0	—	0.15	NUREG/CR3805
AF_H	0.84	0.27	0.15	

Incorporate Structural Response & Ground Motion Factors

① SME Capacity of Block Wall is governed by floor response out-of-plane of wall & not by largest horiz response component. SME is defined as 84% NEP largest horiz. response

$$\frac{\text{Out-of-plane response}}{\text{84\% NEP largest horiz}} = \frac{F_1 F_2}{F_6}$$

F_1, F_2, F_6 from Table 1 of introductory text.

② Must also include structural response variability for high in structure away from resonance. Use F_{SR} derived on page 2 of fragility calcs for starting air tank supports.

③ Moment Response & SME

$$(M_R/SME) = \frac{M_{R\,CDFM}}{AF_{CDFM}} \cdot AF_H \cdot F_{SR} \cdot \left(\frac{F_1 F_2}{F_6}\right)$$

(pg 4 CDFM = 66.6 k"/")

$$SME = M_U / (M_R/SME)$$

Parameter	Median	β_R	β_U
M_U	77.0 k"/"	—	0.14
$(M_R/AF)_{CDFM}$	66.6 k"/"	—	—
AF_H	0.84	0.15	0.27
F_{SR}	1.0	0.10	0.19
F_1	1.0	0.20	—
F_2	1.0	0.15	—
F_6	1.41	—	—
SME	1.94g	0.31	0.33

$$HCLPF = 1.94g\, e^{-1.65(0.64)} = 0.67g$$

Close agreement with CDFM value of 0.62g

APPENDIX B

EQE ENGINEERING, INC.

SUMMARY OF HCLPF CAPACITY CALCULATIONS FOR SELECTED COMPONENTS PERFORMED BY EQE INCORPORATED

Component 1: Flat Bottom Storage Tank

Failure of the vertical storage tank is defined to be gross loss of fluid contents. This is assumed to occur when the tank shell buckles. Horizontal seismic load initiates uplift of the tank shell from its foundation. This uplift is resisted by the anchor bolts, the tank bottom plate, and the tank weight. The anchor bolts, with the modified chair detail, are permitted to yield, so long as their behavior is ductile, since yielding does not directly result in loss of fluid contents. Shell compressive stresses progressively increase until buckling occurs.

The following HCLPF capacities have been obtained:

Fragility Analysis Method	:	0.26 g
CDFM Method	:	0.29 g
Median Capacity	:	0.54 g

These estimates were not revised in the Second Round Calculations following the Study Group discussion.

Component 2: Motor Control Center

The cabinet has a fundamental frequency of 6.5 Hz which is a typical electrical cabinet frequency that results in significant amplification of base motion input. The functional failure mode of the motor control center is governed by chatter of auxiliary contactors on the motor starters. This may result in spurious signals and may adversely affect the equipment controlled by the motor starter. The median fragility was derived using relay and contactor chatter data evaluated during the SSMRP program [Cover et al 1985]. However, the uncertainties for that fragility derivation were large due to the fact that some non-typical very sensitive devices were included in the data base. Therefore, Generic Equipment Ruggedness Spectra (GERS) [ANCO, 1987] were used to establish a lower bound capacity and the uncertainty on strength was derived from the acceleration capacity range between the median and GERS capacity. The HCLPF was then derived from the median capacity and the uncertainties in capacity, building response and equipment response. The deterministic HCLPF capacity was derived using the EPRI methodology and GERS. GERS were reduced by a factor of 1.3 in accordance with the EPRI Seismic Margin Criteria Methodology. In addition, operability during an earthquake is only a concern if there are interlock circuits. Therefore, the 0.87 reduction factor on GERS for interlock circuits, as stated in the GERS report [ANCO, 1987], was used to reduce the HCLPF capacity.

An additional failure mode investigated was operability after the earthquake. ANCO GERS and other generic test data have been used for this HCLPF capacity derivation. Two locations for the cabinet were studied: ground level and high-up in the building. The following HCLPF capacities have been calculated for the MCC:

Case	Median Capacity (g)	HCLPF Capacity (g)	
		Fragility Method	CDFM Method
In-Structure			
Function During	0.36	0.07	0.09
Function After	1.16	0.21	0.26
Ground Mounted			
Function During	1.58	0.39	0.47
Function After	5.06	1.18	1.45

These estimates were not revised in the Second Round Calculations.

Component 3: Starting Air Receiver Tank

The starting air receiver tank is a vertical, skirt supported cylindrical tank which is anchored to the building floor by three angles welded to the tank skirt and bolted to the floor. The leg of the angle is much weaker in bending than the anchor bolts. The angle leg is very ductile in bending and the failure mode is low cycle fatigue. When fracture of the angle occurs, the air tank is assumed to fail through failure of the attached piping.

A low cycle fatigue analysis was conducted to determine the ductility limit at failure, assuming 5 cycles of strong motion input. The resulting failure ductility was used in the fragility analysis to determine the median acceleration capacity. The HCLPF capacity was then computed from the median capacity and the derived uncertainties of the important variables that contribute to building response, equipment response, strength and ductility.

In the EPRI deterministic analysis method, bending of the angle leg was also the governing failure mode. In this case the computed bending stress was compared to the ASME Component Support Code allowable stress for plate bending. In addition, a load factor of 0.8 to account for ductility was used to reduce the seismic load before comparison to the code allowable. This load factor is specified in the EPRI deterministic criteria for ductile failure modes.

This tank is assumed to located high up in the building. Since the nozzle loads were not specified, the analysis has not included any effect of nozzle loads. The following HCLPF capacities have been calculated:

Fragility Analysis Method	:	0.44 g
CDFM Method	:	0.53 g
Median Capacity	:	1.55 g

These estimates were not revised in the Second Round Calculations.

Component 4: Horizontal Heat Exchanger

The failure mode governing the median capacity of the horizontal heat exchanger was combined tension and shear of the anchor bolts. Tension results from overturning of the heat exchanger in the lateral direction while shear results from inertial loads in both horizontal directions. When this anchorage failure occurs, the heat exchanger is assumed to fail through failure of the nozzles and attached piping. The HCLPF capacity was computed from this median capacity and the derived uncertainties in building response, equipment response and strength.

The governing failure mode for the EPRI deterministic HCLPF capacity calculation was pure shear of the anchor bolts. This change in governing failure mode results from the fact that dead weight resists the overturning and tensile stresses do not develop in the bolts at the HCLPF capacity level but do develop at the median capacity level. The difference in the calculated HCLPF is small, however, for the two failure modes of the anchor bolts.

Two locations for this heat exchanger have been studied: ground mounted and high-up in the building. Since nozzle loads were not specified, they have not been considered. The following HCLPF capacities have been calculated:

Case	Median Capacity (g)	HCLPF Capacity (g)	
		Fragility Method	CDFM Method
Ground Mounted	1.87	0.89	0.96
In-Structure	1.08	0.38	0.44

These estimates were not revised in the Second Round Calculations.

Component 5: Reinforced Block Wall

The block wall is represented as a vertical cantilever fixed at its base. Seismic capacity is controlled by out of plane bending moment at the base of the wall. The wall is capable of withstanding seismic excitation levels in excess of those causing initial yielding through ductile response. Failure occurs when the ductility demand reaches a maximum permissible value. This maximum allowable ductility corresponds to a deformation level at which the load carrying capacity begins to significantly degrade.

The sample block wall ultimate strength and load-deflection relationship is determined following the recommendations of the ACI-SEASC Task Committee on Slender Walls. The capacities are estimated using two different stiffness assumptions: equivalent elastic-plastic load-deflection curve and secant stiffness. The following HCLPF capacities have been calculated for block wall:

Fragility Analysis Method: 0.48 g

CDFM Method : 0.48 g

Median Capacity : 1.55 g

In the Second Round Calculations, median damping of 10 % was used along with a median ductility of 3. The revised capacity estimates are:

HCLPF Fragility Analysis Method: 0.63 g

HCLPF CDFM Method : 0.63 g

Median Capacity : 2.10 g

References

1. Cover, L. E., M. P. Bohn, R. D. Campbell, D. A. Wesley, "Handbook of Nuclear Power Plant Fragilities", Seismic Safety Margins Research Program, NUREG/CR-3558, June 1985

2. ANCO, "Generic Seismic Ruggedness of Power Plant Equipment", Prepared by ANCO Engineers for Electric Power Research Institute, EPRI NP-5223, May, 1987

COMPONENT 1

FLAT BOTTOM STORAGE TANK

EQE

ENGINEERING, PLANNING AND MANAGEMENT CONSULTANTS

SHEET NO. ______

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-30-87

CLIENT WNC SUBJECT Tank CHK'D MKR DATE 10-5-87

TANK HCLPF CAPACITY

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SHEET NO. T-1

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-30-87

CLIENT LLNL SUBJECT Tank CHK'D MKR DATE 10-5-87

Summary

The tank HCLPF Capacity is determined following the Maine Yankee RWST calculations (Ref. 1), with revisions incorporated to account for the revised anchor bolt chairs. The tank HCLPF capacity is calculated to be:

Fragility analysis - 0.26g

CDFM - 0.29g

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SHEET NO. T-2

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-30-87

CLIENT LLNL SUBJECT Tank CHK'D MKR DATE 10-5-87

Median Seismic Response

From Maine Yankee RWST calcs (Ref. 1):

Impulsive mode frequency = 5.6 Hz — p. 5, Ref. 1

Sloshing mode frequency = 0.273 Hz — p. 11, Ref. 1

Total Base shear = 1295 k
Total Base moment = 233,800 k·in
Total Base axial load = 5.08 k
} p. 20, Ref. 1

Median Base Moment Capacity

The tank base moment capacity will be determined using the same approach developed for the Maine Yankee RWST. With the new bolt chairs (Ref. 2), the bolts control the tank capacity.

Anchor Bolt Capacity - 2"Φ A307 A.B.

Bolt yield force governs w/ new chairs installed.

Yield On Nominal Area

Min. yield stress = 36 ksi

Estimate $\check{f}_y \approx 44$ ksi — Typical for A36 steel

$A_b = 3.14\ in^2$

$\check{P}_y = 44\,(3.14)$

$= 138\ k$

Tank Axial Load

Estimate $\check{A} \approx 0.5g$

$P_{DL} = 63.7^k$ p.3, Ref. 1

$P_{EQ} = 5.08^k$ p.20, Ref. 1 For 0.3g horiz. PGA

$P_{VERT} = -63.7 + 5.08\left(\frac{0.5}{0.3}\right)$

$= -55.2^k$

Bottom PL Holdown Force

Estimate $\check{A} = 0.5g$

See p. 68, Ref. 1

$S_{av} = 0.378g$ For a 0.3g horiz. PGA, p.16, Ref. 1

$S_{av} = 0.378\left(\frac{0.5}{0.3}\right)$

$= 0.63g$

$w = 62.4(37)[1 - 0.4(0.63)]$

$= 1727$ psf

$= 0.0120$ ksi

Anchor Bolt Dimensions – See p. B4, Ref. 1

h_1 = Dimension from top of chair to bottom of tank

$= 0.75 + 24 + 0.75 + 1.5$ (from tank bottom to bottom of lower chair plate)

$= 27''$ vs. 11.5" old chairs

h_2 = Dimension from top of chair to top of anchor PL

$= 39 + (27 - 11.5)$ 39" = h_2 for old chair

$= 54.5''$

Program Input

Program TANKER calculates the tank capacity following the derivation in App. B, Ref. 1. See p. 70, Ref. 1 for other input.

R = Tank radius = 240"
t_s = Shell thickness = 0.375"
t_b = Bottom PL thickness = 0.25"
E_s = Shell + bottom PL mod. of elasticity = 28,000 ksi
n = # of bolts = 8
D = Bolt diameter = 2"
h_1 = 27"
h_2 = 54.5"
E_b = Bolt modulus of elasticity = 29,000 ksi
ω = 0.0120 ksi
f_{sa} = Shell buckling stress = 23 ksi   p. 67, Ref. 1
P_D = 138 k   p. T-2
P_{VERT} = -55.2 k

Program Output

θ_{NA} = 0.4007 rad

P_s = -1103.1 k
P_b = 890.0 k
P_L = 157.9 k

M_s = 260,500 k-in
M_b = 46,020 k-in
M_L = 10,780 k-in

$\check{M}_U$ = 317,300 k-in

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SHEET NO. T-5

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-30-87

CLIENT LLNL SUBJECT Tank CHK'D MKR DATE 10/5/87

Strength Factor

$\check{M}_U = 317,300$ k-in

$M = 233,800$ k-in

$$\check{F}_S = \frac{317,300}{233,800}$$

$= 1.35$

Uncertainties

Shell Buckling Stress

Estimate $\sigma_{cr-1\beta} = 17.4$ ksi

$M_{-1\beta} = 302,200$ k-in

$$\beta = -\ln \frac{302,200}{317,300}$$

$= 0.05$

Bottom PL Holdown Force

Estimate no bottom PL holdown is an extreme lower bound, 3σ below the mean.

$M_{-3\sigma} = 273,700$ k-in

$$\beta = -\frac{1}{3} \ln \frac{273,700}{317,300}$$

$= 0.05$

A.B. Yield Stress

Estimate $f_{y\,min} = 36$ ksi has 5% confidence of nonexceedence.

Inelastic Energy Absorption Factor

A ductility of 1.3 was estimated for monotonic loading on pp. 71B to 71D (Ref. 1). However, the hysteretic energy implied by this ductility is available for only one strong motion cycle since the top plates are deformed and in one direction only (only one nut included for eq. A.B.). Under subsequent cycles, the top plate resistance is not available until the displacements are those achieved in the previous cycle. Energy dissipation in the subsequent cycles is therefore small. To account for the reduced energy absorption capability, the following effective ductility is estimated for three cycles of strong motion.

$$\mu_e = \frac{\mu - 1}{n} + 1 \qquad n = \#\text{ of cycles}$$

$$= \frac{1.3 - 1}{3} + 1$$

$$= 1.1$$

$\beta = 7\%$ Median damping for welded steel

$q = 3.00\beta^{-0.30}$

$p = q + 1$

$r = 0.48\beta^{-0.08}$

$\phi_\mu = (p\mu - q)^{-r}$

} Ref. 17

$q = 3.0(7)^{-0.30}$
$= 1.67$

$p = 1.67 + 1$
$= 2.67$

$r = 0.48(7)^{-0.08}$
$= 0.41$

EQE

ENGINEERING, PLANNING AND MANAGEMENT CONSULTANTS

SHEET NO. I-8

JOB NO 87218.01 JOB HCLPF Comparison BY PSH DATE 9-30-87

CLIENT LLNL SUBJECT Tank CHK'D MKR DATE 10/5/87

$$\check{F}_\mu = \frac{1}{\phi_\mu}$$

$$= \left[2.67(1.1) - 1.67\right]^{0.41}$$

$$= 1.1$$

Variability

A lower bound F_μ of 1.0 is estimated to be about 3σ below the mean with randomness and uncertainty about equal.

$$\beta_c \simeq -\frac{1}{3} \ln \frac{1}{1.1}$$

$$\doteq 0.03$$

$$\beta_R = \beta_U = \frac{1}{\sqrt{2}}(0.03)$$

$$= 0.02$$

Spectral Shape Factor

Response is in the amplified range, with a single mode dominant

$\check{F}_{SA} = 1.22$

$\beta_R = 0.20$

$\beta_U = 0$

Damping Factor

Median damping of 7% for welded steel structures at or approaching yield was used to define the seismic loads.

$\check{F}_{\delta} = 1.0$

Variability

As noted on p. 7, Ref. 1, 5% damping is considered to be a lower bound. This value is estimated to be about or below the mean.

Median spectral acceleration: 7% damping = 0.567g p.8

" " 5% damping = 0.636g p.22 Ref. 1

$\beta_U = \ln \frac{0.636}{0.567}$

$= 0.11$

$\beta_R \approx 0.2\beta_U$ estimated

$= 0.02$

Modeling Factor

The RWST is a relatively simple structure, with dynamic characteristics reasonably well-defined.

Note, if all of the mass participating in the impulsive mode is 30% above the mean.

$\beta_U \approx 0.10$ estimated

$\beta_R \approx \frac{1}{3} \ln \frac{1}{0.76} = 0.09 \approx 0.10$ ok

Modal Combination Factor

The absolute sum of the modal responses is an upper bound estimated to be 3σ above the mean.

$M_1 = 232,800$ k-in
$M_2 = 21,370$ k-in
$\check{M} = 233,800$ k-in

} p. 20, Ref. 1

$M_{abs} = 232,800 + 21,370$
$= 254,200$ k-in

$\beta_R = \frac{1}{3} \ln \frac{254,200}{233,800}$
$= 0.03$

Combination of EQ Components Factor

For a symmetrical cylindrical structure, the maximum horizontal response is the maximum due to a single horizontal component. Vertical response has little effect on the capacity

$\check{F}_{EC} = 1.0$
$\beta_R = 0.01$
$\beta_U = 0$

SSI Soil Deamplification Factor

The RWST is on rock.

$F_{SD}^{v} = 1.0$

$\beta_R = \beta_U = 0$

SSI Method of Analysis

$F_{MA}^{v} = 1.0$

$\beta_R = 0.01$ Estimated for rock-founded structures

$\beta_U = 0.05$

Ground Spectrum Direction Effects

Because capacity of the tank is governed by the higher of the two ground motion components, the ground spectrum used (which is the higher of the two components) is median-centered.

$F_{\psi}^{v} = 1.0$

$\beta_{\psi} = 0$

EQE

ENGINEERING, PLANNING AND MANAGEMENT CONSULTANTS

SHEET NO. T-12

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-30-87

CLIENT UNL SUBJECT Tank CHK'D MKR DATE 10/5/87

Overall Fragility

$$\check{F} = 1.35(1.1)(1.22)$$
$$= 1.8$$

$$\check{A} = 1.8(0.30g)$$
$$= 0.54g$$

$$\beta_R = \sqrt{0.02^2 + 0.20^2 + 0.02^2 + 0.03^2 + 0.01^2 + 0.01^2}$$
$$= 0.20$$

$$\beta_U = \sqrt{0.18^2 + 0.02^2 + 0.11^2 + 0.10^2 + 0.05^2}$$
$$= 0.24$$

$$\text{HCLPF Capacity} = 0.54g\, e^{-1.65(0.20+0.24)}$$
$$= \underline{0.26g}$$

CDFM Capacity

Seismic Loads

Same loads as calculated in Ref. 1 which are based upon the median NUREG/CR-0098 spectrum scaled to 0.3g at 5% damping.

From p. 25, Ref. 1

$V_{EQ} = 1453^k$

$M_{EQ} = 261,200$ k-in

$P_{EQ} = 5.08$ k

Base Moment Capacity

Allowable Bolt Capacity

Use nominal bolt area factored by 34 ksi per URS anchorage guidelines (Ref. 3).

$P_{all} = 34\,(3.14)$

$= 107^k$

Bottom PL Holdown Force

See p. 48, Ref. 1

Estimate HCLPF capacity ≈ 0.3g

$S_{av} = 0.424g$

$w = 62.4\,(32)\,[1 - 0.4\,(0.424)]$

$= 1917$ psf

$= 13.3$ psi

$= 0.0133$ ksi

Tank Vertical Load

Estimate 0.3g HCLPF capacity

$P_{DL} = 63.7^k$ p. 2, Ref. 1

$P_{EQ} = 5.08^k$ 0.3g PGA

$P_{NET} = -63.7 + 5.08$

$= -58.6^k$

Program Input - see p. T-4

$R = 240''$

$t_a = 0.315''$

$t_b = 0.25''$

$E_s = 28,000$ ksi p. T-4

$n = 8$

$D = 2''$

$h_1 = 27''$

$h_2 = 54.5''$

$E_b = 29,000$ ksi

$w = 0.0133$ ksi p. T-13

$f_{sa} = 16.5$ ksi p. 47, Ref. 1

$P_a = 107^k$ p. T-13

$P_{NET} = -58.6^k$ Above

Program Output

$\Theta_{NA} = 0.4344$ rad

$P_s = -857.4^k$

$P_b = 649.6^k$

$P_L = 149.2^k$

EQE

ENGINEERING, PLANNING AND MANAGEMENT CONSULTANTS

SHEET NO. T-15

JOB NO 81218.01 JOB HCLPF Comparison BY PSH DATE 9-30-87

CLIENT LLNL SUBJECT Tank CHK'D MKR DATE 10/5/87

$M_s = 201{,}930$ k-in
$M_D = 42{,}550$ k-in
$M_L = 10{,}540$ k-in

$M_U = 255{,}000$ k-in

$$\text{HCLPF Capacity} = \frac{255{,}000}{261{,}200}(0.3g) = 0.29g$$

EQE

ENGINEERING, PLANNING AND MANAGEMENT CONSULTANTS

SHEET NO. T-16

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9·30·87

CLIENT LLNL SUBJECT Tank CHK'D MKR DATE 10/5/87

References

1. Maine Yankee Seismic Margin Review, RWST Calculations, Job No.

2. Memo, R.P. Kennedy to HCLPF Panel, July 8, 1987.

3. URS Corporation/John A. Blume & Associates, Engineers, "Seismic Anchorage Guidelines For Nuclear Plant Equipment", prepared for the Electric Power Research Institute, Draft, June, 1986.

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Median Capacity

p. 1/4

T A N K   D A T A

RADIUS = 2.4000E+02
SHELL THICKNESS = 3.7500E-01
BOTTOM PLATE THICKNESS = 2.5000E-01
SHELL YOUNG'S MODULUS = 2.8000E+07

A N C H O R A G E   D A T A

NUMBER OF ANCHOR BOLTS = 8
ANCHOR BOLT DIAMETER = 2.0000E+00
[illegible]SED LENGTH OF BOLT = 2.7000E+01
[illegible] LENGTH OF BOLT = 5.4500E+01
YOUNG'S MODULUS = 2.9000E+07

P R E C A L C U L A T E D   D A T A

EFFECTIVE FLUID WEIGHT = 1.2000E+01
TANK SHELL CRITICAL STRESS= 2.3000E+04
LIMIT ON BOLT CAPACITY = 1.3800E+05
NET VERTICAL BASE REACTION= -5.5200E+04

I T E R A T I O N   P A R A M E T E R S

MAXIMUM # OF ITERATIONS = 100
CONVERGENCE TOLERANCE = 1.00

****

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I N T E R M E D I A T E   R E S U L T S

p. 2/4

ITER #	NEUTRAL AXIS	PS	PB	PL	PSUM	XNORM
1	1.5708E+00	-4.1400E+06	8.9508E+04	4.3233E+04	-4.0073E+06	-3.9521E+06
2	1.0472E+00	-2.8353E+06	3.6439E+05	7.4580E+04	-2.3963E+06	-2.3411E+06
3	7.2273E-01	-1.9771E+06	6.4917E+05	1.0471E+05	-1.2232E+06	-1.1680E+06
4	5.0536E-01	-1.3888E+06	7.8960E+05	1.3596E+05	-4.6325E+05	-4.0805E+05
5	3.5542E-01	-9.7889E+05	9.6334E+05	1.6992E+05	1.5438E+05	2.0958E+05
6	4.3647E-01	-1.2008E+06	8.4751E+05	1.4964E+05	-2.0365E+05	-1.4845E+05
7	3.9790E-01	-1.0953E+06	8.9384E+05	1.5860E+05	-4.2854E+04	1.2346E+04
8	4.1761E-01	-1.1492E+06	8.6857E+05	1.5389E+05	-1.2676E+05	-7.1561E+04
9	4.0787E-01	-1.1226E+06	8.8060E+05	1.5618E+05	-8.5787E+04	-3.0587E+04
10	4.0292E-01	-1.1090E+06	8.8706E+05	1.5738E+05	-6.4580E+04	-9.3799E+03
11	4.0042E-01	-1.1022E+06	8.9041E+05	1.5799E+05	-5.3784E+04	1.4158E+03
12	4.0167E-01	-1.1056E+06	8.8873E+05	1.5768E+05	-5.9199E+04	-3.9986E+03
13	4.0104E-01	-1.1039E+06	8.8957E+05	1.5783E+05	-5.6495E+04	-1.2951E+03
14	4.0073E-01	-1.1030E+06	8.8999E+05	1.5791E+05	-5.5142E+04	5.8281E+01
15	4.0089E-01	-1.1035E+06	8.8978E+05	1.5787E+05	-5.5819E+04	-6.1928E+02
16	4.0081E-01	-1.1033E+06	8.8988E+05	1.5789E+05	-5.5480E+04	-2.8045E+02
17	4.0077E-01	-1.1031E+06	8.8993E+05	1.5790E+05	-5.5311E+04	-1.1108E+02
18	4.0075E-01	-1.1031E+06	8.8996E+05	1.5790E+05	-5.5226E+04	-2.6406E+01
19	4.0074E-01	-1.1031E+06	8.8997E+05	1.5791E+05	-5.5184E+04	1.5906E+01
20	4.0074E-01	-1.1031E+06	8.8997E+05	1.5791E+05	-5.5205E+04	-5.2344E+00
21	4.0074E-01	-1.1031E+06	8.8997E+05	1.5791E+05	-5.5194E+04	6.1719E+00
22	4.0074E-01	-1.1031E+06	8.8997E+05	1.5791E+05	-5.5200E+04	4.3750E-01

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RESULTS OF ANALYSIS

(1) NEUTRAL AXIS LOCATION:

DEGREES	RADIANS
22.9609	4.0074E-01

(2) TENSILE FORCES IN ANCHOR BOLTS:

BOLT #	REF. ANGLE (DEGREES)	FORCE
1	180.0000	1.3800E+05
2	135.0000	1.3800E+05
3	90.0000	1.3800E+05
4	45.0000	9.9985E+04

(3) DIRECT FORCES AT TANK BASE:

LONGITUDINAL FORCE IN SHELL	= -1.1031E+06
SUM OF ANCHOR BOLT FORCES	= 8.8997E+05
BOTTOM PLATE HOLDOWN FORCE	= 1.5791E+05
TOTAL	= -5.5200E+04

(4) RESTORING MOMENT:

FROM LONGITUDINAL FORCES IN SHELL	= 2.6052E+08
FROM ANCHOR BOLTS TENSILE FORCES	= 4.6023E+07
FROM BOTTOM PLATE HOLDOWN FORCE	= 1.0784E+07
TOTAL	= 3.1733E+08

A D D I T I O N A L   R E S U L T S:

```
MAXIMUM LENGTH OF UPLIFTED BOTTOM PLATE= 1.8519E+01
MAXIMUM UPLIFT DISPLACEMENT            = 5.3768E-01
MAXIMUM FIBRE STRESS IN BOTTOM PLATE   = 6.5847E+04
```

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-10 Buckling stress

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```
TANK  DATA
-----------------
RADIUS                    = 2.4000E+02
SHELL THICKNESS           = 3.7500E-01
BOTTOM PLATE THICKNESS    = 2.5000E-01
SHELL YOUNG'S MODULUS     = 2.8000E+07

ANCHORAGE  DATA
--------------------------
NUMBER OF ANCHOR BOLTS    =          8
ANCHOR BOLT DIAMETER      = 2.0000E+00
EXPOSED LENGTH OF BOLT    = 2.7000E+01
TOTAL LENGTH OF BOLT      = 5.4500E+01
BOLT YOUNG'S MODULUS      = 2.9000E+07

PRECALCULATED  DATA
-----------------------------------
EFFECTIVE FLUID WEIGHT    =  1.2000E+01
TANK SHELL CRITICAL STRESS=  1.7400E+04
LIMIT ON BOLT CAPACITY    =  1.3800E+05
NET VERTICAL BASE REACTION= -5.5200E+04

ITERATION  PARAMETERS
----------------------------------------
MAXIMUM # OF ITERATIONS   =   100
CONVERGENCE TOLERANCE     =  1.00
```

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I N T E R M E D I A T E   R E S U L T S

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ITER #	NEUTRAL AXIS	PS	PB	PL	PSUM	XNORM
1	1.5708E+00	-3.1320E+06	6.7715E+04	4.0320E+04	-3.0240E+06	-2.9688E+06
2	1.0472E+00	-2.1450E+06	2.7567E+05	6.9554E+04	-1.7997E+06	-1.7445E+06
3	7.2273E-01	-1.4957E+06	5.9191E+05	9.7651E+04	-8.0612E+05	-7.5092E+05
4	5.0536E-01	-1.0507E+06	7.6535E+05	1.2680E+05	-1.5852E+05	-1.0332E+05
5	3.5542E-01	-7.4055E+05	8.9679E+05	1.5848E+05	3.1471E+05	3.6991E+05
6	4.3647E-01	-9.0843E+05	8.0916E+05	1.3956E+05	4.0285E+04	9.5485E+04
7	4.7208E-01	-9.8201E+05	7.8412E+05	1.3266E+05	-6.5226E+04	-1.0026E+04
8	4.5460E-01	-9.4591E+05	7.9568E+05	1.3596E+05	-1.4270E+04	4.0930E+04
9	4.6341E-01	-9.6412E+05	7.8969E+05	1.3427E+05	-4.0159E+04	1.5041E+04
10	4.6777E-01	-9.7310E+05	7.8685E+05	1.3346E+05	-5.2791E+04	2.4090E+03
11	4.6993E-01	-9.7757E+05	7.8548E+05	1.3306E+05	-5.9032E+04	-3.8324E+03
12	4.6885E-01	-9.7534E+05	7.8616E+05	1.3326E+05	-5.5918E+04	-7.1813E+02
13	4.6831E-01	-9.7422E+05	7.8651E+05	1.3336E+05	-5.4356E+04	8.4408E+02
14	4.6858E-01	-9.7478E+05	7.8634E+05	1.3331E+05	-5.5138E+04	6.2406E+01
15	4.6871E-01	-9.7506E+05	7.8625E+05	1.3328E+05	-5.5528E+04	-3.2791E+02
16	4.6864E-01	-9.7492E+05	7.8629E+05	1.3330E+05	-5.5332E+04	-1.3225E+02
17	4.6861E-01	-9.7485E+05	7.8631E+05	1.3330E+05	-5.5235E+04	-3.4516E+01
18	4.6859E-01	-9.7481E+05	7.8632E+05	1.3330E+05	-5.5186E+04	1.4313E+01
19	4.6860E-01	-9.7483E+05	7.8632E+05	1.3330E+05	-5.5210E+04	-1.0453E+01
20	4.6860E-01	-9.7482E+05	7.8632E+05	1.3330E+05	-5.5198E+04	2.0000E+00
21	4.6860E-01	-9.7483E+05	7.8632E+05	1.3330E+05	-5.5205E+04	-4.7031E+00
22	4.6860E-01	-9.7483E+05	7.8632E+05	1.3330E+05	-5.5201E+04	-9.5313E-01

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R E S U L T S   O F   A N A L Y S I S

(1) NEUTRAL AXIS LOCATION:

DEGREES	RADIANS
26.8487	4.6860E-01

(2) TENSILE FORCES IN ANCHOR BOLTS:

BOLT #	REF. ANGLE (DEGREES)	FORCE
1	180.0000	1.3800E+05
2	135.0000	1.3800E+05
3	90.0000	1.3800E+05
4	45.0000	4.8160E+04

(3) DIRECT FORCES AT TANK BASE:

LONGITUDINAL FORCE IN SHELL	= -9.7483E+05
SUM OF ANCHOR BOLT FORCES	= 7.8632E+05
BOTTOM PLATE HOLDOWN FORCE	= 1.3330E+05
TOTAL	= -5.5201E+04

(4) RESTORING MOMENT:

FROM LONGITUDINAL FORCES IN SHELL	= 2.2888E+08
FROM ANCHOR BOLTS TENSILE FORCES	= 6.3613E+07
FROM BOTTOM PLATE HOLDOWN FORCE	= 9.7444E+06
TOTAL	= 3.0224E+08

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A D D I T I O N A L   R E S U L T S:

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MAXIMUM LENGTH OF UPLIFTED BOTTOM PLATE= 1.5932E+01
MAXIMUM UPLIFT DISPLACEMENT            = 2.9452E-01
MAXIMUM FIBRE STRESS IN BOTTOM PLATE   = 4.8734E+04
```

****
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****

No Bottom PL Holdown

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T A N K   D A T A

RADIUS	=	2.4000E+02
SHELL THICKNESS	=	3.7500E-01
BOTTOM PLATE THICKNESS	=	.0000E+00
SHELL YOUNG'S MODULUS	=	2.8000E+07

A N C H O R A G E   D A T A

NUMBER OF ANCHOR BOLTS	=	8
ANCHOR BOLT DIAMETER	=	2.0000E+00
EXPOSED LENGTH OF BOLT	=	2.7000E+01
TOTAL LENGTH OF BOLT	=	5.4500E+01
BOLT YOUNG'S MODULUS	=	2.9000E+07

P R E C A L C U L A T E D   D A T A

EFFECTIVE FLUID WEIGHT	=	1.2000E+01
TANK SHELL CRITICAL STRESS	=	2.3000E+04
LIMIT ON BOLT CAPACITY	=	1.3800E+05
NET VERTICAL BASE REACTION	=	-5.5200E+04

I T E R A T I O N   P A R A M E T E R S

MAXIMUM # OF ITERATIONS	=	100
CONVERGENCE TOLERANCE	=	1.00

INTERMEDIATE RESULTS

ITER #	NEUTRAL AXIS	PS	PB	PL	PSUM	XNORM
1	1.5708E+00	-4.1400E+06	8.9508E+04	.0000E+00	-4.0505E+06	-3.9953E+06
2	1.0472E+00	-2.8353E+06	3.6439E+05	.0000E+00	-2.4709E+06	-2.4157E+06
3	7.2273E-01	-1.9771E+06	6.4917E+05	.0000E+00	-1.3279E+06	-1.2727E+06
4	5.0536E-01	-1.3888E+06	7.8960E+05	.0000E+00	-5.9921E+05	-5.4401E+05
5	3.5542E-01	-9.7889E+05	9.6334E+05	.0000E+00	-1.5548E+04	3.9652E+04
6	4.3647E-01	-1.2008E+06	8.4751E+05	.0000E+00	-3.5329E+05	-2.9809E+05
7	3.9790E-01	-1.0953E+06	8.9384E+05	.0000E+00	-2.0146E+05	-1.4626E+05
8	3.7723E-01	-1.0387E+06	9.2473E+05	.0000E+00	-1.1395E+05	-5.8750E+04
9	3.6648E-01	-1.0092E+06	9.4290E+05	.0000E+00	-6.6315E+04	-1.1115E+04
10	3.6099E-01	-9.9417E+05	9.5281E+05	.0000E+00	-4.1354E+04	1.3846E+04
11	3.6375E-01	-1.0017E+06	9.4778E+05	.0000E+00	-5.3936E+04	1.2636E+03
12	3.6512E-01	-1.0055E+06	9.4532E+05	.0000E+00	-6.0151E+04	-4.9508E+03
13	3.6443E-01	-1.0036E+06	9.4655E+05	.0000E+00	-5.7050E+04	-1.8499E+03
14	3.6409E-01	-1.0027E+06	9.4716E+05	.0000E+00	-5.5494E+04	-2.9400E+02
15	3.6392E-01	-1.0022E+06	9.4747E+05	.0000E+00	-5.4715E+04	4.8469E+02
16	3.6400E-01	-1.0024E+06	9.4732E+05	.0000E+00	-5.5104E+04	9.6250E+01
17	3.6405E-01	-1.0025E+06	9.4724E+05	.0000E+00	-5.5299E+04	-9.8938E+01
18	3.6403E-01	-1.0025E+06	9.4728E+05	.0000E+00	-5.5201E+04	-1.3750E+00
19	3.6401E-01	-1.0025E+06	9.4730E+05	.0000E+00	-5.5154E+04	4.6375E+01
20	3.6402E-01	-1.0025E+06	9.4729E+05	.0000E+00	-5.5177E+04	2.2563E+01
21	3.6402E-01	-1.0025E+06	9.4728E+05	.0000E+00	-5.5189E+04	1.0625E+01
22	3.6402E-01	-1.0025E+06	9.4728E+05	.0000E+00	-5.5195E+04	4.6250E+00
23	3.6402E-01	-1.0025E+06	9.4728E+05	.0000E+00	-5.5199E+04	6.2500E-01

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R E S U L T S O F A N A L Y S I S

p.3/3

(1) NEUTRAL AXIS LOCATION:

DEGREES	RADIANS
20.8571	3.6402E-01

(2) TENSILE FORCES IN ANCHOR BOLTS:

BOLT #	REF. ANGLE (DEGREES)	FORCE
1	180.0000	1.3800E+05
2	135.0000	1.3800E+05
3	90.0000	1.3800E+05
4	45.0000	1.2864E+05

(3) DIRECT FORCES AT TANK BASE:

LONGITUDINAL FORCE IN SHELL	=	-1.0025E+06
SUM OF ANCHOR BOLT FORCES	=	9.4728E+05
BOTTOM PLATE HOLDOWN FORCE	=	.0000E+00
TOTAL	=	-5.5199E+04

(4) RESTORING MOMENT:

FROM LONGITUDINAL FORCES IN SHELL	=	2.3743E+08
FROM ANCHOR BOLTS TENSILE FORCES	=	3.6297E+07
FROM BOTTOM PLATE HOLDOWN FORCE	=	.0000E+00
TOTAL	=	2.7373E+08

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Lower Bound A.B. Yield Stress   p.1/4

T A N K   D A T A

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-----------------
RADIUS                    = 2.4000E+02
SHELL THICKNESS           = 3.7500E-01
BOTTOM PLATE THICKNESS    = 2.5000E-01
SHELL YOUNG'S MODULUS     = 2.8000E+07
```

A N C H O R A G E   D A T A

```
---------------------------
NUMBER OF ANCHOR BOLTS    =          8
ANCHOR BOLT DIAMETER      = 2.0000E+00
EXPOSED LENGTH OF BOLT    = 2.7000E+01
TOTAL LENGTH OF BOLT      = 5.4500E+01
BOLT YOUNG'S MODULUS      = 2.9000E+07
```

P R E C A L C U L A T E D   D A T A

```
-----------------------------------
EFFECTIVE FLUID WEIGHT    =  1.2000E+01
TANK SHELL CRITICAL STRESS=  2.3000E+04
LIMIT ON BOLT CAPACITY    =  1.1300E+05
NET VERTICAL BASE REACTION= -5.5200E+04
```

I T E R A T I O N   P A R A M E T E R S

```
---------------------------------------
MAXIMUM # OF ITERATIONS   =   100
CONVERGENCE TOLERANCE     =   1.00
```

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p.2/4

INTERMEDIATE RESULTS

ITER #	NEUTRAL AXIS	PS	PB	PL	PSUM	XNORM
1	1.5708E+00	-4.1400E+06	8.9508E+04	4.3233E+04	-4.0073E+06	-3.9521E+06
2	1.0472E+00	-2.8353E+06	3.6439E+05	7.4580E+04	-2.3963E+06	-2.3411E+06
3	7.2273E-01	-1.9771E+06	5.7417E+05	1.0471E+05	-1.2982E+06	-1.2430E+06
4	5.0536E-01	-1.3888E+06	6.6460E+05	1.3596E+05	-5.8825E+05	-5.3305E+05
5	3.5542E-01	-9.7889E+05	7.9100E+05	1.6992E+05	-1.7964E+04	3.7236E+04
6	4.3647E-01	-1.2008E+06	7.2251E+05	1.4964E+05	-3.2865E+05	-2.7345E+05
7	3.9790E-01	-1.0953E+06	7.6884E+05	1.5860E+05	-1.6785E+05	-1.1265E+05
8	3.7723E-01	-1.0387E+06	7.9100E+05	1.6390E+05	-8.3782E+04	-2.8582E+04
9	3.6648E-01	-1.0092E+06	7.9100E+05	1.6681E+05	-5.1406E+04	3.7940E+03
10	3.7189E-01	-1.0241E+06	7.9100E+05	1.6533E+05	-6.7721E+04	-1.2521E+04
11	3.6920E-01	-1.0167E+06	7.9100E+05	1.6606E+05	-5.9597E+04	-4.3969E+03
12	3.6784E-01	-1.0129E+06	7.9100E+05	1.6643E+05	-5.5509E+04	-3.0927E+02
13	3.6716E-01	-1.0111E+06	7.9100E+05	1.6662E+05	-5.3461E+04	1.7392E+03
14	3.6750E-01	-1.0120E+06	7.9100E+05	1.6653E+05	-5.4486E+04	7.1423E+02
15	3.6767E-01	-1.0125E+06	7.9100E+05	1.6648E+05	-5.4998E+04	2.0242E+02
16	3.6776E-01	-1.0127E+06	7.9100E+05	1.6646E+05	-5.5254E+04	-5.4391E+01
17	3.6771E-01	-1.0126E+06	7.9100E+05	1.6647E+05	-5.5126E+04	7.4094E+01
18	3.6774E-01	-1.0127E+06	7.9100E+05	1.6646E+05	-5.5189E+04	1.0875E+01
19	3.6775E-01	-1.0127E+06	7.9100E+05	1.6646E+05	-5.5221E+04	-2.0891E+01
20	3.6774E-01	-1.0127E+06	7.9100E+05	1.6646E+05	-5.5205E+04	-4.9375E+00
21	3.6774E-01	-1.0127E+06	7.9100E+05	1.6646E+05	-5.5197E+04	2.9375E+00
22	3.6774E-01	-1.0127E+06	7.9100E+05	1.6646E+05	-5.5201E+04	-1.0469E+00
23	3.6774E-01	-1.0127E+06	7.9100E+05	1.6646E+05	-5.5200E+04	7.8125E-02

****
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****

R E S U L T S   O F   A N A L Y S I S

p.-3/4

(1) NEUTRAL AXIS LOCATION:

DEGREES	RADIANS
21.0699	3.6774E-01

(2) TENSILE FORCES IN ANCHOR BOLTS:

BOLT #	REF. ANGLE (DEGREES)	FORCE
1	180.0000	1.1300E+05
2	135.0000	1.1300E+05
3	90.0000	1.1300E+05
4	45.0000	1.1300E+05

(3) DIRECT FORCES AT TANK BASE:

LONGITUDINAL FORCE IN SHELL	= -1.0127E+06
SUM OF ANCHOR BOLT FORCES	= 7.9100E+05
BOTTOM PLATE HOLDOWN FORCE	= 1.6646E+05
TOTAL	= -5.5200E+04

(4) RESTORING MOMENT:

FROM LONGITUDINAL FORCES IN SHELL	= 2.3978E+08
FROM ANCHOR BOLTS TENSILE FORCES	= 2.7120E+07
FROM BOTTOM PLATE HOLDOWN FORCE	= 1.0994E+07
TOTAL	= 2.7789E+08

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*****************************************************************************************************************************
                    1)+@NORMAL.STY10-01-87    08:35:34
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p.4/4

A D D I T I O N A L   R E S U L T S:

```
MAXIMUM LENGTH OF UPLIFTED BOTTOM PLATE= 1.9353E+01
MAXIMUM UPLIFT DISPLACEMENT            = 6.4128E-01
MAXIMUM FIBRE STRESS IN BOTTOM PLATE   = 7.1912E+04
```

****

1)*4NORMAL.STY10-06-87   10:45:49

****

CDFM Capacity

TANK DATA

RADIUS = 2.4000E+02
SHELL THICKNESS = 3.7500E-01
BOTTOM PLATE THICKNESS = 2.5000E-01
SHELL YOUNG'S MODULUS = 2.8000E+07

ANCHORAGE DATA

NUMBER OF ANCHOR BOLTS = 8
ANCHOR BOLT DIAMETER = 2.0000E+00
EXPOSED LENGTH OF BOLT = 2.7000E+01
TOTAL LENGTH OF BOLT = 5.4500E+01
BOLT YOUNG'S MODULUS = 2.9000E+07

PRECALCULATED DATA

EFFECTIVE FLUID WEIGHT = 1.3300E+01
TANK SHELL CRITICAL STRESS= 1.6500E+04
LIMIT ON BOLT CAPACITY = 1.0700E+05
NET VERTICAL BASE REACTION= -5.8600E+04

ITERATION PARAMETERS

MAXIMUM # OF ITERATIONS = 100
CONVERGENCE TOLERANCE = 1.00

2/4

I N T E R M E D I A T E   R E S U L T S

ITER #	NEUTRAL AXIS	PS	PB	PL	PSUM	XNORM
1	1.5708E+00	-2.9700E+06	6.4212E+04	4.2979E+04	-2.8628E+06	-2.8042E+06
2	1.0472E+00	-2.0340E+06	2.6141E+05	7.4141E+04	-1.6985E+06	-1.6399E+06
3	7.2273E-01	-1.4183E+06	4.8971E+05	1.0409E+05	-8.2452E+05	-7.6592E+05
4	5.0536E-01	-9.9632E+05	6.0645E+05	1.3516E+05	-2.5471E+05	-1.9611E+05
5	3.5542E-01	-7.0225E+05	7.3109E+05	1.6893E+05	1.9777E+05	2.5637E+05
6	4.3647E-01	-8.6145E+05	6.4800E+05	1.4876E+05	-6.4687E+04	-6.0865E+03
7	3.9790E-01	-7.8576E+05	6.8123E+05	1.5767E+05	5.3149E+04	1.1175E+05
8	4.1761E-01	-8.2444E+05	6.6310E+05	1.5299E+05	-8.3509E+03	5.0249E+04
9	4.2714E-01	-8.4314E+05	6.5522E+05	1.5082E+05	-3.7096E+04	2.1504E+04
10	4.3183E-01	-8.5234E+05	6.5153E+05	1.4978E+05	-5.1028E+04	7.5723E+03
11	4.3415E-01	-8.5690E+05	6.4975E+05	1.4927E+05	-5.7890E+04	7.0953E+02
12	4.3531E-01	-8.5918E+05	6.4887E+05	1.4901E+05	-6.1296E+04	-2.6963E+03
13	4.3473E-01	-8.5804E+05	6.4931E+05	1.4914E+05	-5.9596E+04	-9.9564E+02
14	4.3444E-01	-8.5747E+05	6.4953E+05	1.4920E+05	-5.8743E+04	-1.4338E+02
15	4.3430E-01	-8.5719E+05	6.4964E+05	1.4924E+05	-5.8317E+04	2.8303E+02
16	4.3437E-01	-8.5733E+05	6.4958E+05	1.4922E+05	-5.8530E+04	6.9828E+01
17	4.3441E-01	-8.5740E+05	6.4955E+05	1.4921E+05	-5.8637E+04	-3.6688E+01
18	4.3439E-01	-8.5737E+05	6.4957E+05	1.4922E+05	-5.8584E+04	1.6109E+01
19	4.3440E-01	-8.5738E+05	6.4956E+05	1.4921E+05	-5.8610E+04	-9.8906E+00
20	4.3439E-01	-8.5738E+05	6.4956E+05	1.4922E+05	-5.8597E+04	2.5781E+00
21	4.3440E-01	-8.5738E+05	6.4956E+05	1.4922E+05	-5.8603E+04	-3.1094E+00
22	4.3439E-01	-8.5738E+05	6.4956E+05	1.4922E+05	-5.8600E+04	-2.9688E-01

R E S U L T S   O F   A N A L Y S I S

3/4

(1) NEUTRAL AXIS LOCATION:

DEGREES	RADIANS
24.8890	4.3439E-01

(2) TENSILE FORCES IN ANCHOR BOLTS:

BOLT #	REF. ANGLE (DEGREES)	FORCE
1	180.0000	1.0700E+05
2	135.0000	1.0700E+05
3	90.0000	1.0700E+05
4	45.0000	5.7281E+04

(3) DIRECT FORCES AT TANK BASE:

LONGITUDINAL FORCE IN SHELL	= -8.5738E+05
SUM OF ANCHOR BOLT FORCES	= 6.4956E+05
BOTTOM PLATE HOLDOWN FORCE	= 1.4922E+05
TOTAL	= -5.8600E+04

(4) RESTORING MOMENT:

FROM LONGITUDINAL FORCES IN SHELL	= 2.0193E+08
FROM ANCHOR BOLTS TENSILE FORCES	= 4.2555E+07
FROM BOTTOM PLATE HOLDOWN FORCE	= 1.0541E+07
TOTAL	= 2.5502E+08

****

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4/4

ADDITIONAL RESULTS:

```
MAXIMUM LENGTH OF UPLIFTED BOTTOM PLATE= 1.5935E+01
MAXIMUM UPLIFT DISPLACEMENT            = 3.2672E-01
MAXIMUM FIBRE STRESS IN BOTTOM PLATE   = 5.4038E+04
```

COMPONENT 2

MOTOR CONTROL CENTER

EQE

ENGINEERING, PLANNING AND MANAGEMENT CONSULTANTS

SHEET NO. 1

JOB NO. 87-218.01 JOB HCLPF STUDY BY RDC DATE 10-1-87

CLIENT LLNL SUBJECT MOTOR CONTROL CENTER CHK'D MKR DATE 10/4/87

DEVELOP FRAGILITY OF MOTOR CONTROL CENTER.

AVAILABLE INFORMATION AND GROUND RULES ARE:

1) CABINET HAS f_n = 6.5 Hz

2) NO QUALIFICATION TEST, THUS USE ANCO GERS OR OTHER GENERIC SOURCE

3) CABINET IS MOUNTED IN STRUCTURE FOR WHICH MEDIAN FLOOR SPECTRA ARE GIVEN FOR A 0.18 G PEAK GROUND ACCEL. ALSO DO CASE FOR GROUND MOUNTED

4) EQ IS DEFINED AS 84% NEP EARTHQUAKE

5) INVESTIGATE AUXILIARY CONTACTOR CHATTER FRAGILITY "Function During E.Q." ALSO DEVELOP FRAGILITY FOR "FUNCTION AFTER EQ"

References:

1. Generic Seismic Ruggedness of Power Plant Equipment EPRI NP-5223, May 1987, Prepared by Anco Engrs.
2. Evaluation of Nuclear Power Plant Seismic Margin, Draft EPRI Report, Prepared by NTS Engineering, RPK Consulting, Pickard Lowe & Garrick, Woodward Clyde Associates and Duke Power Co. Aug, 1987

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SHEET NO. 2

JOB NO. 87212-01 JOB HCLPF Study BY RDC DATE 10-1-87

CLIENT LLNL SUBJECT MCC CHK'D MKR DATE 10/4/87

Do two cases for MCC's,

1) Function during earthquake assuming unenergized auxiliary contactors and relays which are in interlock circuits.

2) Only function after earthquake is important (ie), circuit diagram review indicates no consequence of relay chatter.

The GERS report has TRS's for 13 tests, in 5 of which some form of functional failure occured and 8 of which no anomolies were observed. The "functional during the EQ" GERS are a lower bound of the failures while the "functional after" are an upper bound of success but not above any failure case that was not functional after the seismic test. We cannot derive any real statistical median

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SHEET NO. 3

JOB NO. 87218-01 JOB HCLPF Study BY RDC DATE 10/2/87

CLIENT LLNL SUBJECT MCC CHK'D MKR DATE 10/4/87

fragility from the GERS as presented since we do not know the cabinet frequencies vs the frequency content of the input motions. We must then make some approximate estimates of a broad banded fragility curve.

One other source of data from the SSMRP program lists a median relay chatter fragility at 2.07g for a broad banded spectrum from 5-500 Hz. The median was derived from 17 shock tests in MCCs, switchgear and control cabinets during the SAFEGUARD PROGRAM. The uncertainty was very large ($\beta_c \sim 1.46$) due to the fact that some very sensitive relays were included in the original testing. This obviously biased the median.

From the GERS we can deduce that there are MCC's that would function during an EQ with greater than 4 g Sa at the base. Likewise, since there is not very much

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SHEET NO. 4

JOB NO. 87218-01 JOB HCLPF Study BY RDC DATE 10/1/87

CLIENT LLNL SUBJECT MCC CHK'D MKR DATE 10/4/87

data, there may be failures to function below 1.5g. Let us use the SSMRP median and estimate a 95% confidence value using the lower bound GERS, the suggested 0.87 factor for interlock circuits, Ref. 1, and the EPRI seismic margin reduction factor of 1.3, Ref. 2.

(Note that SSRAP suggests a reduction of 1.5 for A-46, however, this is a licensing issue for SSE)

Since SSMRP median is biased by low capacity relays, Increase SSMRP by 20% as best median estimate

Median = 1.2 (2.07) = 2.48g

HCLPF on MCC alone is $\frac{1.5(0.87)}{1.3} \approx 1.0g$

$$\beta = \frac{1}{1.65} \ln \frac{2.48}{1.0} = 0.55$$

The upper 5% confidence bound is then

$$5\%\ \text{Conf} = 2.48 e^{1.65(0.55)} = 6.15g$$

This appears reasonable with the information available.

Thus for functions during the EQ.

$\check{S}_a = 2.48\ g$

$\beta_U = 0.55$

this is contingent upon meeting the GERS ground rules that the individual relays and contactors have a 4.5g minimum ZPA input chatter capability.

Now, for function after the earthquake, the 4.0 g GERS is in the lower tail of the fragility curve. The only readily referenceabl source for "function after" would be the "trip" fragility from the SSMRP program. This value was derived from some 16 shock tests of MCCs, switchgear and control panels.

Use $\check{S}_a = 7.97\ g$ from SSMRP

Estimate the HCLPF as $GERS/1.3 = \frac{4}{1.3} = 3.08\ g$

$$\beta = \frac{1}{1.65} \ln \frac{7.97}{3.08} = 0.58$$

This is lower than the β of 0.77 from SSMRP due to the fact that there may have been some inherently weak components in the SAFEGUARDS test data plus the tests included other types of cabinets as well as MCC's.

Derive Strength factors

Mounted In Structure

Ref. GERS and instructure plots on pgs. 8 & 9.

For "Function during the EQ.", Sa = 1.65g at 6.5 Hz,

$$F_S = \frac{2.48}{1.65} = 1.50$$

$$\beta_U = 0.55$$

For Function after the EQ

$$F_S = \frac{7.97}{1.65} = 4.83$$

$$\beta_U = 0.58$$

Ground Mounted. - "Function During Quake"

5% $S_a = 2.12(0.18) = 0.38g$ (2.12 is 5% ampl. from NUREG0098 Median Spectral Shape)

$F.S._1 = \frac{2.48}{0.38} = 6.53$

$\beta_U = 0.55$

Ground Mounted - Function After Quake

$F.S. = \frac{7.97}{0.38} = 20.97$

$\beta_U = 0.58$

(8)

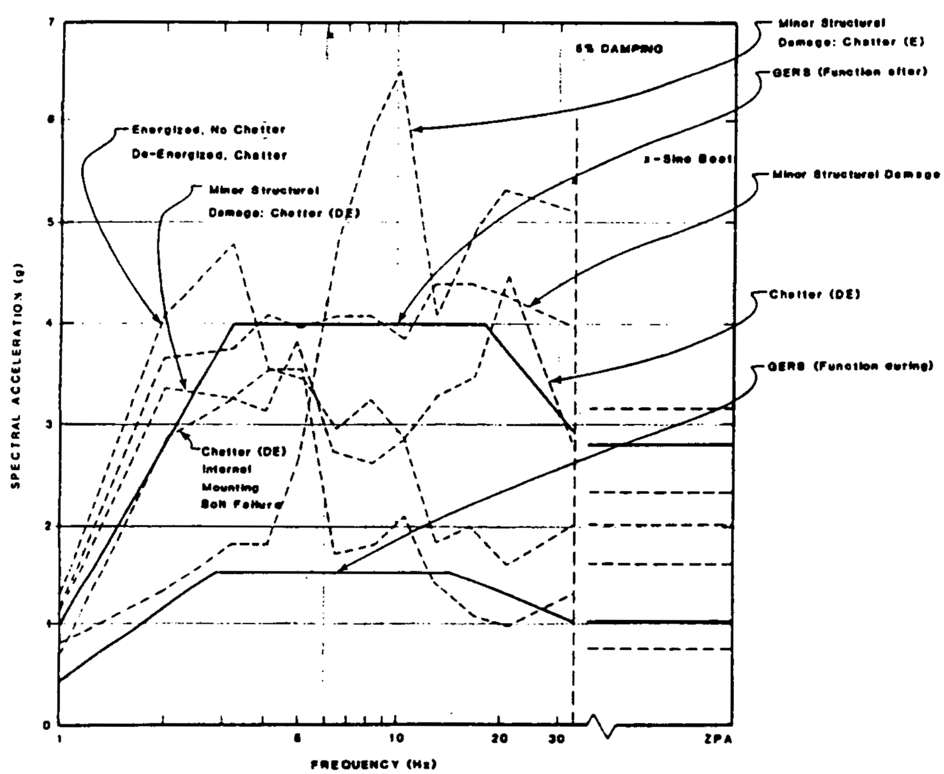
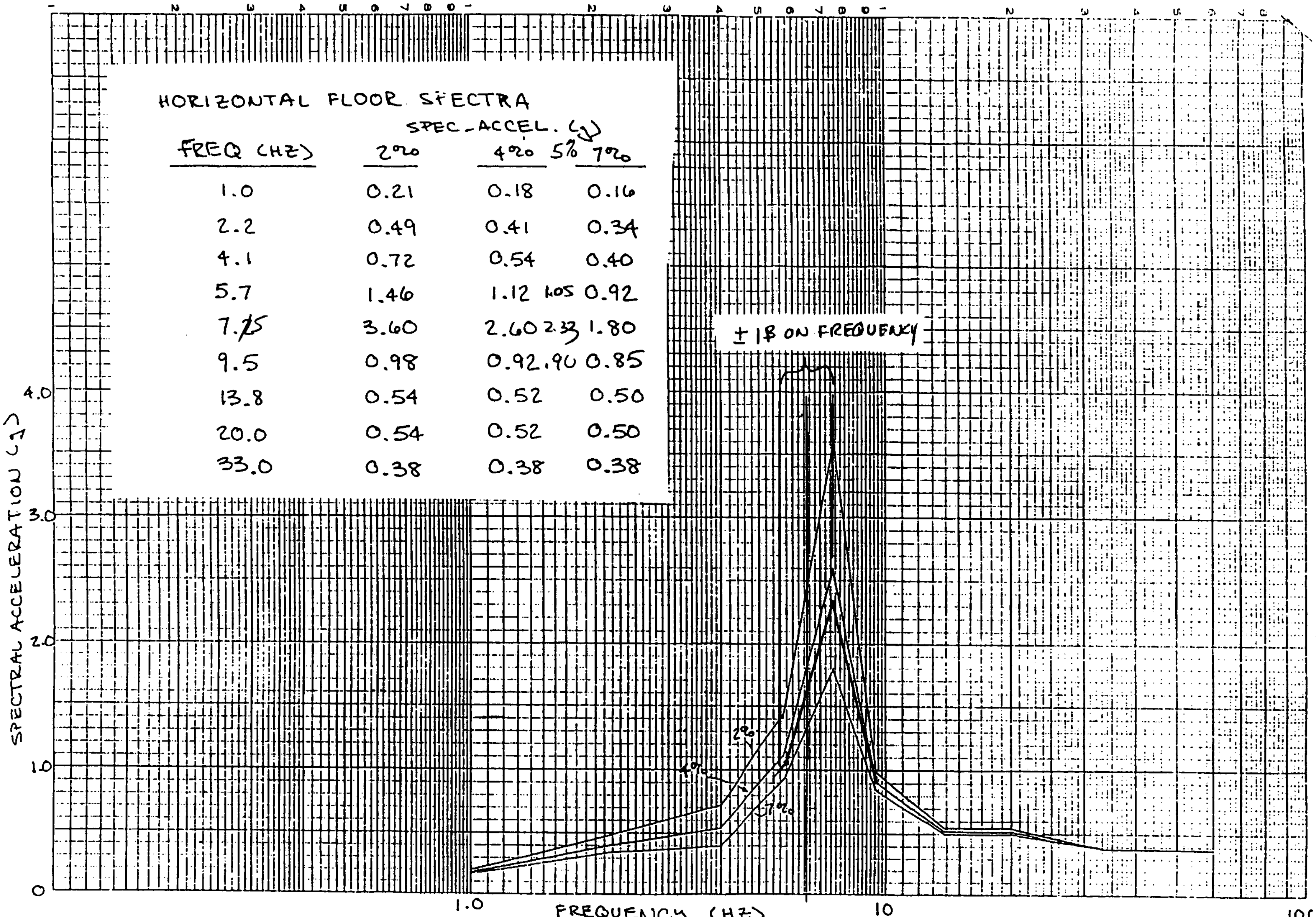


Figure 2. Comparison of GERS with failure data: function during and after for MCC.

HORIZONTAL FLOOR SPECTRA

SPEC. ACCEL. (g)

FREQ (HZ)	2%	4%	5%	7%
1.0	0.21	0.18		0.16
2.2	0.49	0.41		0.34
4.1	0.72	0.54		0.40
5.7	1.46	1.12	1.05	0.92
7.~~2~~5	3.60	2.60	2.33	1.80
9.5	0.98	0.92	.90	0.85
13.8	0.54	0.52		0.50
20.0	0.54	0.52		0.50
33.0	0.38	0.38		0.38



(4)

Equipment Response factor

Variables are:

Method of analysis

Modeling

Damping

Mode Combination

Earthquake Component Combination

Method of Analysis:

The frequency was given and assumed to be derived from analysis or insitu modal testing. Since we are comparing a SDOF response to a broad banded spectrum to determine the capacity factor, some error may result which will be addressed in other variables. There is no deliberate bias in the response calculation such as in a design situation where 1.5 peak Sa is commonly used, thus, this variable is NA,

Modeling

There are two sources of uncertainty here;

1) Frequency (stiffness) error

2) Inaccuracy in predicting response

Frequency:

Assume the error in frequency prediction is represented by a COV (~B) of 0.15

$\pm 1\beta \; f_n = 6.5e^{\pm 0.15} = \begin{matrix} 7.55 \\ 5.59 \end{matrix}$

$Sa_{f=7.55} = 2.33$

$\beta_U = \ln \frac{2.33}{1.65} = 0.35$

Response Prediction:

A SDOF representation of response for comparison to GERS is likely not biased but there are other modes which can contribute (either more or less)

This uncertainty is small compared to frequency error. Assume $\beta_U = 0.1$

$\beta_{U_M} = (0.35^2 + 0.1^2)^{1/2} = 0.37$

DAMPING:

5% is medium

$F_D = 1.0$

Assume 7% as +1β

$$\beta_{D_U} = \ln \frac{Sa_{5\%}}{Sa_{7\%}} = \ln \frac{1.65}{1.35} = 0.2$$

$$\beta_R \approx 0.2 \beta_U = 0.04$$

MODE COMBINATION:

Treated as SDOF system, this uncertainty is considered in modeling; therefore,

MC is NA.

EARTHQUAKE COMPONENT COMBINATION:

The fragility is based on biaxial test input used in developing GERS and in the SAFEGUARDS program, thus,

$F_{ECC} = 1.0$

Relays tend to be sensitive to unidirectional input only but, from summary of relay test data in EPRI margin study, Appendix A, Ref: 2,

and GERS study, Appendix A, Ref: 1,
we could expect the difference between broad banded random input biaxial tests and single axis input be no more than about $1/0.7 = 1.43$.

Let 1.43 be $3\,\beta_R$

$$\beta_{R_{ECC}} = \tfrac{1}{3}\ln 1.43 = 0.12$$

EQUIPMENT Response:

$$F_{RE} = F_m\, F_D\, F_{ECC} = (1)\,(1)\,(1) = 1.0$$

$$\beta_R = \left(0^2 + 0.04^2 + 0.12^2\right)^{1/2} = 0.13$$

$$\beta_U = \left(0.37^2 + 0.2^2 + 0\right)^{1/2} = 0.42$$

This applys to "in-structure" case.

For ground mounted case the modeling uncertainty due to frequency shifting is essentially zero since we are on the flat portion of the spectrum.

The damping uncertainty is:

$$\beta_{U_D} = \ln \frac{Sa_{5\%}}{Sa_{7\%}} = \ln \frac{2.12}{1.89} = 0.11$$

$$\beta_{RD} = 0.2\, \beta_U = 0.02$$

For Ground Case:

$$F_{RE} = 1.0$$

$$\beta_R = \left(0 + 0.02^2 + 0.12^2\right)^{1/2} = 0.12$$

$$\beta_U = \left(0.1^2 + 0.11^2 + 0\right)^{1/2} = 0.15$$

STRUCTURAL RESPONSE

Use PSH values as appropriate.

For In Structure Case:

Spectral Shape:

$F_{SS} = 1.22$

$\beta_R = 0.20$

EQ Direction Content:

$F_{EQD} = 1.1$

$\beta_R = 0.15$

Damping

$F_D = 1.0$

$\beta_{U_D} = 0.14$

$\beta_{R_D} = 0.03$

Modeling

$F_M = 1.0$

$\beta_U = 0.16$

SSI

$F_{SSI} = 1.0$

$\beta_R = 0.01$

$\beta_U = 0.05$

Structure Inelastic Response.

For in-structure case, structural inelastic response results in lower frequency which could increase MCC response. However, for the "function during EQ" case, the relay chatter capacity is low and the structure is probably elastic, therefore, this is ignored. For "function after the EQ", the structure is likely to be inelastic, but, the MCC is also probably yielding around the anchorage locations so that both frequencies shift, tending to negate the effect.

For both cases let

$$F_{SIR} = 1.0$$

$$\beta_{U_{SIR}} = 0.1$$

$$F_{SR} = 1.22(1.1)(1)(1)(1)(1) = 1.34$$

$$\beta_{R_{SR}} = (0.2^2 + 0.15^2 + 0.03^2 + 0 + 0.01^2 + 0)^{1/2} = 0.25$$

$$\beta_{U_{SR}} = (0 + 0 + 0.14^2 + 0.16^2 + 0.05^2 + 0.1^2)^{1/2} = 0.24$$

For ground mounted case:

Spectral Shape and EQ Direction Content are applicable,

$F_{SR} = 1.22(1.1) = 1.34$

$\beta_R = (0.2^2 + 0.15^2)^{1/2} = 0.25$

$\beta_U = 0$

<u>In Structure</u> – "Function During"

$\check{A} = (1.5)(1.0)(1.34)(0.18) = 0.36\,g$

$\beta_R = (0 + 0.13^2 + 0.25^2)^{1/2} = 0.28$

$\beta_U = (0.55^2 + 0.40^2 + 0.24^2)^{1/2} = 0.72$

$HCLPF = 0.36\,e^{-1.65(0.72+0.28)} = 0.07\,g$

In Structure – Function After

$\check{A} = (4.83)(1.0)(1.34)(0.18) = 1.16\,g$

$\beta_R = (0 + 0.13^2 + 0.25^2)^{1/2} = 0.28$

$\beta_U = (0.58^2 + 0.42^2 + 0.24^2)^{1/2} = 0.76$

$HCLPF = 0.21\,g$

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SHEET NO. 18

JOB NO. ____ JOB HCLPF Study BY RDC DATE 10/2/87

CLIENT ____ SUBJECT MCC CHK'D MKR DATE 10/4/87

On Ground - "Function During EQ"

$$\check{A} = (6.53)(1.0)(1.34)(0.18) = 1.58\,g$$

$$\beta_R = (0 + 0.12^2 + 0.25^2)^{1/2} = 0.28$$

$$\beta_U = (0.55^2 + 0.15^2 + 0)^{1/2} = 0.57$$

$$HCLPF = 1.58\,e^{-1.65(.57+.28)} = 0.39\,g$$

On Ground, "Function After"

$$\check{A} = (20.97)(1.0)(1.34)(0.18) = 5.06\,g$$

$$\beta_R = (0 + 0.12^2 + 0.25^2)^{1/2} = 0.28$$

$$\beta_U = (0.58^2 + 0.15^2 + 0)^{1/2} = 0.60$$

$$HCLPF = e^{-1.65(0.28+0.60)} = 1.18\,g$$

EQE

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SHEET NO. 19

JOB NO. ______ JOB HCLPF String BY RDC DATE 10/2/87

CLIENT ______ SUBJECT AICC CHK'D ______ DATE 10/4/87

EPRI DETERMINISTIC METHOD.

EQ + STRUCTURAL RESPONSE ARE MEDIAN.

COMPARE DEMAND TO GERS/1.3

FOR ASSUMED INTERLOCK CIRCUITS, REDUCE GERS by 0.87 FACTOR.

IN STRUCTURE, "FUNCTION DURING"

$$* \quad HCLPF = \frac{0.87(1.5)(0.18)}{1.3(1.65)} = 0.11g$$

IN STRUCTURE - "FUNCTION AFTER"

$$* \quad HCLPF = \frac{4.0(0.18)}{1.3(1.65)} = 0.34g$$

ON Ground - Function During

$$HCLPF = \frac{0.87(1.5)(0.18)}{1.3(2.12)(0.18)} = 0.47g \text{ vs } 0.39g$$

ON Ground - Function After

$$HCLPF = \frac{4.0(0.18)}{1.3(2.12)(0.18)} = 1.45g \text{ vs } 1.19g$$

* Sa at 6.5 Hz fn used for HCLPF. Spectrum is on linear slope so average over 20% bandwidth is Sa at 6.5 Hz worst case would be for - frequency shifted spectrum.

EQE

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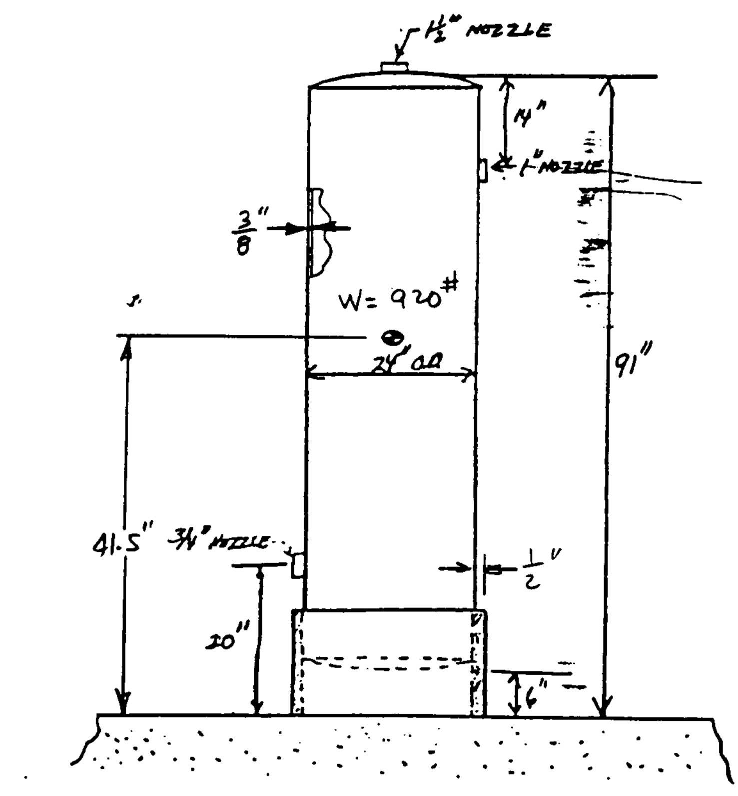
SHEET NO. 2

JOB NO. ______ JOB HCLPF Study BY RDC DATE 9/29/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D MKR DATE 10/6/87

VENDOR CALCS DID NOT DETERMINE NATURAL FREQUENCY. TANK AND SKIRT ARE RIGID BUT MOUNTING ANGLES ARE FLEXIBLE.



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SHEET NO. 20

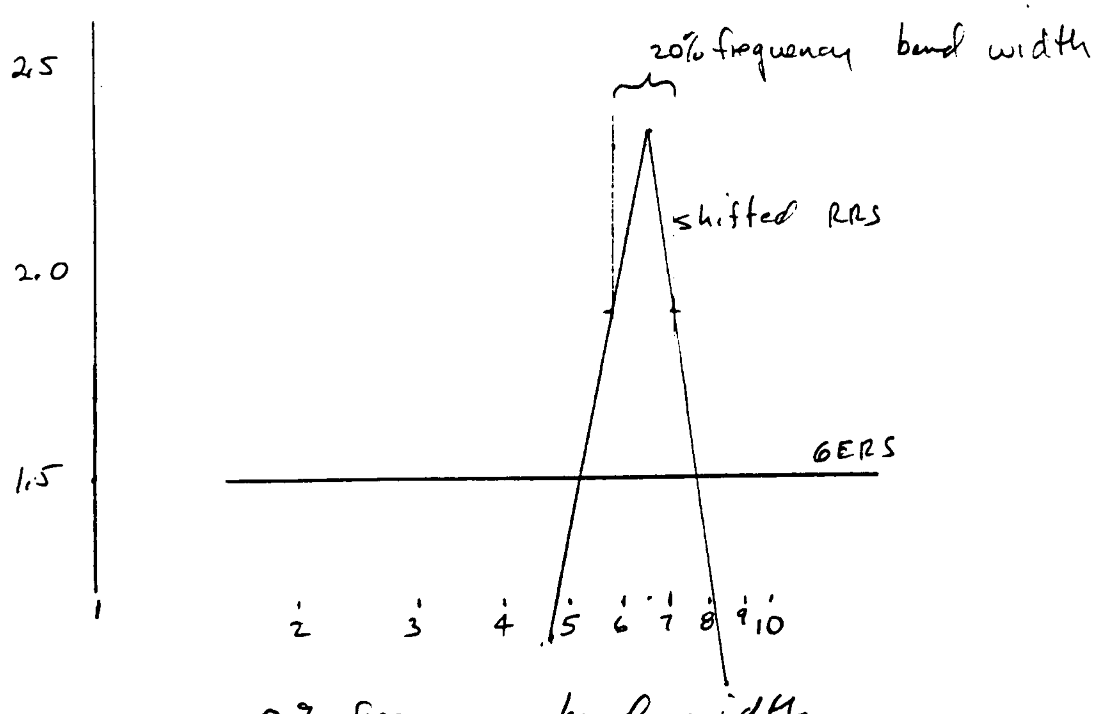
JOB NO. ______ JOB HCLPF Study BY RDC DATE 10/2/87

CLIENT ______ SUBJECT MCC CHK'D MKR DATE 10/4/87

Shift RRS by −15%

Peak is then at 6.37 Hz. Let peak be at f_n of component. The HCLPF would then be based upon maintaining a SF of 1.3 average over a 20% band width about the peak.

For peak at 6.5 Hz, ± 10% = 5.85 − 7.15 Hz.



Avg Sa over 20% frequency band width

$\bar{S_a} = 2.11\ g.$

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SHEET NO. 21

JOB NO ______ JOB HCLPF Simon BY RDC DATE 10/2/87

CLIENT ______ SUBJECT MCC CHK'D MKR DATE 10/4/87

In Structure - shifted frequency - "Function During"

$$HCLPF = \frac{0.87(1.5)(0.18)}{2.11(1.3)} = 0.086g$$ vs 0.06 by fragility method

In Structure - shifted frequency - "Function After"

$$HCLPF = \frac{4.0(0.18)}{2.11(1.3)} = 0.26g$$ vs 0.21 by fragility method

Frequency shifting does not apply to ground mounted case as RRS is broadbanded flat spectrum at fn of component, thus shifting results in same answer.

COMPONENT 3

STARTING AIR RECEIVER TANK

DEVELOP FRAGILITY OF STARTING AIR RECEIVER TANK.

REFERENCE:

GILBERT ASSOCIATES CALCULATION, FILE 1.362/d-1 7/19/79

ASSUMPTIONS

1) TANK IS MOUNTED IN STRUCTURE

2) FLOOR SPECTRA ARE FROM BUILDING ANALYSIS USING 7% DAMPING BUT ASSUME THAT THEY ARE MEDIAN CENTERED, ZPA = 0.18g

3) EARTHQUAKE IS DEFINED AS 84% NEP FOR THE LARGER OF TWO COMPONENTS OF EQ.

4) NOZZLE LOADS ARE NOT PROVIDED, THEREFORE IGNORE NOZZLE REACTIONS IN ANALYSIS

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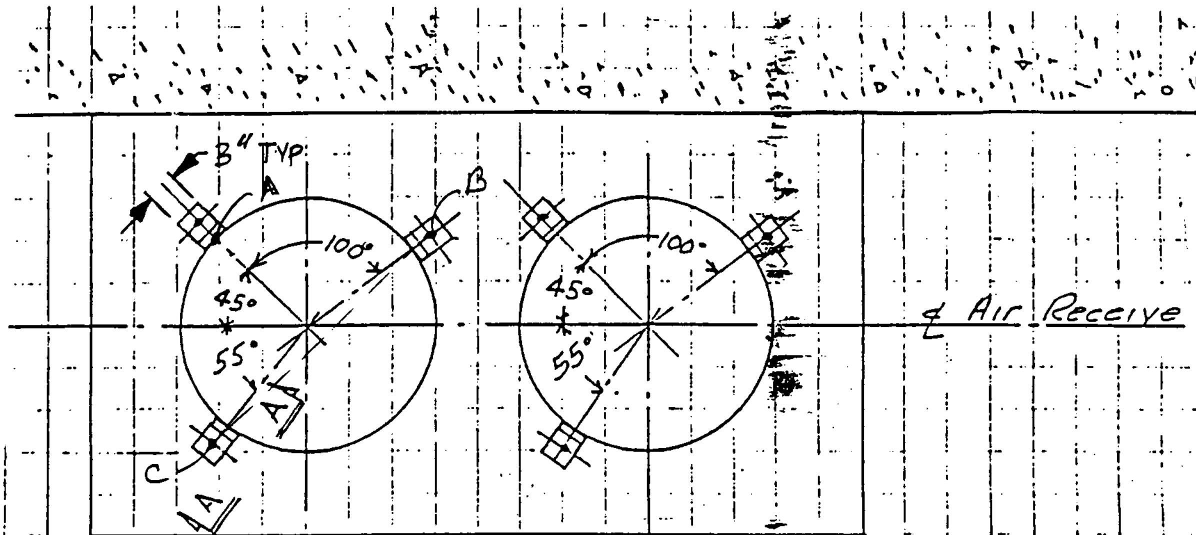
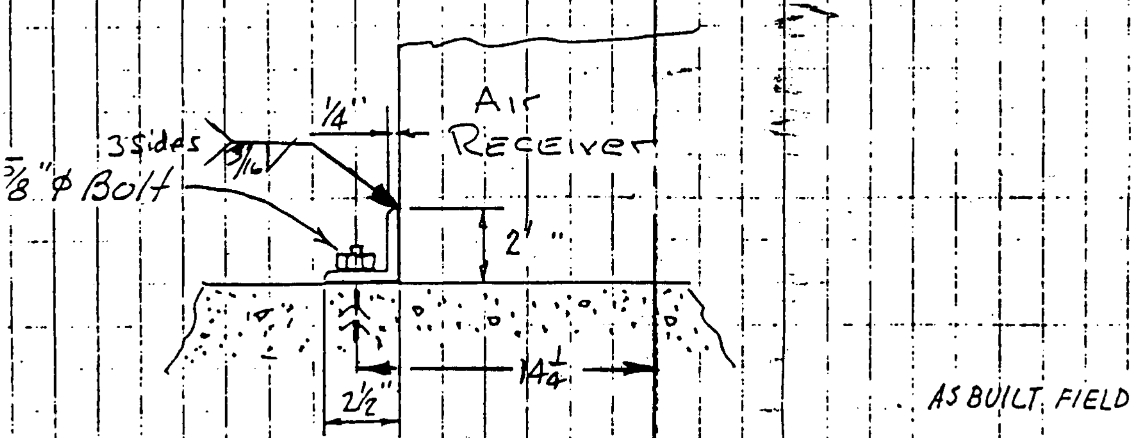
SHEET NO. 3

S NO. ______ JOB HCLPF Sitiay BY RDC DATE 9/24/87

ENT ______ SUBJECT Air Receiver Tank CHK'D MKR DATE 10/6/87



PLAN



SECTION A-A

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SHEET NO. 4

JOB NO. ______ JOB HCLPF Study BY RDC DATE 9/29/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D MKR DATE 10/5/87

ESTIMATE NATURAL FREQUENCY

FOR MOST FLEXIBLE DIRECTION FOR ANGLES, TANK ROCKS ABOUT A RESULTING IN TENSION ON BOLTS AT B & C

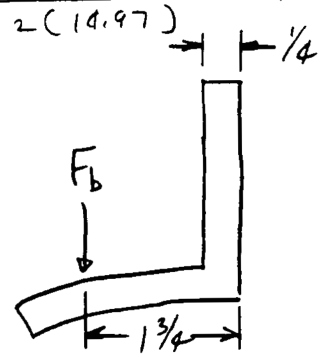
1 g LATERAL AT TANK CG PRODUCES

$M_{OT} = 920^{\#} (41.5'') = 38,180 \text{ IN}^{\#}$

$\text{FORCE / BOLT} = \frac{M_{OT}}{2(d)}$

$d = 12.5'' + 14.25 \sin 10° = 14.97''$

$F_b = \frac{38,180}{2(14.97)} = 1275^{\#}$



(Note: Bolt is not considered stiff enough in rotation to force guided cantilever bending shape. Likewise, prying action is not considered as this phenomenon is negated after bolt stretch and rotation)

Bracket Acts as cantilever Beam ~ 1½" long.

$\delta = \frac{F_b l^3}{3EI} = \frac{1275(1.5)^3(12)}{3(29 \times 10^6)(3)(0.25)^3} = 1.266 \times 10^{-2}$ in

EQE

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SHEET NO. 5

JOB NO. ______ JOB HCLPF Study BY RDC DATE 9/29/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D MKR DATE 10.6/87

Deflection at CG of Tank $= \frac{41.5}{14.97}(0.01266) = 0.0351$ in

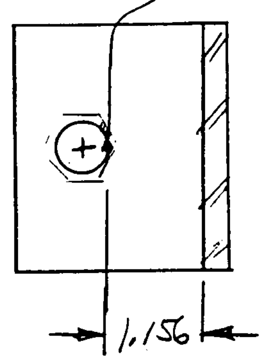
$$f_n = \frac{1}{2\pi}\sqrt{\frac{g}{\delta_{st}}} = \frac{1}{2\pi}\sqrt{\frac{386.4}{0.0351}} = 16.7\ Hz.$$

This is on a flat portion of the floor spectrum. so a more accurate calculation is not warranted. Use 16.7 as f_n for elastic system. If angle bracket yields, system is much softer.

Angle bracket appears weak relative to bolt.

5/8 - 11 bolt has thread stress area of 0.2256 in^2

Yield of bolt occurs at 0.2256 (36000) = 8122#



Put effective load at edge of hole for case where angle bracket rotates plastically. Hole is ~ 11/16 Dia.

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SHEET NO. 6

JOB NO. ______ JOB HCLPF Study BY RDC DATE 9/30/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D MKR DATE 10/6/87

Limit moment at full hinge is

$$M_L = \frac{\sigma_y t^2 \ell}{4}$$

$$M_L = \frac{36{,}000\,(0.25)^2(3)}{4} = 1687 \text{ in}^{\#}$$

$$F_L = \frac{1687}{1.156} = 1460^{\#}$$

Much weaker than bolt.

$$\bar{\sigma}_y = 1.25\,\sigma_y \text{ code}$$

$$\bar{M}_L = 1.25(1687) = 2109 \text{ in}^{\#}$$

$$\bar{F}_L = 1.25(1460) = 1825^{\#}$$

Anchor bolt detail is not given.

Assume shell anchor as worst case.

From NP 5228, mean pull out strength ~~for~~ 5/8 expansion anchors with minimum embedment is 9.51 k. which is 5.2 times the load required to form a plastic hinge in the mounting bracket. Thus, the mounting bracket is clearly the weak link.

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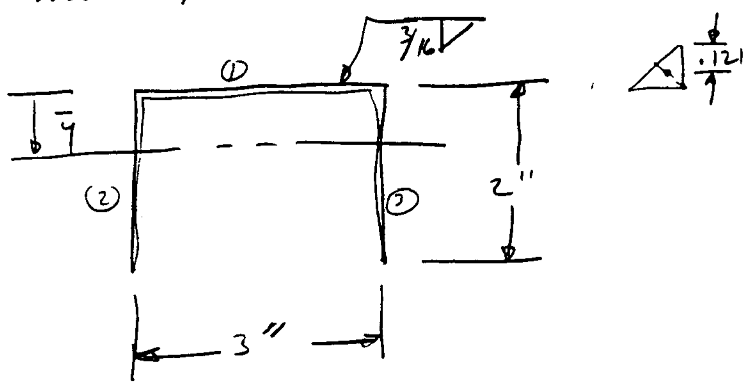
SHEET NO. 7

JOB NO. ______ JOB HCLPF Study BY RDC DATE 9/30/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D MKE DATE 10/6/87

Check capability of bracket to develop hinge moment.



Ele	A	y	Ay
1	.398	0.121	.0482
2	.263	1.0	.263
3	.263	1.0	.263
	.924		.574

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{.574}{.924} = .621$$

	A	d	Ad^2	I_{yy}
1	.398	.500	.100	—
2	.263	.379	.038	.088
3	.263	.379	.038	.088
			.176	.176

$I = .176 + .176 = .352\ in^4$

At $\bar{M}_L = 1687$ in#

$$f_v = \frac{1687(1.379)}{.352} = 6609 \text{ psi}$$

weld can carry limit moment

Failure Mode is clearly plastic bending of bracket, ie, low cycle fatigue.

Assume 5 cycles

Adjust code fatigue curve to remove factor of safety. In low cycle range, F.S. = 20 on cycles.

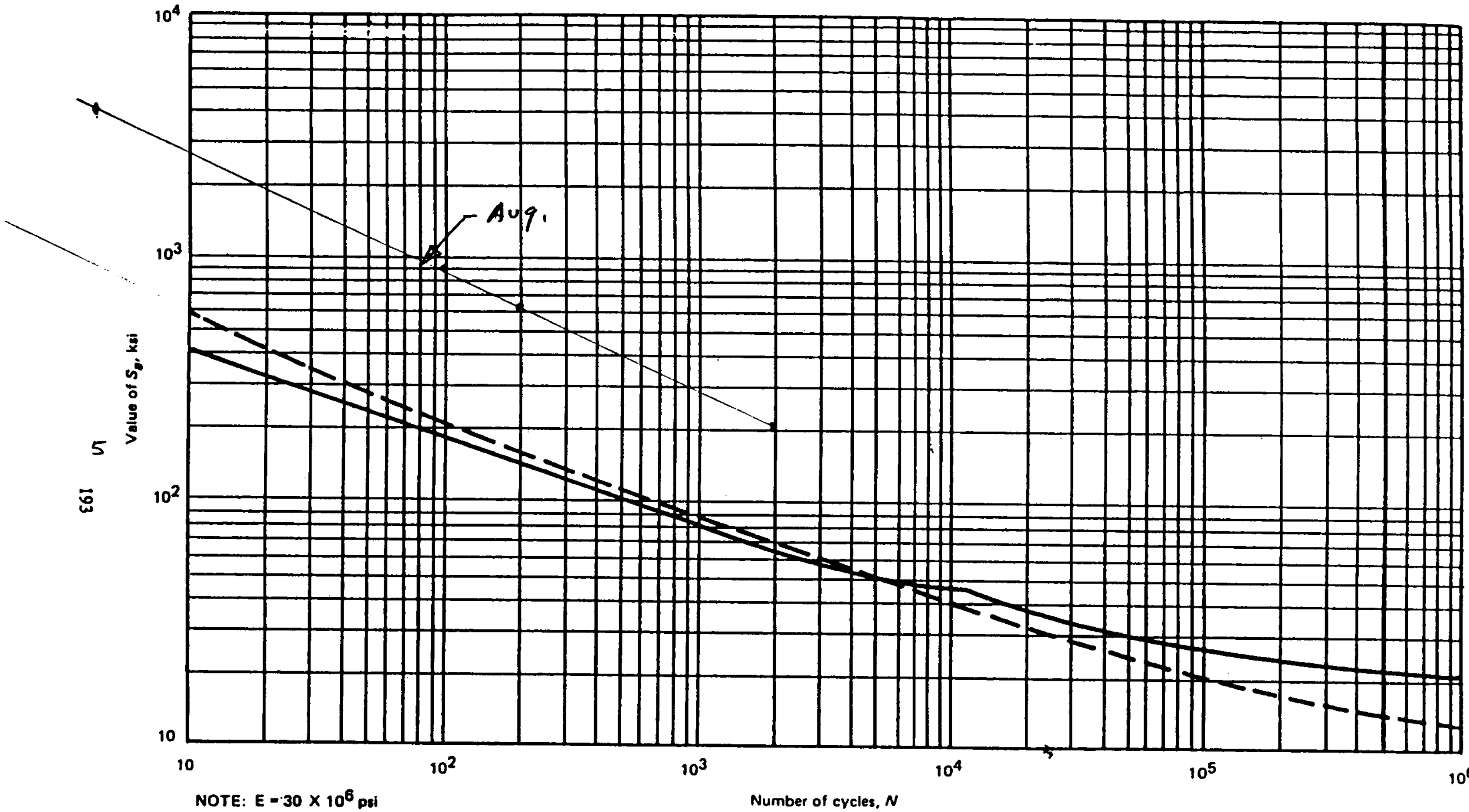
From fatigue curve with no F.S., Sa for 5 cycles is ~ 4X10^6 psi. The Sa was derived from strain controlled fatigue tests. For a defined ductility limit, the angle bracket bending can be considered to be strain controlled.

$$\varepsilon_a = \frac{4 \times 10^6}{29 \times 10^6} = .138 = 13.8\%$$

There is plastic strain concentration so that the effective nominal strain is less.

$k_e \approx 1/n$ where $n = 0.2$ for low alloy steel

Ref: NB 3228.5 of Section III, ASME Code.



NOTE: $E = 30 \times 10^6$ psi
– – – UTS < 80.0 ksi
—— UTS 115.0 – 130.0 ksi
Interpolate for UTS 80.0 – 115.0 ksi

FIG. I-9.1 DESIGN FATIGUE CURVES FOR CARBON, LOW ALLOY, AND HIGH TENSILE STEELS FOR METAL TEMPERATURES NOT EXCEEDING 700°F
Table I-9.1 Contains Tabulated Values and a Formula for Accurate Interpolation of These Curves

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SHEET NO. 9

JOB NO. ______ JOB HCLPF Study BY RPC DATE 9/30/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D MKR DATE 10/6/87

$k_e = 5$

nominal strain $= \frac{0.138}{5} = 0.0276$

0.2% offset yield strain is $\frac{1.2(36000)}{29 \times 10^6} + 0.002$

$\epsilon_y = 3.49 \times 10^{-3}$

$\mu \sim \frac{0.0276}{3.49 \times 10^{-3}} = 7.9$ say 8

F_μ based on elastic frequency may be interpolated from F_μ in highly amplified range and F_μ in rigid range.

From Newmark & Riddell Paper.

$F_\mu = [p\mu - q]^r$

$p = q+1$

$q = 3.0\gamma^{-0.3}$ for amplified acceleration range

$r = 0.48\gamma^{-0.08}$ for amplified acceleration range

$\gamma =$ % critical damping taken as 5%

$q = 3.0(5)^{-0.3} = 1.851$

$r = 0.48(5)^{-0.08} = 0.422$

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SHEET NO. 10

JOB NO. ______ JOB HCLPF Study BY RPC DATE 9/30/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D VKR DATE 10/6/87

$$F_\mu = \left[2.851(\mu) - 1.851\right]^{0.422}$$

Modify μ for duration

$$\mu^* = 1.0 + C_D(\mu - 1.0)$$

For steel, $C_D \sim 1.0$

$$\mu^* = 1.0 + 1.0(\mu - 1.0) = \mu.$$

$$F_\mu = \left[2.851(8) - 1.851\right]^{0.422} = 3.6$$

For rigid range,

$$F_\mu \sim \mu^{0.13}$$

$$F_\mu \sim 8^{0.13} = 1.31$$

Let $F_\mu = 3.6$ at spectral peak at $7\frac{1}{2}$ Hz

$F_\mu = 1.31$ at 33 Hz.

At 16.7 Hz

$$\text{Log } F_\mu = \log 1.31 + \left[\frac{\log 33 - \log 16.7}{\log 33 - \log 7.5}\right]\left[\log 3.6 - \log 1.31\right]$$

$$\text{Log } F_\mu = 0.319$$

$$F_\mu = 2.08$$

Effective peak study shows that F_μ can approach 1.0, thus assume that $F_\mu = 1.0$ is 1% (-2.33β) case

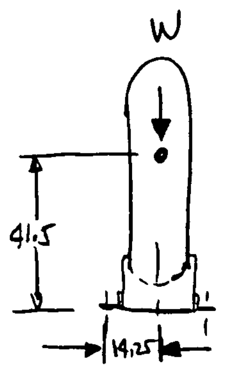
$$\beta_c = \frac{1}{2.33} \ln \frac{2.08}{1} = 0.32$$

$$\beta_R = 0.8 \beta_c = 0.26$$

$$\beta_U = 0.6 \beta_c = 0.19$$

Strength Factor

Bolt reaction at limit load = 1825# (Pg. 6)



Bolt reaction per g lateral = 1275# (Pg. 4)

Bolt reaction per g vertical - ΣMA (A is on edge as shown on Pg. 3.

$$F_b = -\frac{W(1-g_v)(14.25)}{2(14.97)} = -0.476(W)(1-g_v)$$

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SHEET NO. 12

JOB NO. ______ JOB HCLPF Study BY RDC DATE 9/30/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D MKR DATE 10/6/87

From median centered floor spectra,

Sa_H at 16.7 Hz, 5% damping = 0.51

Sa_V (Rigid) = 0.2

Using 100%, 40%, 40% phasing

$$F_b = 0.51(1275) - 0.476(920)(1 - 0.2(0.4))$$

$$F_b = 247$$

Scale up EQ until $F_b = F_L = 1825^{\#}$

$$1825 = F_s(0.51)(1275) - 0.476(920)(1 - F_s(0.2)(0.4))$$

$$1825 = 650F_s - 438 + 35.03F_s$$

$$F_s = 3.30$$

There are a number of uncertainties associated with strength which include the material yield strength, the effective length of the cantilever leg of the bracket and the direction of rocking (ie, the strength model is for one axis). Some quick studies of rocking about other axes and including two horizontal components of EQ

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SHEET NO. 13

JOB NO. ______ JOB HCLPF Study BY RDC DATE 9/30/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D MKR DATE 10/6/87

indicate that the model used is median centered. Phasing will be considered in equipment response factor.

MATERIAL STRENGTH

Code value is 95% Confidence

$$\beta_{U_M} = \frac{1}{1.65} (\ln 1.25) = 0.14$$

Other uncertainty, add $\beta_U = 0.10$

$$\beta_{U_S} = (0.14^2 + 0.10^2)^{1/2} = 0.17$$

EQUIPMENT RESPONSE FACTOR

METHOD OF ANALYSIS -

THERE IS NO DELIBERATE BIAS IN THE ANALYSIS METHOD, THUS, THIS FACTOR DOES NOT APPLY. VARIABILITIES IN ANALYSIS ARE PICKED UP IN EVALUATION OF OTHER VARIABLES.

MODELING

THE TWO CONSIDERATIONS ARE

1) FREQUENCY

2) MODEL ACCURACY

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SHEET NO. 14

JOB NO. ______ JOB HCLPF Study BY RDC DATE 9/30/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D MKR DATE 10/6/87

FREQUENCY

Stiffness of angle support may have been optimistic but seismic excitation in a different direction might have resulted in a stiffer model. Model was simple but crude (rotary inertia was not considered but effect is small)

Estimate that β_F is 0.2

-1β Frequency $= 16.7\, e^{-0.2} = 13.7$ Hz

From horizontal spectrum

$\frac{Sa_{13.7}}{Sa_{16.7}} = 1.0$ No change.

At -2β $f = 16.7\, e^{-2(0.2)} = 11.2$ Hz

$\frac{Sa_{11.2}}{Sa_{16.7}} = \frac{0.70}{0.51} = 1.373$

$\beta_f = \frac{1}{2} \ln 1.373 = 0.16 = \beta_U$

MODEL

Use $\beta_U = 0.1$ for error in actual model for representing bolt reaction.

β MODELING

$\beta_{M_U} = (0.16^2 + 0.1^2)^{1/2} = 0.19$

DAMPING

5% Considered Median.

Use 7% as +1β

At 16.7 Hz

$$\beta_D = \frac{Sa_{5\%}}{Sa_{7\%}} = \ln\frac{0.51}{0.50} = 0.02$$

consider all β_U.

Mode Combination

Most dynamic response is rigid body rocking, thus, response is all in a single mode.

Let $\beta_{MC} = \beta_R \sim 0.05$

Earthquake Component Combination

Analysis was done looking at one horizontal component with 40% of vertical component effective. Consider both horizontal components in phase as a +3β case and one horizontal and the vertical component in phase as a +3β case.

Horizontal Phasing

Only a single direction horizontal component was used in the strength calculation.
A median horizontal acceleration vector would be (using 100%, 40%, 40% phasing)

$$\bar{S}_0 = (1^2 + 0.4^2)^{1/2} = 1.08$$

$$F_{ECC} = \frac{1}{1.08} = 0.93$$

$$\beta_{ECC} = \frac{1}{3} \ln \frac{\sqrt{2}}{1.08} = 0.09 = \beta_R$$

Vertical Phasing

Consider H + V in phase.

Use equations on Pg. 11 + 12 for bolt reaction due to horizontal and vertical loading

$$F_b = 1275g - \frac{W(1-g_v)(14.25)}{2(14.97)} = 1275g - 0.476\,W(1-g_v)$$

$$Sa_H = 0.51g$$
$$Sa_V = 0.2g$$

$$F_b = 0.51(1275) - 0.476(920)(1-0.2) = 563\#$$

Scale up to $F_b = F_L = 1825\#$

$$1825 = F_S(0.51)(1275) - 0.476(920)(1-0.2F_S)$$
$$1825 = 650F_S - 438 + 87.6F_S$$
$$F_S = \frac{2263}{737.6} = 3.07 \quad \text{compared to } 3.3$$

$$\beta = \frac{1}{3}\ln\frac{3.3}{3.07} = 0.025 \text{ say } 0.03 \quad \text{all } \beta_R$$

$$F_{RE} = F_m\,F_D\,F_{MC}\,F_{ECC} = (1)(1)(1)(0.93)(1.0) = 0.93$$
$$\beta_{R\,RE} = (0^2 + 0^2 + 0.05^2 + 0.09^2 + 0.03^2) = 0.11$$
$$\beta_{U\,RE} = (0.19^2 + 0.02^2 + 0 + 0)^{1/2} = 0.19$$

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SHEET NO. 17

JOB NO. ______ JOB HCLPF Study BY RDL DATE 9/30/87

CLIENT ______ SUBJECT Air Receiver Tank CHK'D MKR DATE 10/6/87

STRUCTURAL RESPONSE FACTOR

USE PSH ANALYSIS AS APPROPRIATE.

Spectral Shape (Peak to Peak Variation and EQ Direction Content)

✓ $F_{SS} = 1.35$

$\beta_R = 0.25$

Damping –

✓ $F_D = 1.0$

$\beta_R = 0.03$

$\beta_U = 0.14$

Modeling

✓ $F_M = 1.0$

$\beta_R = 0$

$\beta_U = 0.16$

SSI

$F_{SSI} = 1.0$

$\beta_R = 0.01$

$\beta_U = 0.05$

$F_{SR} = 1.35(1)(1)(1) = 1.35$

$\beta_R = (0.25^2 + 0.03^2 + 0 + 0.01^2)^{1/2} = 0.25$

$\beta_U = (0 + 0.14^2 + 0.16^2 + 0.05^2)^{1/2} = 0.22$

FRAGILITY.

$\check{A} = F_S F_\mu F_{RE} F_{RS} \; (0.18) =$

$(3.3)(2.08)(0.93)(1.35)(0.18) = \;\; 1.55 g$

$\beta_R = (\;0\; + 0.26^2 + 0.11^2 + 0.25^2)^{1/2} = 0.38$

$\beta_U = (\;0.17^2 + 0.19^2 + 0.19^2 + 0.22^2)^{1/2} = 0.39$

$HCLPF = 1.55\, e^{-1.65(0.38+0.39)} = 0.44 g$

CALCULATE HCLPF BY EPRI METHOD.

MOUNTING BRACKET WOULD GOVERN

USING APPENDIX F CRITERIA FOR ASME COMPINENT SUPPORT.

FOR Class 3 plate & shell component support, primary bending allowable is 1.5 X Membrane allowable of $1.2 S_y$ or $0.7 S_u$

$0.7 S_u = 0.7(58,000) = 40,600$ psi governs

Allowable = $1.5(40,600) = 60,900$ psi

Our Base Fragility considered one horizontal direction acting in the weakest axis. However, we have two components of horizontal earthquake at the floor level, the 100%, 40%, 40% phasing rule is applicable to the margin criteria, thus we can scale up our horizontal model load by

$$F = (1^2 + 0.4^2)^{1/2} = 1.08$$

$$\sigma_b = \frac{Mc}{I} = \frac{Fb(1.50)(0.125)(1/2)}{3(0.25^3)} = 48\ Fb$$

(Note that l taken as 1.5" to center of bolt hole for elastic deterministic calculation.)

$$Fb = \frac{60,900}{48} = 1269\#$$

From Equations on Pg. 12

$$1269 = 1.08(0.51)F_s(1275) - 0.476(920)(1 - F_s(0.2)(0.4))$$

$$1269 = 702F_s - 438 + 35.03F_s$$

$$F_s = 2.32$$

$$HCLPF = 2.32(0.18) = 0.42\ g$$

In addition, in the EPRI method, we can use 0.8 as seismic load factor

Thus $HCLPF = \frac{0.42}{0.8} = 0.53g$ compared to 0.44g by fragility method.

Alternatively, the code method allows use of limit analysis which in this case would result in the same answer

Note, that if the linear support rules are used and the bending is considered as bending of a compact section, the allowable is

$F(0.75 S_y)$ where F is lesser of 2 or

$1.167 \frac{S_u}{S_y}$ ← governs

$F = 1.88$

$\sigma_{all} = 1.88 (0.75) S_y = 50,764$ compared to 60,900 if angle leg bending is considered a plate bending.

AISC criteria would allow $1.7 (0.75 S_y) = 45,900$ psi

Thus, HCLPF varies for EPRI method depending upon classification of component and interpretation of code.

COMPONENT 4

HORIZONTAL HEAT EXCHANGER

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SHEET NO. 1

JOB NO. 87-218.01 JOB HCLPF STUDY BY RDC DATE 9-26-87

CLIENT LLNL SUBJECT COMPONENT HEAT EXCHANGER CHK'D MKR DATE 10/3/87

DEVELOP FRAGILITY DESCRIPTION OF COMPONENT HEAT EXCHANGER.

REFERENCES:
1) ATLAS INDUSTRIAL MFR. CO. DRAW D-1260-4 31-240 COMPONENT COOLING HEAT EXCHANGER, FEB. 7, 1967
2) WOODWARD CLYDE CALCULATIONS OF COMPONENT COOLING HEAT EXCHANGER FOR GINNA, 3-4-80
3) GILBERT ASSOCIATES CALCULATIONS, COMPONENT COOLING HEAT EXCHANGER, FILE 1:36.2/C-1, 6-21-79.

ASSUMPTIONS AND GROUND RULES:

1. TANK IS MOUNTED AT GRADE (DO ALSO FOR CASE OF TANK MOUNTED IN STRUCTURE)
2. EARTHQUAKE IS DEFINED AS 84% NEP FOR THE LARGER OF TWO COMPONENTS OF E.Q.
3. FLOOR SPECTRA ARE FROM BUILDING ANALYSIS USING 7% DAMPING BUT ASSUME THAT THEY ARE MEDIAN CENTERED. ZPA = 0.18 g.
4. NOZZLE LOADS ARE NOT PROVIDED, HENCE IGNORE NOZZLE LOADS IN ANALYSIS.

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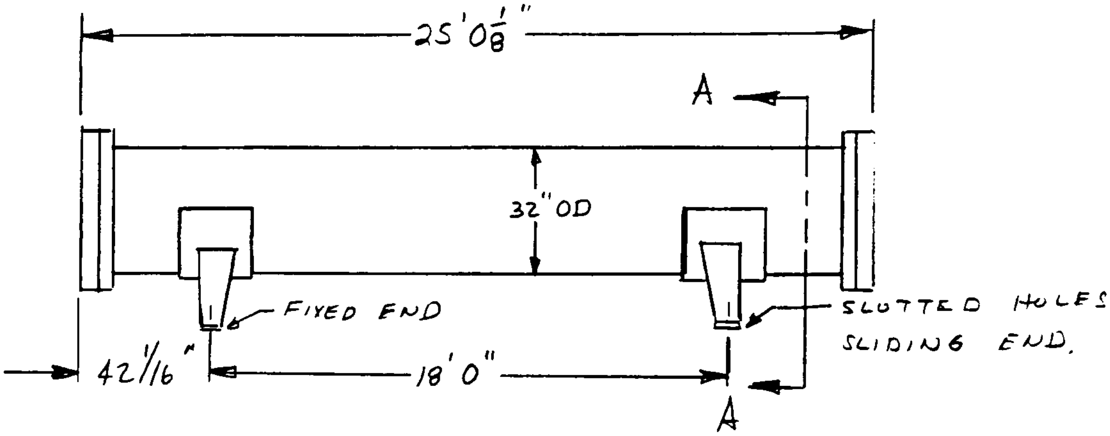
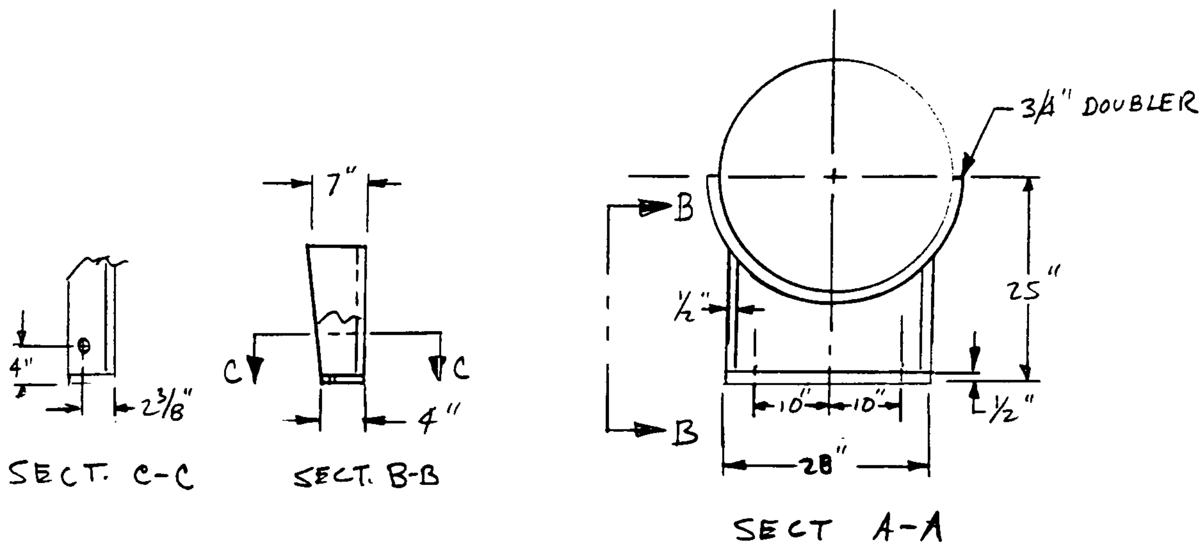
SHEET NO. 2

JOB NO ______ JOB HCLPF Study BY RDC DATE 9-26-87

CLIENT ______ SUBJECT Heat Exchanger CHK'D MKF DATE [illegible]





WT - EMPTY = 15,000 LB

FULL = 23,500 LB

HEAT EXCHANGER IS MOUNTED ON RIGID FRAME.

DETERMINE GOVERNING FAILURE MODE AND CALCULATE MEDIAN CAPACITY.

THERE ARE THREE AREAS OF CONCERN:

1) LONGITUDINAL BENDING OF SADDLE AND SADDLE / DOUBLER WELD (NOTE THAT COMPLETE CROSS SECTION IS <u>NOT</u> EFFECTIVE IN BENDING AS WAS ASSUMED IN REFS 2+3.)

2. BOLT LOADING DOMINATED BY LATERAL LOADS.

3. BASE PLATE BENDING FROM BOLT REACTION (NOTE, REFS 2+3 USED INCORRECT DIMENSIONS FOR THIS ANALYSIS).

CASE 1, LONGITUDINAL BENDING OF SADDLE.

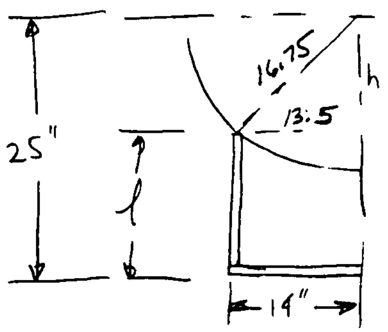
USE ONLY LONGITUDINAL EQ LOAD FOR PRELIMINARY CHECK ON RELATIVE STRENGTHS.

ESTIMATE LONGITUDINAL FREQUENCY

ONLY ONE SADDLE CARRIES LONGITUDINAL LOAD.

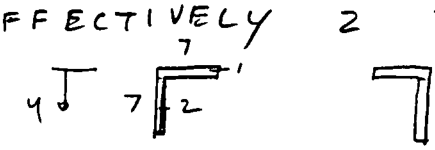
SADDLE ACTS AS A CANTILEVER BEAM

FIXED AT THE DOUBLER PLATE.



$$l = 25 - (16.75^2 - 13.5^2)^{1/2} = 15.08''$$

COMPLETE SADDLE CROSS SECTION IS NOT EFFECTIVE IN BENDING. ASSUME THAT THE SADDLE IS EFFECTIVELY 2 7X7X½ ANGLES, OF l = 15.08''



Element	A	y	Ay
1	3.5	0.25	0.875
2	3.25	3.75	12.187
	6.75		13.06

$$\bar{y} = \frac{13.06}{6.75} = 1.935$$

Ele.	A	d	Ad^2	I_0
1	3.5	1.685	9.937	0.073
2	3.25	1.815	10.706	11.443
			20.643	11.515

$I = 32.158\ IN^4$ PER ANGLE

$I = 64.32\ IN^4$

1g static deflection is:

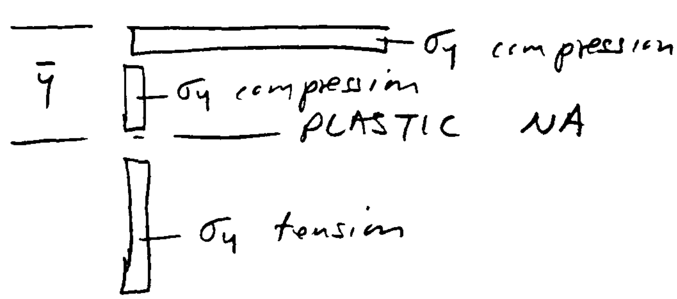
$$\delta = \frac{P\ell^3}{3EI} = \frac{23,500\,(15.08)^3}{3\,(29\times10^6)(61.32)} = 0.0144''$$

$$K = \frac{23,500}{0.0144} = 1.63\times10^6$$

$$f_n = \frac{1}{2\pi}\sqrt{\frac{K}{m}} = \frac{1}{2\pi}\sqrt{\frac{1.63\times10^6\,(386.4)}{23,500}} = 26\ Hz.$$

SYSTEM is ALMOST RIGID.

LOOK AT PLASTIC CAPACITY OF SADDLE WELD TO DOUBLER, NO WELD DETAIL IS GIVEN. ASSUME THERE IS SUFFICIENT WELD METAL TO DEVELOP FULL STRENGTH OF PLATE. THUS THE CAPACITY IS BASED ON A 7X7X½ ANGLE BEING EFFECTIVE



Use σ_y median of A 516-60 ~ 44 ksi

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			SHEET NO. 6
JOB NO.	JOB HCLPF Study	BY RDC	DATE 9-26-87
CLIENT	SUBJECT Heat Exchanger	CHK'D MCR	DATE 10/4/87

PLASTIC NA BY TRIAL. ASSUME $\bar{y} = 1\frac{3}{4}''$

ΣM_{NA} Try $\bar{y} = 1\frac{3}{4}$

$$\sigma_y(\tfrac{1}{2})(7)(1\tfrac{1}{2}) + \sigma_y(1\tfrac{1}{4})(\tfrac{1}{2})(\tfrac{5}{8}) = \sigma_y(\tfrac{1}{2})(5\tfrac{1}{4})(2\tfrac{5}{8})$$

$$5.64\,\sigma_y = 6.89\,\sigma_y$$

Try $\bar{y} = 2''$

$$\sigma_y(\tfrac{1}{2})(7)(1\tfrac{3}{4}) + \sigma_y(1\tfrac{1}{2})(\tfrac{1}{2})(\tfrac{3}{4}) = \sigma_y(\tfrac{1}{2})(5)(2\tfrac{1}{2})$$

$$6.687\,\sigma_y = 6.25\,\sigma_y$$

Try 1.93''

$$\sigma_y(\tfrac{1}{2})(7)(1.68) + \sigma_y(1.43)(0.715)(\tfrac{1}{2}) = \sigma_y(\tfrac{1}{2})(5.07)(2.535)$$

$$6.39 = 6.926\,\sigma_y$$

NA ~ 1.93''

Moment capacity per leg = $(6.39 + 6.93)\,\sigma_y = 12.82\,\sigma_y$

Capacity of both legs = $2(12.82)\,\sigma_y$

$$M = 2(12.82)(44000) = 1.128 \times 10^6 \text{ IN}\#$$

$M_{applied} = 15.08\ W\ Sa$

$$Sa_{LIMIT} = \frac{1.128 \times 10^6}{15.08\,(23{,}500)} = 3.18\ g's$$

CAPACITY COULD BE LOWER IF WELD IS NOT CAPABLE OF DEVELOPING PARENT METAL STRENGTH BUT SOME CREDIT MAY BE TAKEN FOR DUCTILITY.

CHECK CAPACITY IN BENDING IN OPPOSITE DIRECTION. BUCKLING OF GUSSET PLATE WOULD CONTROL. GUSSET PLATE DOES NOT QUALIFY AS A COMPACT SECTION

PER NF 3322.1 (d)(1)(a) PER NF 3322.1(d)(1)(b)(2)

$b_f/2t_f = 7/2(0.5) = 7 < 65/\sqrt{S_y} = 65/\sqrt{44} = 9.8$

$F_b = S_y\left[0.79 - 0.002\,(b_f/2t_f)\,\sqrt{S_y}\right] = 0.70F_y$

0.66 F_y for Compact sections would govern

Faulted Allowable would be the smaller of

$2(F_b)$ or $1.167\left(\frac{S_u}{S_y}\right)F_b$

Use Specified values for $\frac{S_u}{S_y} = \frac{60}{35}$

$1.167\,\frac{S_u}{S_y}\,F_b = 2.0F_b = 2(0.66) = 1.32F_y$

$M_{Faulted} = \frac{1.32 F_y I}{c} = \frac{(1.32)(44)(64.32)}{5.065} = 737$ in k

There is nominally a FS of 1.5 remaining for buckling in faulted allowables.
Estimated median buckling capacity is ~ 1.5 (737 in k) = 1.106 x 10^6 in# compared to full plastic hinge capacity of 1.128 x 10^6 in#

Check other failure modes. If this mode governs, refine the buckling calculation.

(Note that bolts were found to have significantly less capacity so concerns about buckling and weld area are relieved)

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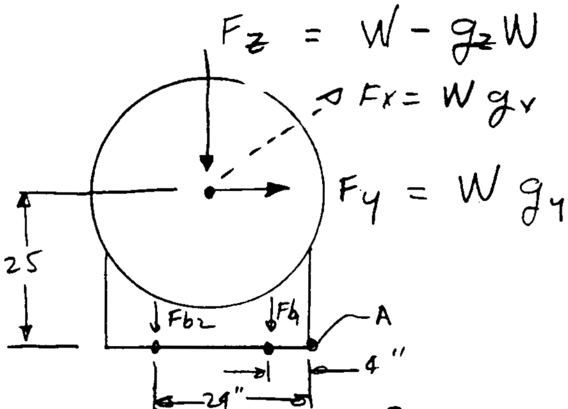
SHEET NO. 9

JOB NO. ______ JOB HCLPF Study BY RDC DATE 9/26/87

CLIENT ______ SUBJECT Heat Exchanger CHK'D MKR DATE 10/2/87

CHECK BOLT STRENGTH.



Bolts see direct shear from X and Y loads and direct tension from F_y overturning. Tank is rigid in the vertical direction and is about 26 Hz in X direction. Assume y direction is rigid. At ground, the vertical acceleration is 2/3 of horizontal. In the structure, the V/h ratio is lower. (~1/2). Use ground for this relative strength calculation. At 26 Hz the Sa is about 1.13X ZPA. For median calculations, assume $g_z = 2/3\, g_y$ and $g_x \sim 0.9\, g_y$

ΣM_A.

$14 F_z - 25 F_y + 2[24 F_{b2} + 4 F_{b1}] = 0$

If Tank and Saddle rotate as a rigid body, $F_{b1} = \frac{4}{24} F_{b2}$

$14(W)(1 - \frac{2}{3} g_y) - 25 W g_y + 2[24 F_{b2} + \frac{4(4)}{24} F_{b2}] = 0$

$329000 - 219333 g_y - 587{,}500 g_y + 49.332 F_{b2} = 0$

$F_{b2} = \frac{806833 g_y - 329000}{49.332} = 16355 g_y - 6669$

with 100%, 40%, 40% phasing

$14(23{,}500)(1 - \frac{0.4(2) g_y}{3}) - 25(23{,}500) g_y + 2[24 F_{b2} + \frac{4(4)}{24} F_{b2}] = 0$

$329{,}000 - 87733 g_y - 587{,}500 g_y + 49.332 F_{b1} = 0$

$F_{b2} = \frac{675233 g_y - 329000}{49.333} = 13687 g_y - 6669$

Shear Stress, use 100%, 40%, 40% Phasing.

$g_x = 0.4(0.9)(1.13) g_y = 0.406 g_y$

4 bolts take y load, 2 bolts take x load.

$f_v = \frac{23{,}500 g_y}{4} + \frac{0.406 g_y (23{,}500)}{2} = 10{,}645 g_y$ #/bolt

If threads in bolt dont take direct shear then the failure plane is one for either shear and tension on the full area or tension only on the thread area.

Tension only on thread area.

Bolts are 7/8"-9 UNC A-307, S_y = 36 ksi, S_u = 58 ksi } Code Values

Stress area = 0.461 in²
Full area = 0.601 in²

At code UTS,

$$F_{b.} = 0.461(58{,}000) = 13687 g_y - 6669$$

$$g_y = 2.44 < \text{saddle/Doubler weld.}$$

For Tension + Shear in full area.

Use Code Parabolic Equation and $F_v = 0.6 F_t$

$$\left(\frac{f_t}{F_t}\right)^2 + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0$$

$$\left(\frac{13687 g_y - 6669}{(0.601)(58{,}000)}\right)^2 + \left(\frac{10{,}695 g_y}{0.6(0.601)(58000)}\right)^2 \leq 1.0$$

$$0.154 g^2 - 0.150 g + 0.0366 + 0.259 g^2 \leq 1.0$$

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SHEET NO. 12

JOB NO. ____ JOB HCLPF Study BY RDC DATE 9/26/87

CLIENT ____ SUBJECT Heat Exchanger CHK'D MKR DATE 10/4/87

$$0.413 g^2 - 0.150 g - 0.9634 = 0$$

$$g_y = \frac{0.150 \pm \sqrt{0.150^2 + 4(0.413)(0.9634)}}{2(0.413)}$$

$g_y = 1.72$ governs bolt failure.

Actual ultimate material strength may be used. For low strength carbon steel such as A 307, the mean UTS is about 1.2 times code. (Ref. NUREG/CR-2137, SA-106 B STL).

$g \sim 1.72(1.2) = 2.06$ g capacity

Now, see if plate can develop the ultimate strength of the bolt.

$F_{ULT} = 13687 g_y - 6669 =$
$13687(2.06) - 6669 = 21,526$ #

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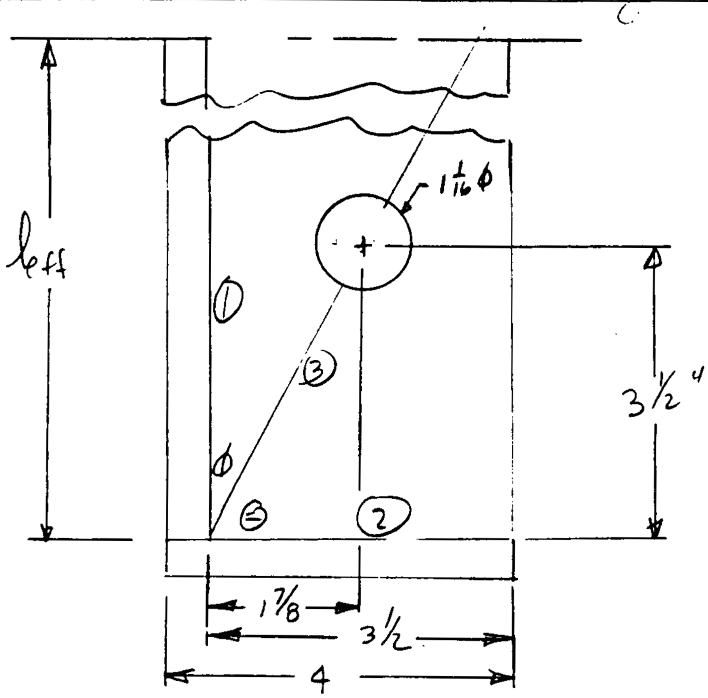
SHEET NO. 13

JOB NO. ______ JOB HCLPF Study BY RPC DATE 9-26-87

CLIENT ______ SUBJECT Heat Exchanger CHK'D MKR DATE 10/4/87

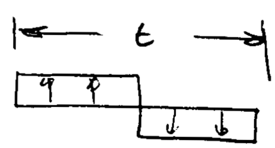


NO WELD DETAIL GIVEN. ASSUME FULL PENETRATION

ASSUME 3 YIELD LINES FORM AT ①, ② & ③

$l_{eff} = 2(3\frac{1}{2}'') = 7''$

Limit Moment =



$$M_L = 2\left[\sigma_y \frac{t}{2} \cdot \frac{t}{4}\right] = \frac{\sigma_y t^2}{4} / in$$

$$\sigma_y \approx 44 \text{ ksi}$$

$$M_{L_1} = \frac{7(44)(0.5^2)}{4} = 19.25 \text{ in k.}$$

$$M_{L_2} = \frac{3.5(44)(0.5^2)}{4} = 9.625 \text{ in k}$$

$$M_{L_3} = \frac{\left[(7^2+3\frac{1}{2}^2)^{1/2} - 1\frac{1}{16}\right] 44(0.5^2)}{4} = 18.60 \text{ in k}$$

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SHEET NO. 14

JOB NO ______ JOB HCLPF study BY RDC DATE 9-26-87

CLIENT ______ SUBJECT Heat Exchanger CHK'D MKR DATE 10/4/87

$$Work = F_b \delta = M_{L1}\alpha_1 + M_{L2}\alpha_2 + M_{L3}\alpha_3 + M_{L3}\alpha_4$$

$$\alpha_i = \frac{\delta}{l_i}$$

$$\alpha_1 = \frac{\delta}{1.875}$$

$$\alpha_2 = \frac{\delta}{3.5}$$

$$\alpha_3 = \alpha_1 \cos\phi \quad \text{where } \phi = 90^\circ - \tan^{-1}\frac{3.5}{1.875} = 28.179^\circ$$

$$\alpha_4 = \alpha_2 \sin\phi$$

$$F\delta = \delta \sigma_y t^2 \left[\frac{7}{(4)1\frac{7}{8}} + \frac{3.5}{4(3.5)} + \frac{6.76 \cos\phi}{4\ (1\frac{7}{8})} + \frac{6.76 \sin\phi}{4\ (3.5)} \right]$$

$$F = 44000(0.5)^2 \left[0.933 + 0.25 + 0.79 + 0.228 \right]$$

$$F = 24211^{\#}$$

Bolt tension at ultimate failure in shear and tension is 22,622#, thus the bottom plate can develop sufficient resistance to fail the attachment bolts.

Develop fragility curve based on bolt failure.

Recalculate capacity of bolts in terms of g_y using in-structure spectra. Ref. Pg. 10 for equations.

$$g_x\ ZPA = 90\%\ g_y\ ZPA$$

$$g_x \text{ at } 26\ Hz = 0.9\left(\frac{0.45}{0.38}\right) g_y = 1.066\ g_y$$

$$g_z = \frac{0.2}{0.38}\ g_y = 0.526\ g_y$$

Bolt tension using 100%, 40%, 40% Phasing

$$14\ W\left(1-0.4(.526)g_y\right) - 25\ W g_y + 2\left[24 F_{b2} + \frac{4(4)}{24} F_{b2}\right] = 0$$

$$329{,}000 - 69222\ g_y - 587{,}500 g_y + 49.332 F_{b2} = 0$$

$$F_{b2} = \frac{656722\ g_y - 329{,}000}{49.332} = 13{,}312 g_y - 6669$$

Bolt Shear, Using 100%, 40%, 40% Phasing

$$\frac{23{,}500\ g_y}{4} + \frac{0.4(1.066) g_y (23{,}500)}{2} = 10{,}885 g_y \ \#/\text{bolt}.$$

$$\left(\frac{13,312\,g_y - 6669}{0.601(58000)}\right)^2 + \left(\frac{10,885\,g_y}{0.6(0.601)(58,000)}\right)^2 = 1.0$$

$$.1458\,g_y^2 - 0.1461\,g_y + 0.0366 + 0.2709\,g_y^2 = 1.0$$

$$0.4167\,g_y^2 - .1461\,g_y - 0.9634 = 0$$

$$g_y = \frac{0.1461 \pm \sqrt{.1461^2 + 4(.4167)(.9634)}}{2(.4167)}$$

$$g_y = 1.7058$$

For medium ult strength

$$g_y = 1.2(1.7058) = 2.05\ g$$

CAPACITY FACTOR

Median capacity for anchor bolt failure was 2.06 g Sa (Pg. 12)

Code strength is 95% confidence so if median is ~ 1.2 x code.

$$\beta_U = \frac{1}{1.65} \ln 1.2 = 0.11$$

There is additional uncertainty in the failure strength and failure mode of attachment bolts in combined tension and shear. Estimate β_U ~ 0.15. In addition, because of clearances in the bolt holes and local bending effects, the bolts will not be uniformly loaded as calculated. Assume 10% reduction in strength.

The resulting strength is then

$$\check{S}_a = \frac{2.06}{1.1} = 1.87\,g \text{ in y dir.}$$
assumed to be max direction

$$\beta_U = \sqrt{0.11^2 + 0.15^2} = 0.19$$

$$F_s = \frac{1.87}{0.3} = 6.23 \text{ for ground mounted tank}$$

(Note that y dir is rigid & ZPA is reference acceleration)

For In Structure Case (Pg. 16)

$$ZPA = 2.05/1.1 = 1.86 \text{ in y dir}$$

$$F_s = \frac{1.86}{0.38} = 4.90$$ relative to in structure ZPA in y dir.

DUCTILITY - Component is rigid, failure is brittle and UTS used for strength; Ductility NA.

EQUIPMENT RESPONSE FACTOR

The variables are:

Method of analysis

Modeling

Damping

Mode Combination

Earthquake component combination

Method of Analysis

NA for rigid body evaluation
modeling error covered below

Modeling

There are two considerations,

1) frequency error
2) model accuracy

The lowest fundamental frequency is ~26 Hz. If the ± 1β on frequency is ~0.15 the −1β frequency is

$$f_{-1\beta} = 26e^{-0.15} = 22.3 \text{ Hz}.$$

From either the ground spectrum or the in-structure spectrum the ratio of $Sa_{22.3}/Sa_{26}$ is about 1.11.

$$\beta_f = \ln 1.11 = 0.10$$

Estimate the error in the hand calculation method to be represented by β ~ 0.10

$$\beta_M = \beta_U = (0.10^2 + 0.10^2)^{1/2} = 0.14$$

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SHEET NO. 20

JOB NO. ______ JOB HCLPF Study BY RPC DATE 4/28/87

CLIENT ______ SUBJECT Heat Exchanger CHK'D MKR DATE 10/4/87

Damping

At 26 Hz, there is very little difference in Sa for different damping values. From the in structure spectra curves, there is only about 5% difference between 4% and 7%. For 5% [illegible] damping and 7% representing a [illegible]

[illegible] CONSIDER ALL [illegible]

ξ_D would be similar for a ground mounted tank.

Mode Combination

NA for a rigid body response.

Earthquake Component Combination

The 100%, 40%, 40% rule was used in the capacity analysis as a median value.

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JOB NO. ______ JOB HCLPF Study BY RDC DATE 9/28/87

CLIENT ______ SUBJECT Heat Exchanger CHK'D MKR DATE 10/4/87

Consider all components in phase to be a +3β condition. Recalculate bolt strength assuming all components in phase, Ref Page 10 for capacity equation for ground case

$$F_{b1} = 16,355 g_y - 6669$$

$$f_v = \frac{23,500 g_y}{4} + \frac{0.9(1.13)(23,500) g_y}{2} = 17,825 g_y \ \#/bolt$$

For tension and shear combined on full body,

$$\left(\frac{16,355 g_y - 6669}{0.601 (58,000)}\right)^2 + \left(\frac{17825 g_y}{0.601 (0.6)(58000)}\right)^2 \le 1.0$$

$$0.2201 g_y^2 - 0.1795 g_y + 0.0366 + 0.726 g_y^2 \le 1$$

$$0.946 g^2 - 0.1795 g - 0.9634 = 0$$

$$g = \frac{0.1795 \pm \sqrt{0.1795^2 + 4(0.946)(0.9634)}}{2(.946)} =$$

$g = 1.108$ based on code properties.

$g = 1.2(1.108) = 1.33$ based on median strength

$$\beta_{ECC} = \frac{1}{3} \ln \frac{2.06}{1.33} = 0.15$$

All β_R.

$$F_{RE} = 1.0$$

$$\beta_{R_{RE}} = 0.15$$

$$\beta_{U_{RE}} = \left(0.14^2 + 0.03^2\right) = 0.14$$

ASSUME SAME β's FOR IN STRUCTURE TANK

STRUCTURAL RESPONSE

Use FRS for block walls (Pg 30) for in structure mounted tank except, effects of frequency shift for inelastic structure are not of concern but effect of inelasticity on in spectra ZPA may be considered to be about the same. Relationship between EQ direction magnitude has been included in fragility calculation but we need to include uncertainty.

$$F_{RS} = F_{SS}\, F_{EQ\,Dir}\, F_D\, F_M\, F_{SSI}\, F_{INEC}$$
$$= 1.22(1.0)(1.0)(1.0)(1.0)(1.0)$$

(Note, E.Q. Dir factor included in capacity calc thus, $F_{ED} = 1.0$)

$$F_{RS} = 1.22$$

$$\beta_R = (0.20^2 + .15^2 + 0.03^2 + 0 + 0.01^2 + 0.06)^{1/2} = 0.26$$

$$\beta_U = (0^2 + 0 + 0.14^2 + 0.16^2 + 0.05^2 + 0.06^2)^{1/2} = 0.23$$

For ground mounted Tank.

Peak to peak variation is not pertinent to rigid tank.

The E.Q direction content β_R of ~0.15 is appropriate.

Structural Damping, structural modeling and inelastic structural response are not applicable.

Use SSI uncertainties

$$F_{RS} = F_{EQ\,Dir} \cdot F_{SSI} = 1.0$$
$$\beta_R = (0.15^2 + 0.01^2)^{1/2} = 0.15$$
$$\beta_U = (0.0 + 0.05^2)^{1/2} = 0.05$$

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SHEET NO. 24

JOB NO. ______ JOB HCLPF Study BY RPC DATE 9/29/87

CLIENT ______ SUBJECT Heat Exchanger CHK'D MKR DATE 10/4/87

FRAGILITY

GROUND MOUNTED TANK.

$$\check{A} = 0.3 (F_C)(F_{RE})(F_{RS})$$
$$= 0.3 (6.23)(1.0)(1.0)$$
$$\check{A} = 1.87 g$$
$$\beta_R = (0^2 + 0.15^2 + 0.15^2)^{1/2} = 0.21$$
$$\beta_U = (0.19^2 + 0.14^2 + 0.05^2)^{1/2} = 0.24$$
$$HCLPF = 1.87 e^{-1.65 (0.21+0.24)} = 0.89 g.$$

IN STRUCTURE TANK

$$\check{A} = 0.18 (F_C)(F_{RE})(F_{RS})$$
$$= 0.18 (4.90)(1.0)(1.22)$$
$$\check{A} = 1.08 g$$
$$\beta_R = (0^2 + 0.15^2 + 0.26^2)^{1/2} = 0.30$$
$$\beta_U = 0.19^2 + 0.14^2 + 0.23^2)^{1/2} = 0.33$$
$$HCLPF = 1.08 e^{-1.65 (0.30+.33)} = 0.38 g$$

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SHEET NO. 25

JOB NO. ______ JOB HCLPF Study BY RPC DATE 9/27/87

CLIENT ______ SUBJECT Heat Exchanger CHK'D MKR DATE 9/.../87

USING EPRI MARGIN METHOD.

84% Non Exceedance EQ.

Median Centered Spectrum in structure.

Faulted Allowable for A-307 Ferritic steel Bolts is:

F_{tb} = lesser of 0.7 S_u or S_y

S_y = 36 ksi governs

F_{vb} = lesser of 0.42 S_u or 0.6 S_y

0.6 S_y = 21.6 ksi governs.

GROUND MOUNTED TANK.

EPRI Margin Criteria Use 100%, 40%, 40% rule with equal horizontal components.

Recompute g capacity from equations on page 10

$\Sigma M_A = 0$

$F_{bz} = 13{,}687\, g_y - 6669$

For shear

$g_x = 0.4(1.13)\,g_y$ at 26 Hz.

$$f_v = \frac{23{,}500\,g_y}{4} + \frac{0.4(1.13)g_y(23{,}500)}{2} = 11186\,g_y \ \#/\text{bolt}$$

$$\left(\frac{f_t}{F_{tb}}\right)^2 + \left(\frac{f_v}{F_{vb}}\right)^2 \le 1.0$$

$$\left(\frac{13{,}687\,g_y - 6669}{0.601\,(36{,}000)}\right)^2 + \left(\frac{11{,}186\,g_y}{21{,}600\,(0.601)}\right)^2 \le 1.0$$

$$0.400\,g_y^2 - 0.390\,g_y + 0.095 + 0.7425\,g_y^2 \le 1.0$$

$$1.1425\,g_y^2 - 0.39\,g_y - 0.905 = 0$$

$$g_y = \frac{0.39 \pm \sqrt{0.39^2 + 4(1.1425)(0.905)}}{2(1.1425)}$$

HCLPF = g_y = 1.077g compared to 0.89 g by Fragility Method.

In Structure Case – EPRI method

Ref Pg. 15 for Equations.

$g_x = \frac{0.95}{0.30} g_y = 1.184\, g_y$ at 26 Hz.

$g_z = 0.526\, g_y$

Tension

$f_{b_2} = 13312 g_y - 6669$

Shear.

$f_{vb} = \frac{23{,}500 g_y}{4} + \frac{0.4(1.184)\, g_y\, (23{,}500)}{2} = 11{,}440 g_y$ #/rdt

$$\left(\frac{13312 g_y - 6669}{0.601(36000)}\right)^2 + \left(\frac{11440\, g_y}{0.601(21600)}\right)^2 \leq 1.0$$

$$0.3786 g_y^2 - 0.3793 g_y + 0.095 + 0.7766 g_y^2 = 1.0$$

$$1.1552 g_y^2 - 0.3793\, g_y - 0.905 = 0$$

$$g_y = \frac{0.3793 \pm \sqrt{0.3793^2 + 4(1.1552)(0.905)}}{2(1.1552)}$$

$g_y = 1.06\, g$

HCLPF $= \frac{1.06}{0.38}(0.18) = 0.50g$ compared to 0.38g by fragility method.

Compare pure shear in anchor bolts for CCW HX to case analyzed for tension due to overturning and shear.

Bolt shear in anchored end is

$$f_v = \left[\left(\frac{W g_x}{2A_b}\right)^2 + \left(\frac{(0.4) W g_y}{4 A_b}\right)^2\right]^{\frac{1}{2}}$$

$A_b = .601\ in^2$ in full section

$W = 23,500$

In structure -

$g_x = .45$
$g_y = .9(.38)$ for 0.18g PGA

$F_v - 0.6\ S_y = 0.6(36) = 21.6\ ksi$

$$f_v = \left[\left(\frac{23.5(.45)}{2(.601)}\right)^2 + \left(\frac{(0.4)23.5(.9)(.38)}{4(.601)}\right)^2\right]^{1/2}$$

$f_v = 8.9\ ksi$

$HCLPF = \frac{21.6}{8.9}(.18) = 0.44$ vs .50 for OT case.

Use HCLPF = 0.44g for CDFM method for Tank mounted in structure. (Different failure mode than median calculation)

Recompute CDFM HCLPF for ground mounted case considering pure shear in the anchor bolts.

From pg. 9, at 26 Hz in longitudinal (x) direction, the g_x is ~ 1.13 x ZPA. Let lateral ZPA (g_y) = 0.9 longitudinal ZPA (g_x).

$$g_x = 1.13(.3) = .34$$

$$g_y = 0.9(.3) = .27$$

$$f_v = \left[\left(\frac{W g_x}{2A_b}\right)^2 + \left(\frac{0.4 W g_y}{4 A_b}\right)^2\right]^{\frac{1}{2}}$$

$$f_v = \left[\left(\frac{23.5(.34)}{2(.601)}\right)^2 + \left(\frac{0.4(23.5)(.27)}{4(.601)}\right)^2\right]^{\frac{1}{2}} = 6.73\ ksi$$

$$HCLPF = \frac{21.6}{6.73}(.3) = 0.96 \quad \text{vs } 1.08 \text{ for OT case}$$

Use HCLPF = 0.96 g for ground mounted tank.

$$\beta_c = -\frac{1}{3} \ln \frac{1.0}{2.1}$$

$$= 0.25$$

$$\beta_R = \beta_U = \frac{1}{\sqrt{2}} (0.25)$$

$$= 0.17$$

Spectral Shape Factor (EQ Definition) – Structure Response

The earthquake is defined as the 84% non-exceedance probability spectrum within the frequency range of interest for the larger horiz. component of motion.

Peak to Peak Variation.

Estimate peak to peak variability, $\beta_{PP} \approx 0.20$

$$\check{F}_{PP} = e^{0.20}$$

$$= 1.22$$

EQ Directional Content

The calculated response is based on the larger of the two horiz. components. Wall failure results from only one of the components. The following values are estimated:

$$\check{F}_{\psi} = 1.1$$

$$\beta_{\psi} = 0.15$$

Total

$\check{F}_{SA} = 1.22(1.1)$
$= 1.35$

$\beta_R = \sqrt{0.20^2 + 0.15^2}$
$= 0.25$

Damping - Structure Response

Floor spectra were generated for 10% structure damping, which is judged to be median centered,

$\check{F}_\delta = 1.0$

Variability

Estimate 7% damping is an 84% exceedance value.

Median amplification factor, 10% damping = 1.64 } NUREG/CR-0098
" " 7% damping = 1.89 }

$\beta_U = \ln \frac{1.89}{1.64}$
$= 0.14$

Estimate $\beta_R = 0.2\beta_U$
$= 0.03$

Modeling Factor - Structure Response

The building model is assumed to be median-centered.

$\check{F}_M = 1.0$

Estimate $\beta_{ms} \approx 0.15$
$\beta_f \approx 0.30$ } Typical values

Fundamental freq. ≈ 7.2 Hz

$f_{+1\beta} = 7.2e^{0.30}$
$= 9.7$ Hz

$$\log AF = \frac{\log 1.64 - \log 1}{\log 8 - \log 33} (\log 9.7 - \log 33) + \log 1$$

$AF = 1.53$

$\beta_{mf} = -\ln \frac{1.53}{1.64}$
$= 0.07$

$f_{-1\beta} = 7.2e^{-0.3}$
$= 5.3$ Hz

$\beta_{mf} = 0$

Estimate $\beta_{mf} = 0.04$

$\beta_U = \sqrt{0.15^2 + 0.04^2}$
$= 0.16$

Soil-Structure Interaction

$F_{SSI} = 1.0$
$\beta_R = 0.01$
$\beta_U = 0.05$
} Nominal values for rock founded structures

Structure Inelastic Response

Structure inelastic response results in a reduction in structure frequency, which would imply an increase in block wall response. However, inelastic response of the block wall causes a reduction in wall frequency and resulting shift away from the floor spectrum peak. The block wall response as calculated is judged to be median-centered. Per Lin & Mahin, the mean amplification factor for high period eqmt. response due to nonlinear structure response is about 1.1 for stiffness degrading structures. Estimate this is an upper bound for a nonlinear component.

$$\beta_R \approx \beta_U = \frac{1}{1.65} \ln \frac{1.1}{1}$$
$$= 0.06$$

COMPONENT 5

REINFORCED BLOCK WALL

EQE

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SHEET NO. ____

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-30-87

CLIENT LLNL SUBJECT Block Wall CHK'D MKR DATE 10-5-87

BLOCK WALL HCLPF CAPACITY

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SHEET NO. BW-1

JOB NO. 8728.01 JOB HCLPF Comparison BY PSH DATE 9-28-87

CLIENT LLNL SUBJECT Block Wall CHK'D MKR DATE 10/5/87

Summary

The sample block wall ultimate strength and load-deflection relationship is determined following the recommendations of the ACI-SEASC Task Committee On Slender Walls (Ref. 1). Fragilities are developed for two cases using different stiffness assumptions:

1. Equivalent elastic-plastic load-deflection curve.
2. Secant stiffness

The HCLPF capacity for the first case is calculated to be 0.32g. This value is likely conservative since it doesn't explicitly account for the wall response shifting down from the rising part of the floor spectrum. The HCLPF capacity for the second case is calculated to be 0.48g.

The block wall HCLPF capacity by the fragility analysis is the greater of the two calculated values:

HCLPF capacity = 0.48g.

By the CDFM method, the HCLPF capacity is also calculated to be 0.48g.

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SHEET NO. BW-2

JOB NO. 2?18.01 JOB HCLPF Comparison BY PSH DATE 9-18-87

CLIENT ~~~ SUBJECT Block Wall CHK'D NKR DATE 10/5/87

Material Properties

Reinforcement

Estimated values for Grade 60 rebar:

f_{ym} = 69 ksi

COV = 0.07

Masonry Compressive Strength

Average unit strength = 3000 psi

f_m' = 1700 psi Per Table 4.3, ACI 531-79

Estimated values:

Median f_m' ≈ 2000 psi

Mortar Compressive Strength

Estimate value for Type S mortar

mor = 1.4 (1800)

= 2500 psi

COV ≈ 0.2

Other Necessary Data

Other data necessary to determine the wall HCLPF capacity is contained in the transmittal to the HCLPF Study Group (Ref. 4).

The force-deformation curve will be established following the recommedations of Ref. 1. This approach appears to be consistent with the results of other testing shown in Figures 3.7 and 3.8 of Ref. 2.

Cracking Load and Deflection

Cracking Stress

Per Ref. 1

$\sigma_{cr} = 3\sqrt{f_m'}$

$= 3\sqrt{2000}$

$= 134$ psi

$\frac{2.5\sqrt{f_m'}}{\phi} = \frac{2.5\sqrt{f_m'}}{0.8} = 3\sqrt{f_m'}$

p. 7-20, Ref. 1

Alternatively, estimate mortar modulus of rupture is 3 times the ACI 531 allowable

$\sigma_{all} = 1.0\sqrt{m_o}$ Grouted units

$= 1.0\sqrt{2500}$

$= 50$ psi

$\sigma_{cr} = 3(50)$

$= 150$ psi

Use lesser of the two values (difference is small)

$\sigma_{cr} = 134$ psi

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SHEET NO. BW-4

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-21-87

CLIENT LLNL SUBJECT Block Wall CHK'D NKR DATE 10/5/87

Cracking Load

$S = \frac{1340}{11.63/2}$ $I_{gr} = 1340\ in^4/ft$ Sample calc, p. 3, Ref. 4

$= 230\ in^3/ft$

$M_{cr} = 230(134)$ Neglecting axial load

$= 30{,}800\ in\text{-}lb/ft$

$= 2570\ ft\text{-}lb/ft$

$w_{cr} = \frac{2(2570)}{10^2}$

$= 51\ psf$

Cracking Deflection

$I_{gr} = 1340\ in^4/ft$

$E_m = 1000\ f_m'$

$= 2.0 \times 10^6\ psi$

$\Delta_{cr} = \frac{51(10)^4(1728)}{8(2.0\times10^6)(1340)}$

$= 0.041\ in$

Yield Load and Deflection

Cracked Moment of Inertia

$A_s = 0.31\ in^2/16"$ #5@ 16"

$= 0.233\ in^2/ft$

$n = \frac{29 \times 10^6}{2.0 \times 10^6}$

$= 15$

$d = 5.8"$

Assume $(kd) > t_f$

$\frac{1}{2}(16)(kd)^2 - \frac{1}{2}(5.9)[(kd) - 1.5]^2 - 15(0.31)[5.8 - (kd)] = 0$

$5.05(kd)^2 + 13.50(kd) - 33.61 = 0$

$(kd) = 1.57" > t_f$ ok $t_f = 1.5\ in.$

$I_{cr} = \frac{1}{3}(16)(1.57)^3 - \frac{1}{3}(5.9)(1.57 - 1.5)^3 + 15(0.31)(5.8 - 1.57)^2$

$= 104\ in^4/block$

$= 78\ in^4/ft$

Yield Load

$f_{ym} = 69\ ksi$ p. BW-2

$M_y = \frac{69{,}000\ (78)}{15\ (5.8 - 1.57)}$ Neglecting axial load

$= 84{,}800\ in\text{-}lb/ft$

$= 7070\ ft\text{-}lb/ft$

$$w_y = \frac{2(7070)}{10^2}$$
$$= 141 \text{ psf}$$

Yield Deflection

Per Ref. 1, the additional deflection in excess of that causing cracking is based on the cracked moment of inertia.

$$\Delta_y = 0.041 + \frac{(141-51)(10)^4(1728)}{8(2.0\times10^4)(78)}$$
$$= 0.041 + 1.25$$
$$= 1.29 \text{ in}$$

Ultimate Strength

Per Refs. 1, 2 the ultimate strength can be determined using the equivalent stress block approach developed for reinforced concrete. Axial load acting at an estimated median capacity of about 1.5g PGA will be included. Rebar strain-hardening is neglected.

Effective Axial Load

$P_{DL} = 111(10)$ Wall weighs 111psf, p. 2, sample calc, Ref. 4
$= 1110$ lb/ft

Vert. ZPA $= 0.20g$ 0.18g PGA Ref. 4
$= 1.67g$ 1.5g PGA

$P = 1110[1 - 0.4(1.67)]$ 40% of max. vert response concurrent w/ max horiz. response.
$= 370$ plf

$f_m' = 2000$ psi p. BW-2

$$a = \frac{0.233(69{,}000) + 370}{0.85(2000)(12)}$$

$= 0.81$ in $< t_f$ ok

$$M_u = [0.233(69{,}000) + 370]\left(5.8 - \frac{0.81}{2}\right)$$

88,760 in-lb/ft

$= 7400$ ft-lb/ft

$$w_u = \frac{2(7400)}{10^2}$$

$= 148$ psf

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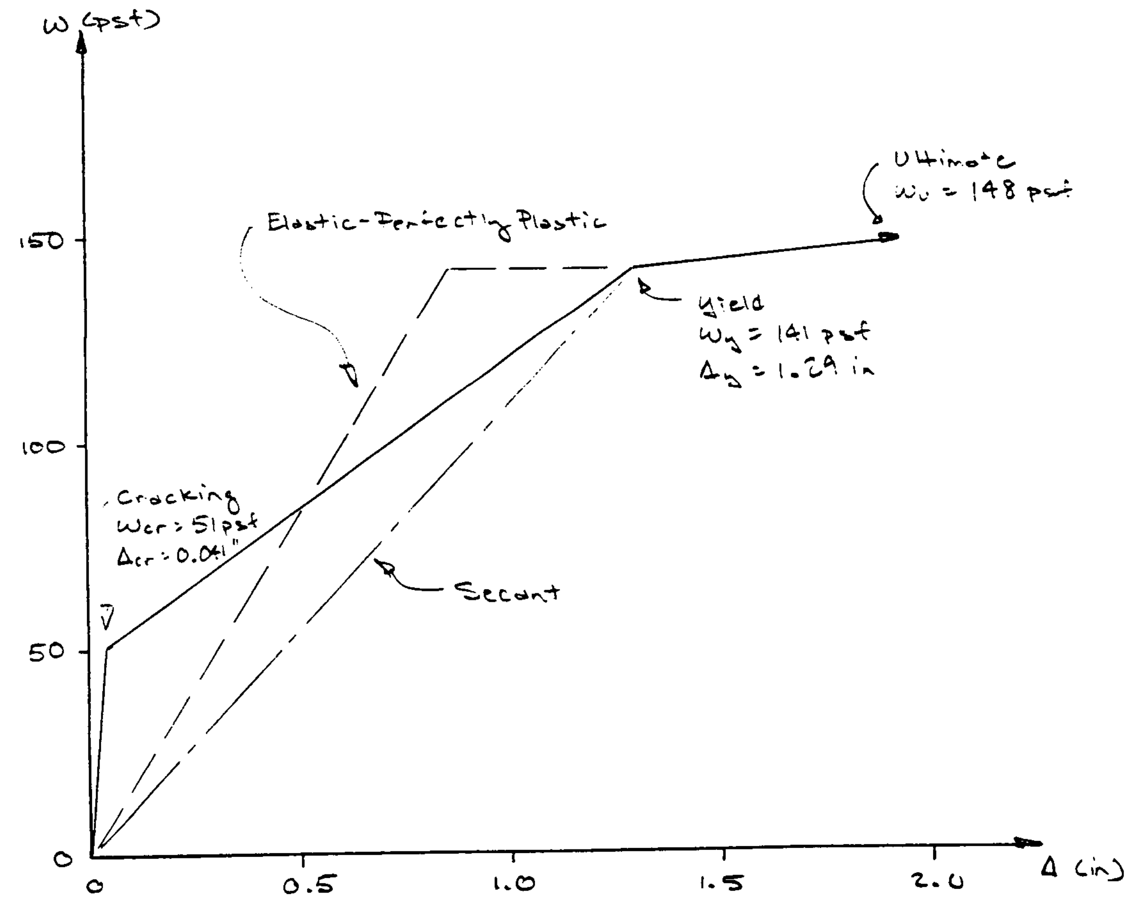
SHEET NO. BW-8

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-21-87

CLIENT LLNL SUBJECT Block Wall CHK'D [illegible] DATE [illegible]

Load - Deflection Curve



Equivalent Elastic Stiffness

Depending on the applied load, the wall stiffness may vary from the uncracked ($I = 1340\ in^4/ft$) to the cracked ($I = 78\ in^4/ft$) values. Because the wall response and seismic capacity will be sensitive to frequency and thus stiffness, two candidate values will be investigated:

1. Elastic stiffness of the elastic-perfectly plastic load-deflection curve that preserves the total energy up to yield.

2. Secant stiffness to yield. (a lower bound value).

Equivalent Elastic-Perfectly Plastic Curve

$$E = \tfrac{1}{2}(51)(0.041) + \tfrac{1}{2}(141 + 51)(1.29 - 0.041)$$
$$= 121$$

$$\tfrac{1}{2}(141)(\Delta_e) + 141(1.29 - \Delta_e) = 121$$

$$\Delta_e = 0.86''$$

$$I_e = \frac{141(10)^4(1728)}{8(2.0\times10^6)(0.86)}$$
$$= 177\ in^4/ft$$

Secant Stiffness

$$I_e = \frac{141(10)^4(1728)}{8(2.0\times10^6)(1.29)}$$
$$= 118\ in^4/ft$$

Fragility Based On Elastic - Perfectly Plastic Curve

Fundamental Frequency

$I_e = 177$ in⁴/ft

$\bar{m} = 0.0239$ lb-sec²/in²/ft p. 3, sample calc, Ref. 4

$L = 120''$

$E = 2.0 \times 10^6$ psi

$$\omega = 1.875^2 \sqrt{\frac{2.0\times10^6 (177)}{0.0239 (120)^4}}$$

$= 29.7$ rps

$f = 4.7$ Hz

Seismic Loads

Because block wall inelastic energy absorption will be explicitly accounted for, estimate median damping of 7%.

$S_a = 0.65g$ $f = 4.7$ Hz, $\xi = 7\%$

$w = 0.65 (111)$
$= 72$ psf

$V = 72 (10)$
$= 720$ plf

$M = \frac{1}{2} (72)(10)^2$
$= 3600$ ft-lb/ft

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SHEET NO. BW-11

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-21-87

CLIENT LLNL SUBJECT Block Wall CHK'D DATE

Strength Factor

$w = 72$ psf

$w_u = 148$ psf

$\check{F}_S = \frac{148}{72}$

$= 2.05$

Uncertainty

Material Properties

Strength is primarily a function of the reinforcement

$\beta = \beta_{fy}$

$= 0.07$

Accuracy of Strength Equation, Workmanship

A ϕ factor of 0.8 is recommended for ultimate strength design of masonry in Refs. 1 and 3. Estimating that this corresponds to 95% conf. of exceedence:

$\beta = -\frac{1}{1.65} \ln \frac{0.8}{1.0}$

$= 0.14$

The following variabilities can be determined from Fig. 4 of Ref. 3:

For $\alpha = 0.59$

$\left(\frac{M_u}{M_n}\right)_{50\%} = 1.1$

$\left(\frac{M_u}{M_n}\right)_{5\%} = 0.93$

$\beta = -\frac{1}{1.65} \ln \frac{0.93}{1.1}$

$= 0.10$

For $\alpha = 0.33$

$$\left(\frac{M_u}{M_n}\right)_{50\%} = 1.0$$

$$\left(\frac{M_u}{M_n}\right)_{5\%} = 0.85$$

$$\beta = -\frac{1}{1.65} \ln \frac{0.85}{1.0}$$

$$= 0.10$$

Increasing these values to account for lab vs. field workmanship is appropriate

Estimate a β of 0.14.

$$\beta_U = \sqrt{0.07^2 + 0.14^2}$$
$$= 0.16$$

Increase to 0.18 to account for other sources (block size, rebar area, etc.)

$\beta_U = 0.18$

Inelastic Energy Absorption Factor

Loading on the walls tested in Ref. 1 was stopped when it was judged that failure was near. Comparison of maximum deflections to calculated yield deflections indicates ratios varying from 1.1 to 3.0 with an average of about 1.7 for the concrete block walls. Comparable values are exhibited by Figs. 3.7 and 3.8 of Ref. 2. Estimate a median ductility to actual failure of 2.0.

The calculated ultimate deflection is then:

$$\Delta_U = 2.0(1.3)$$
$$= 2.6''$$

The effective ductility ratio for the equivalent elastic-perfectly plastic load-deflection curve is then:

$$\mu_e = \frac{\Delta_u}{\Delta_e} = \frac{2.6}{0.86} = 3.0$$

The inelastic energy absorption factor will be determined using Riddell-Newmark (may be conservative since the estimated frequency is on the soft side of the peak). Median damping of 7% is judged to be appropriate when ductile response is explicitly accounted for. Estimate a duration coefficient, C_D, of unity is appropriate (EQ's centered on M6.3).

$$q = 3.0(7)^{-0.30} = 1.67$$

$$p = 1.67 + 1 = 2.67$$

$$r = 0.48(7)^{-0.08} = 0.41$$

} Acceleration region

$$\check{F}_\mu = [2.67(3.0) - 1.67]^{0.41} = 2.1$$

Variability

Estimate $F_\mu = 1.0$ is 3σ below mean, randomness and uncertainty are about equal.

$\beta_c = -\frac{1}{3} \ln \frac{1.0}{2.1}$

$= 0.25$

$\beta_R = \beta_U = \frac{1}{\sqrt{2}} (0.25)$

$= 0.18$

Spectral Shape Factor (EQ Definition) – Structure Response

The earthquake is defined as the 84% non-exceedance probability spectrum within the frequency range of interest for the larger horiz. component of motion.

Peak to Peak Variation

Estimate peak to peak variability, $\beta_{PP} \simeq 0.20$

$\check{F}_{PP} = e^{0.20}$

$= 1.22$

EQ Directional Content

The calculated response is based on the larger of the two horiz. components. Wall failure results from only one of the components. The following values are estimated:

$\check{F}_{\psi} = 1.1$

$\beta_{\psi} = 0.15$

Total

$\check{F}_{SA} = 1.22(1.1)$
$= 1.35$

$\beta_R = \sqrt{0.20^2 + 0.15^2}$
$= 0.25$

<u>Damping - Structure Response</u>

Floor spectra were generated for 10% structure damping, which is judged to be ^(near) median centered at wall failure. The structure fundamental mode is at about 7.2 Hz (amplified range)

$\check{F}_\delta = 1.0$

Variability

Estimate 7% damping is an 84% exceedance value.

Median amplification factor, 10% damping = 1.64 } NUREG/CR-0098
" " 7% damping = 1.89

$\beta_U = \ln \frac{1.89}{1.64}$
$= 0.14$

Estimate $\beta_R = 0.2\beta_U$
$= 0.03$

<u>Modeling Factor - Structure Response</u>

The building model is assumed to be median-centered.

$\check{F}_M = 1.0$

Estimate $\beta_{ms} \approx 0.15$
$\beta_f \approx 0.30$ } Typical values

Fundamental freq. ≈ 7.2 Hz

$f_{+1\beta} = 7.2e^{0.30}$
$= 9.7$ Hz

$$\log AF = \frac{\log 1.64 - \log 1}{\log 8 - \log 33} (\log 9.7 - \log 33) + \log 1$$

$AF = 1.53$

$\beta_{mf} = -\ln \frac{1.53}{1.64}$
$= 0.07$

$f_{-1\beta} = 7.2e^{-0.3}$
$= 5.3$ Hz

$\beta_{mf} = 0$

Estimate $\beta_{mf} = 0.04$

$\beta_U = \sqrt{0.15^2 + 0.04^2}$
$= 0.16$

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SHEET NO. BW-17

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-22-87

CLIENT LLNL SUBJECT Block Wall CHK'D MKR DATE 10/5/87

Soil-Structure Interaction

$F_{SSI} = 1.0$

$\beta_R = 0.01$

$\beta_U = 0.05$

} Nominal values for rock founded structures

Structure Inelastic Response

Structure inelastic response results in a reduction in structure frequency, which would imply an increase in block wall response. However, inelastic response of the block wall causes a reduction in wall frequency and resulting shift away from the floor spectrum peak. The block wall response as calculated is judged to be median-centered. Per Lin & Mahin (Ref. 5) the mean amplification factor for high period eqmt. response due to nonlinear structure response is about 1.1 for stiffness degrading structures. Estimate this is an upper bound for a nonlinear component.

$$\beta_R \approx \beta_U = \frac{1}{1.65} \ln \frac{1.1}{1}$$

$$= 0.06$$

Peak Broadening & Smoothing

The floor spectra are assumed to be raw spectra.

$F_{SS_1} = 1.0$
$\beta_R = \beta_U = 0$

Artificial Time-History Generation

Estimated typical values.

$\check{F}_{SS_2} = 1.1$
$\beta_U = 0.05$

Damping - Block Wall

Estimate 5% damping is a -1σ value for block walls when inelastic response is explicitly accounted for.

$S_a \simeq 0.75g$

$$\beta_U = \ln \frac{0.75}{0.65}$$
$$= 0.14$$

$$\beta_R \simeq 0.2(\beta_U)$$
$$\simeq 0.03$$

Modeling – Block Wall

Estimate $\beta_f = 0.10$

$f = f_{+1\beta} = 4.7 e^{0.10} = 5.2$ Hz

$S_a = 0.8g$

$\beta_{mf} = \ln \frac{0.8}{0.65}$

$= 0.21$

$f = f_{-1\beta} = 4.7 e^{-0.10} = 4.3$ Hz

$S_a = 0.5g$

$\beta_{mf} = -\ln \frac{0.5}{0.65}$

$= 0.26$

Estimate $\beta_U \approx 0.25$ (increased to account for use of effective elastic stiffness, derivation of fragility from equivalent uniform load, etc.).

Modal Combination

Response is predominantly influenced by the fundamental mode. Estimate $\beta_R \approx 0.05$.

Combination of EQ Components

Failure of the wall occurs primarily due to response in a single horizontal direction, while vertical response has minor effect on the resistance. Estimate $\beta_R \approx 0.05$

Overall Fragility Parameters

$$\check{F} = 2.05(2.1)(1.35)(1.1) = 6.4$$

$$\check{A} = 6.4(0.18g) = 1.15g$$

$$\beta_R = [0.18^2 + 0.25^2 + 0.03^2 + 0.01^2 + 0.06^2 + 0.03^2 + 0.05^2 + 0.05^2]^{1/2} = 0.32$$

$$\beta_U = [0.18^2 + 0.18^2 + 0.14^2 + 0.16^2 + 0.05^2 + 0.06^2 + 0.05^2 + 0.14^2 + 0.25^2]^{1/2} = 0.45$$

EQE

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SHEET NO. BW-21

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-25-87

CLIENT LLNL SUBJECT Block Wall CHK'D MKR DATE 10/5/87

HCLPF Capacity

$$\text{HCLPF Capacity} = 1.15g\, e^{-1.65(0.32+0.45)}$$

$$= 0.32g$$

Fragility Based On Secant Stiffness

Fundamental Frequency

$I_e = 118 \text{ in}^4/\text{ft}$

$f = 4.7\sqrt{\frac{118}{177}}$

$= 3.8 \text{ Hz}$

Seismic Loads

$S_a = 0.40g$ $\qquad f = 3.8 \text{ Hz}, \ \xi = 7\%$

$w = 0.40(111)$

$= 44 \text{ psf}$

$V = 44(10)$

$= 440 \text{ plf}$

$M = \frac{1}{2}(44)(10)^2$

$= 2200 \text{ ft-lb/ft}$

Strength Factor

$w = 44 \text{ psf}$

$w_u = 148 \text{ psf}$

$\breve{F}_s = \frac{148}{44}$

$= 3.4$

$\beta_u = 0.18$

Inelastic Energy Absorption Factor

Base effective ductility ratio and F_μ on ratio of ultimate to yield deflections. This is somewhat conservative since energy in the actual load-deflection curve above the second curve is neglected.

$\Delta_y = 1.3''$
$\Delta_u = 2.6''$

$\mu = 2$

$\check{F}_\mu = [2.67(2) - 1.67]^{0.41}$
$= 1.7$

$\beta_c = -\frac{1}{3} \ln \frac{1}{1.7}$
$= 0.17$

$\beta_R = \beta_U = \frac{1}{\sqrt{2}}(0.17)$
$= 0.13$

Modeling - Block Wall

Estimate $\beta_f \simeq 0.10$

$f = f_{+1\beta} = 3.8e^{0.10} = 4.2\ Hz$
$S_a = 0.48g$

$\beta_{mf} = \ln \frac{0.48}{0.40}$
$= 0.18$

$$f = f_{-1\beta} = 3.8 e^{-0.10}$$
$$= 3.4 \text{ Hz}$$
$$S_a = 0.38 g$$
$$\beta_{ms} = -\ln \frac{0.38}{0.40}$$
$$= 0.05$$

Estimate $\beta_U = 0.20$ (Weighted towards the stiffer side since this frequency should be more of a lower bound, includes boost for use of uniform pressure, etc.)

Overall Fragility Parameters

Other factors and variabilities should be about the same as previously calculated for the elastic-perfectly plastic case. Values may be somewhat different since wall response occurs in the frequency range where the floor spectra are not strongly influenced by structure response. However, the HCLPF capacity should not be very sensitive to this approximation.

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SHEET NO. BW-25

JOB NO. 87218.01 JOB HCLPF Comparison BY FSH DATE 9-25-87

CLIENT LLNL SUBJECT Block Wall CHK'D MKR DATE 10/5/87

$$\check{F} = 3.4\,(1.7)(1.35)(1.1) = 8.6$$

$$\check{A} = 8.6\,(0.18g) = 1.55g \quad \text{vs. } 1.15g$$

$$\beta_R = \sqrt{0.32^2 - 0.17^2 + 0.13^2} = 0.30 \quad \text{vs. } 0.32$$

$$\beta_U = \sqrt{0.45^2 - 0.17^2 + 0.13^2 - 0.25^2 + 0.20^2} = 0.41 \quad \text{vs. } 0.45$$

$$\text{HCLPF Capacity} = 1.55g\, e^{-1.65(0.30+0.41)} = 0.48g \quad \text{vs. } 0.32g$$

μ	F_μ	$\check{A}$	β_R	β_U	HCLPF
3	2.1	1.9	0.32	0.45	0.5
5	2.7	2.45	0.36	0.45	0.64

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SHEET NO. BW-26

JOB NO. 87218.01 JOB HCLPF Comparison BY PSH DATE 9-25-87

CLIENT LLNL SUBJECT Block Wall CHK'D MKR DATE 10/5/87

CDFM HCLPF Capacity

Material Properties

Use the following properties:

$f_y = 60$ ksi

$f'_m = 1700$ psi

$m_o = 1800$ psi

Secant Stiffness

Cracking Load and Deflection

$\sigma_{cr} = 2.5\sqrt{1700}$ Table 7-20, Ref. 1

$= 103$ psi

$M_{cr} = 230(103)$

$= 23,700$ in-lb/ft

$= 1970$ ft-lb/ft

$w_{cr} = \dfrac{2(1970)}{10^2}$

$= 39$ psf

$E_m = 1000(1700)$

$= 1.7 \times 10^6$ psi

$\Delta_{cr} = \dfrac{39(10)^4(1728)}{8(1.7\times10^6)(1340)}$

0.037 in

Yield Load and Deflection

$$n = \frac{29\times10^6}{1.7\times10^6} = 17$$

$$\tfrac{1}{2}(16)(kd)^2 - \tfrac{1}{2}(5.9)[(kd)-1.5]^2 - 17(0.31)[5.8-(kd)] = 0$$

$$5.05(kd)^2 + 14.12(kd) - 37.20 = 0$$

$$(kd) = 1.66"$$

$$I_{cr} = \tfrac{1}{3}(16)(1.66)^3 - \tfrac{1}{3}(5.9)(1.66-1.5)^3 + 17(0.31)(5.8-1.66)^2$$

$$= 115 \text{ in}^4/\text{block}$$

$$= 86 \text{ in}^4/\text{ft}$$

$$M_y = 0.8\,\frac{60{,}000(86)}{17(5.8-1.66)}$$

$$= 58{,}700 \text{ in-lb/ft}$$

$$= 4890 \text{ ft-lb/ft}$$

$$w_y = \frac{2(4890)}{10^2} = 98 \text{ psf}$$

$$\Delta_y = 0.037 + \frac{(98-29)(10)^4(1728)}{8(1.7\times10^6)(86)}$$

$$= 0.037 + 0.872$$

$$= 0.91 \text{ in}$$

Moment of Inertia

$$I_e = \frac{98(10)^4(1728)}{8(1.7\times10^6)(0.91)}$$

$= 137\ in^4/ft$

Effective Axial Load

Estimate HCLPF ≈ 0.5g

Vert. ZPA = $0.20g\left(\frac{0.5}{0.18}\right)$

$= 0.56g$

$P = 1110[1-0.4(0.56)]$

$= 863\ lb/ft$

Ultimate Strength

$\phi = 0.8$

$$a = \frac{0.233(60,000)+863}{0.85(1700)(12)}$$

$= 0.86\ in$

Table 7-4, Ref. 1

$$M_u = 0.8\left[0.233(60,000)+863\right]\left(5.8-\frac{0.86}{2}\right)$$

$= 63,800$ in-lb/ft

$= 5310$ ft-lb/ft

$$w_u = \frac{2(5310)}{10^2}$$

$= 106$ psf

Seismic Response

$$\omega = 1.875^2\sqrt{\frac{1.7\times10^6(137)}{0.0239(120)^4}}$$

$= 24.$ rps

$f = 3.8$ Hz

Use 7% damping

$S_a = 0.40g$ 7% damping.

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SHEET NO. BW-29

JOB NO. 87218.01 JOB HCLPF Comparison BY TSH DATE 9-25-87

CLIENT LLNL SUBJECT Block Wall CHK'D MKR DATE 10/5/87

Since only slight inelastic energy absorption effect will be considered, estimate 10% damping is median.

$S_a \approx 0.34g$ 10% damping

$w = 0.34(111)$
$= 38$ psf

At the wall HCLPF capacity, estimate structure damping is about 7% median

$F = \frac{1.64}{1.89}$
$= 0.87$

Include 1.1 factor for mild amount of ductile behavior.

HCLPF Capacity $= \frac{106}{38}(0.87)(1.1)(0.18g)$
$= 0.48g$

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SHEET NO. BW-30

JOB NO. 77218.01 JOB HCLPF Comparison BY PSH DATE 9-21-87

CLIENT LLNL SUBJECT Block Wall CHK'D MKR DATE 10/5/87

References

1. ACI - SEASC Task Committee On Slender Walls, "Test Report On Slender Walls", 1982.

2. Omote, Y., et al, "A Literature Survey - Transverse Strength of Masonry Walls", Earthquake Engineering Research Center, UCB/EERC-77/07, March, 1977.

3. Suter, G.T. and G.A. Fenton, "Flexural Capacity of Reinforced Masonry Members", ACI Journal, January-February, 1986.

4. Letter transmittal, M.K. Ravindra (EQE) to R.C. Murray (LLNL), July 16, 1987.

5. Lin, J. and S.A. Mahin, "Seismic Response of Light Subsystems On Inelastic Structures," Journal of the Structural Division, ASCE, February, 1985.

Round 2 Fragility Analysis Calculations

The following modifications will be made:

1. Use 10% median damping.

2. Further inspection of testing in Ref. 1 indicates that ratio of maximum to calculated yield deflections range from 1.3 to 5.0 with an average of about 2.3. Because loading was stopped prior to actual failure, estimate a median ductility against calculated yield of about 3.

Only secant stiffness calcs will be modified.

For f = 3.8 Hz, 10% damping, $S_a \approx 0.34g$. ✓

$\omega = 0.34(111)$

$= 38$ psf ✓

Strength Factor

$F_S^* = \frac{148}{38}$

$= 3.9$ ✓

Inelastic Energy Absorption Factor

$\Delta_u = 3(1.3)$ ✓

$= 3.9"$ ✓

$\Delta_y = 1.3"$

$\mu = \frac{3.9}{1.3}$

$= 3$

$$\delta = 3(10)^{-0.30}$$
$$= 1.50 \checkmark$$

$$p = 1 + 1.50$$
$$= 2.50$$

$$r = 0.48(10)^{-0.08}$$
$$= 0.40$$

$$\check{F}_\mu = [2.50(3) - 1.5]^{0.40}$$
$$= 2.0$$

$$\beta_c = -\frac{1}{3} \ln \frac{1}{2.0}$$
$$= 0.23 \checkmark$$

$$\beta_R = \beta_U = \frac{1}{\sqrt{2}}(0.23)$$
$$= 0.16$$

Modeling - Block Wall

For $f = f_{+1\beta} = 4.2\,Hz$, 10% damping, estimate $S_a = 0.42g$

$$\beta_{mf} = \ln \frac{0.42}{0.34}$$
$$= 0.21$$

For $f = f_{-1\beta} = 3.4\,Hz$, 10% damping, estimate $S_a = 0.32g$

$$\beta_{mf} = -\ln \frac{0.32}{0.34}$$
$$= 0.06$$

Estimate $\beta_m \approx 0.20$

Damping - Block Wall

Estimate 7% damping is ~1σ

$S_a \approx 0.40g$

$\beta_U = \ln \frac{0.40}{0.34}$

$= 0.16$

$\beta_R = 0.2(0.16)$

$= 0.03$

Overall Fragility Parameters

Other values same as before

$\check{F} = 3.9(2.0)(1.35)(1.1)$

$= 11.6$ ✓

$\check{A} = 11.6(0.18g)$

$= 2.1g$ ✓

$\beta_R = \sqrt{0.16^2 + 0.25^2 + 0.03^2 + 0.01^2 + 0.06^2 + 0.03^2 + 0.05^2 + 0.05^2}$

$= 0.31$ ✓

$\beta_U = \sqrt{0.18^2 + 0.16^2 + 0.14^2 + 0.16^2 + 0.05^2 + 0.06^2 + 0.05^2 + 0.16^2 + 0.20^2}$

$= 0.42$ ✓

$HCLPF = 2.1g\, e^{-1.65(0.31+0.42)}$

$= 0.63g$ ✓

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SHEET NO BW-34

JOB NO 87128.01 JOB HCLPF Comparison BY PSH DATE 1-6-88

CLIENT LLNL SUBJECT Block Wall CHK'D MKR DATE 1-7-88

2nd Round CDFM Calculations

The following modifications will be made:

1. Use full $F_\mu = \frac{1}{0.8} = 1.25$

2. Structure is approaching yield, use 10% structure damping.

$$HCLPF = \frac{106}{38}(1.25)(0.18g)$$
$$= 0.63g$$

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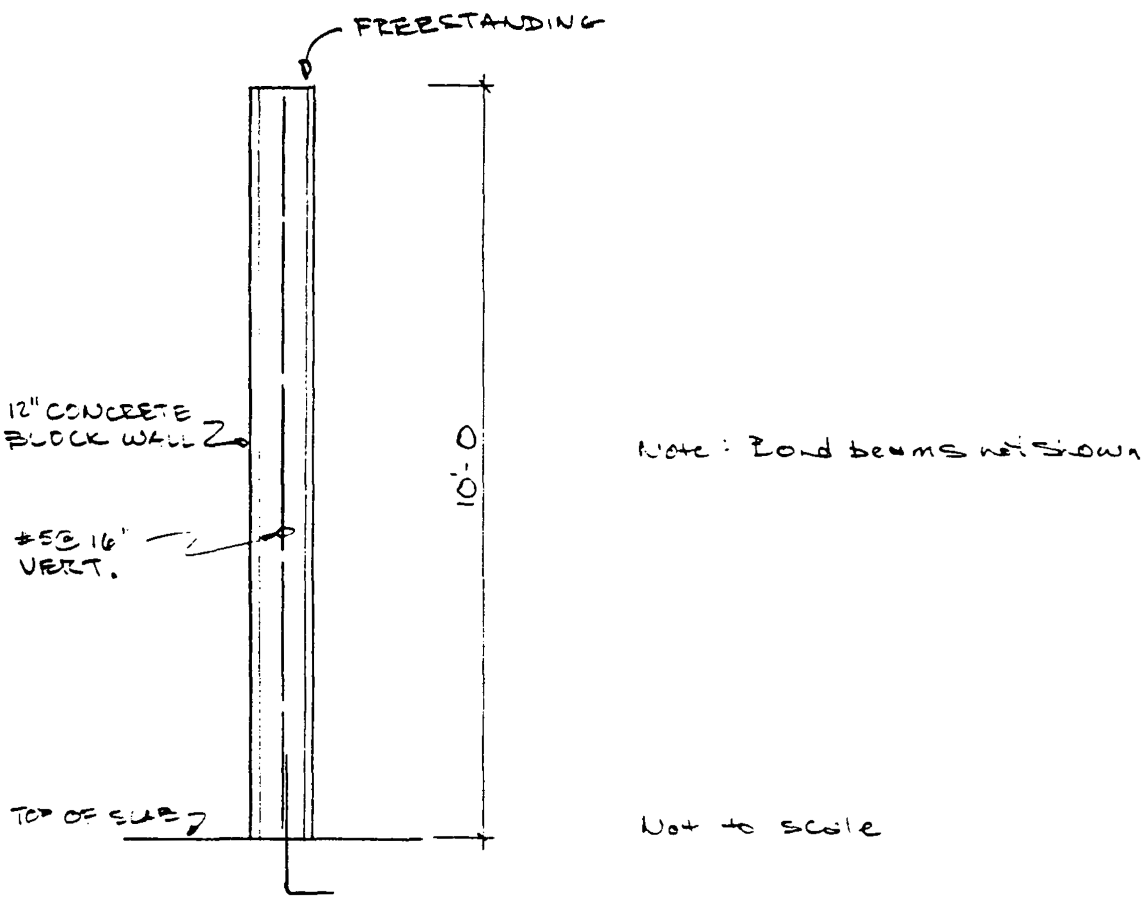
SHEET NO 1

JOB NO 87218.01 JOB HCLPF Comparison BY PSH DATE 7-1?-87

CLIENT LLNL SUBJECT Sample Block Wall CHK'D ____ DATE ____



Wall is constructed in running bond, cells w/ reinf. grouted solid

Block: 12" hollow units, normal weight, ASTM C90 Grade N

Mortar: ASTM C270 Type S

Vertical Reinforcement: #5@ 16" o.c., Grade 60 steel

Horizontal Reinforcement: Extra heavy, 3 rod, Durowal at 16" o.c.

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SHEET NO 3

JOB NO 9729.01 JOB HCLPF Comparison BY FSH DATE 2-16-87

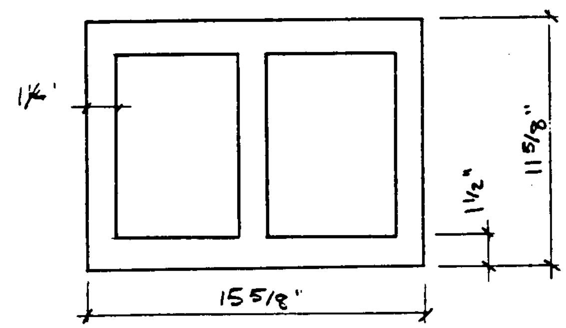
CLIENT LLNL SUBJECT Sample Block Wall CHK'D ____ DATE ____

Assumptions:

1. Wall is long compared to its height, isolated from the supporting structure at the top and sides. Can be modeled as a one way member, base supported, spanning vertically.

2. No attached equipment or other masses.

3. Standard two cell block used:



4. Wall weighs approximately 111 psf for cores grouted at 16".

5. Special inspection requirements are met.

6. In-plane stresses are relatively small and can be neglected.

7. Average unit compressive strength on net area of 3000 psi.

Verify that the wall meets criteria in Appendix A to S.R.P. Section 3.8.4. Check against Load Combination 2 (b) (4):

$$D + L + T_o + R_o + E'$$

Uncracked Stiffness & Frequency (Upper Bound)

$I_o \simeq \frac{1}{12}(16)(11.63)^3 - \frac{1}{12}(5.9)(8.63)^3$
$= 1781 \text{ in}^4/\text{block}$
$= 1340 \text{ in}^4/\text{ft}$

$\bar{m} = \dfrac{111}{12(386.4)}$
$= 0.0239 \text{ lb-sec}^2/\text{in}^2/\text{ft}$

Unit compressive strength, net area = 3000 psi

$f_m' = \dfrac{3000 - 2500}{4000 - 2500}(2000 - 1550) + 1550$ Table 4.3, ACI 531-79
$= 1700 \text{ psi}$

$E_m = 1000 f_m'$ Table 10.1, ACI 531-79
$= 1.7 \times 10^6 \text{ psi}$

$L = 120''$

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SHEET NO 4

JOB NO 87219.01 JOB HCLPF Comparison BY PSW DATE 7-16-87

CLIENT LLNL SUBJECT Sample Block Wall CHK'D ___ DATE -87

$$\omega = (1.875)^2 \sqrt{\frac{1.7\times10^6 (1243)}{0.0239\,(120)^4}}$$

$= 75$ rps

$f = 12$ Hz

Spectral acceleration ≅ 0.75 g — 7% damped, broadened, E-W Maine Yankee spectrum, EL 4" turb./serv. bldg

<u>Cracked Stiffness & Frequency</u> (Lower Bound)

$$A_s = \frac{0.31(12)}{16}$$

$= 0.233$ in²/ft

$$n = \frac{29\times10^6}{1.7\times10^6}$$

$= 17$

$$d = \frac{11.63}{2}$$

$= 5.8"$

$$\tfrac{1}{2}(12)(kd)^2 - 17(0.233)[5.8 - (kd)] = 0$$

$kd = 1.65" \quad > \quad t_f = 1.5"$

$$\tfrac{1}{2}(16)(kd)^2 - \tfrac{1}{2}(5.9)[(kd) - 1.5]^2 - 17(0.31)[5.8 - (kd)] = 0$$

$$5.05(kd)^2 + 14.12(kd) - 37.20 = 0$$

$(kd) = 1.66"$

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SHEET NO 5

JOB NO 87218.01 JOB HCLPF Capacity BY FSH DATE 7-16-87

CLIENT UNL SUBJECT Sample Block Wall CHK'D ____ DATE ____

$$I_{cr} = \frac{1}{3}(16)(1.66)^3 - \frac{1}{2}(6.9)(1.66-1.5)^3 + 17(0.31)(5.8-1.66)^2$$

$$= 115 \text{ in}^4/\text{block}$$

$$= 86 \text{ in}^4/\text{ft}$$

$$f = 12\sqrt{\frac{86}{1340}}$$

$$= 3.0 \text{ Hz}$$

Spectral acceleration ≈ 0.45g

Out of Plane Seismic Loads

Check for spectral acceleration of 0.75g

$$w = 0.75(111)$$
$$= 83 \text{ psf}$$

$$V_{max} = 83(10)$$
$$= 830 \text{ lb/ft}$$

$$M_{max} = \frac{1}{2}(83)(10)^2$$
$$= 4150 \text{ ft-lb/ft}$$
$$= 49{,}800 \text{ in-lb/ft}$$

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SHEET NO 6

JOB NO 8728.01 JOB HCLPF Capacity BY PSH DATE 7-16-87

CLIENT LLNL SUBJECT Sample Block Wall CHK'D ___ DATE ___

Check Reinforcement Tensile Stress

Allowable stress = 24,000 psi Sect. 10.2.1.1, ACI 531-79

Factored allowable = 2.0 (24,000) SRP Section 3.8.4, App. A
= 48,000 psi < 0.9 f_y

$f_s = \frac{49,800\,(5.8 - 1.66)}{86}\,(17)$

= 40,800 psi < 48,000 psi ok

Conservatively neglects axial load

Check Masonry Compressive Stress

$f_m' = 1700$ psi

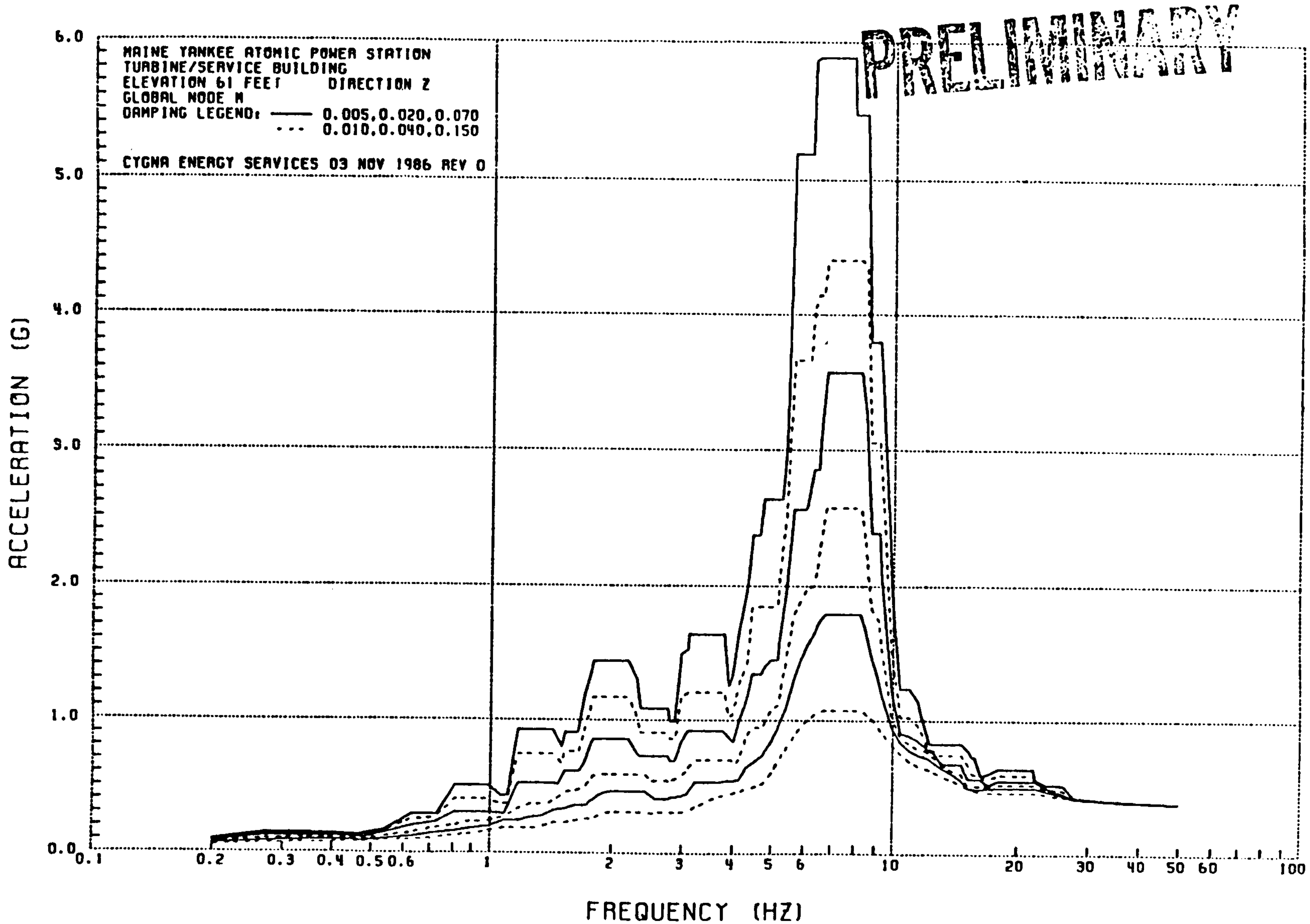
Allowable stress = $0.33 f_m'$ Table 10.1, ACI 531-79
= 0.33 (1700)
= 560 psi

Factored allowable = 2.5 (560) SRP Sect. 3.8.4, App. A
= 1400 psi

$f_m = \frac{49,800\,(1.66)}{86}$

= 960 psi < 1400 psi ok

Adequate margin if axial load included



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PAGE A126

Check Shear Stress

Allowable stress = $1.1\sqrt{f_m'}$ Table 10.1, ACI 531-79

$= 1.1\sqrt{1700}$

$= 45$ psi < 50 psi.

Factored allowable = 1.3 (45)

= 58 psi

Effective shear area = [5.9 + 2(1.25)] (5.8)

= 48.7 in²/block

= 37 in²/ft

$f_v = \frac{830}{37}$

= 22 psi < 58 psi ok

APPENDIX C

JOHN W. REED

BY	DATE 10/6/87	PROJECT	PAGE 1 OF
CHKD. BY	DATE 1/13/88	SUBJECT	JOB NO. 105-170

SUMMARY OF ANALYSES

	84% NEP MAX Horiz Direction (g)				
	ORIGINAL CALCS			Revised Calcs	
	HCLPF		MEDIAN	FA	
Component	FA	CDFM (BACKFIT)	FA	HCLPF	Median
Water Tank	0.27g	0.28g	0.53g	0.28g	0.55g
Auxiliary Cont. Cabinet:					
Ground level	0.48	0.48	1.20	Not Revised	
Floor Level	0.11	0.10	0.43	"	
Starting Air Tank	0.43	0.41	1.40	"	
Heat Exchanger	0.39	0.36	1.0	"	
Block Wall	0.38	0.33	1.41	0.52	1.96

Appendix A - Gives $\check{F}$, and β_R for effect of coupling, peak-to-peak variation, and horiz component to component variability

Appendix B Gives factor of 1.4 to convert from median prob average horizontal direction ground input to 84% NEP maximum direction ground input

Appendix C Describes procedure and provides the basis for backfitting a fragility analysis HCLPF to a single deterministic analysis (i.e. CDFM)

Appendix D Listings of ground & Floor response spectra used

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BY ______ DATE ______ PROJECT ______ PAGE 2 OF ______
CHKD. BY ______ DATE ______ SUBJECT ______ JOB NO. ______

General Assumptions\Comments (Special)

1. Vertical component = 2/3 horizontal, No variability included in this ratio

2. 7% damping used as median for building. This may be low - depends on stress state in building at median equipment level

3. Floor response spectra seem very high. This is why the HCPLFs seem low

4. Computer Programs are run on MATHCAD Ver 1.1 by Mathsoft

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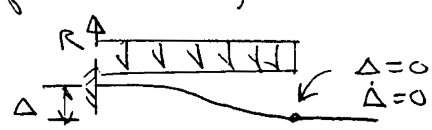
BY: [illegible] DATE: 10/3/87 PROJECT: PAGE: 1 OF:
CHKD. BY: [illegible] DATE: 12/1/87 SUBJECT: JOB NO.: 105-17?

FLAT-BOTTOM STORAGE TANK

Assumptions (special)

1. Capacity is controlled by anchor bolt yielding and buckling of tank shell

2. The water hold down force is obtained by the force on the raised bottom plate considered as a fixed beam, i.e



3 Seismic Tank Forces based on: Summary of Tank Calculations, by EQE.

	HCLPF		Median
Approach	Median Prob Ave Hor. Dir	84% NEP Max Horiz Dir	84% NEP Max Horiz Dir
Fragility Analysis	0.19 g	0.27 g	0.53 g
CDFM (backfit)	0.20	0.28	—
Revised Fragility*	0.20	0.28	0.55

* after 11/12-13/87 Meeting

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Strength

Program TANK.HC2 gives the calculation for the median analysis (see following pages)

$$\check{pga} = 0.346 g$$

Note: that this is based on a median level ground response spectrum (i.e. 50% NEP) and the maximum direction motion (i.e. input scaled by 1.11 - see Appendix B.)

pages 3 - 12 give the analysis

MCAD FILES TANK.HC2

CONSISTENT SET OF UNITS:

BASE UNITS: $in \equiv 1L$

$m \equiv 1M$

$sec \equiv 1T$

DERIVED UNITS: $ft \equiv 12 \cdot in$

$$lb \equiv m \cdot \frac{in}{sec^2}$$

$$k \equiv 1000 \cdot lb$$

$$g \equiv 32.2 \cdot \frac{ft}{sec^2}$$

$$ksi \equiv \frac{k}{in^2}$$

$$MPa \equiv 0.144 \cdot ksi$$

DEFINE MEDIAN PROPERTIES OF TANK:

$R := 240 \cdot in$	Radius of tank
$t_s := 0.375 \cdot in$	Thickness of tank wall
$\mu := 1.24$	Shell imperfection factor - for buckling cap.
$\delta := 0 \cdot in$	Shell imperfection size - assumed equal to shell thickness (normal construction)
$t := 0.25 \cdot in$	Thickness of tank bottom
$E_s := 28000. \cdot ksi$	Modulus of elasticity of tank steel
$A_b := 3.14 \cdot in^2$	Area of tank holddown anchor bolt for siffness calculations - use 2.50 for strength

$E_b := 29000 \cdot ksi$	Modulus of elasticity of bolt steel
$P_y := 135 \cdot k$	Yield strength of bolt: use effective yield stress equal to average of median yield and median ultimate (44 + 64)/2 = 54 ksi times the net bolt area = 2.50 sq in. This produces total capacity = 135 k.
$h_1 := 27 \cdot in$	Height of bolt from nut to bottom of tank
$h_2 := 54.5 \cdot in$	Height of bolt from nut to embedment plate
$f_y := 37 \cdot ksi$	Tank median yield stress
$P_{DL} := 63.7 \cdot k$	Dead weight of tank shell and roof
$P_{EQ} := 63.7 \cdot k$	Vertical earthquake force due to tank wall and roof calculated for a 1.0g vertical acceleration.
$\delta_w := 62.4 \cdot \frac{lb}{ft^3}$	Density of water
$h_w := 37 \cdot ft$	Height of water in tank
$n := 8$	Number of bolts
$i := 1 .. n$	Loop over all bolts to define angle from symmetric axis to each bolt:
$\theta_i := (i-1) \cdot \frac{2 \cdot \pi}{n}$	Angle from symmetric axis to each bolt, i
$VOH := \frac{2}{3}$	Ratio of vertical to horizontal earthquake components
$ampv := 1.89$	Vertical amplification for the fluid due to tank radial expansion (median at 7% damping)
$inp1 := 1.00$	Input scale factor for higher spectral amplification relative to median input at 7% damping.
$inp2 := 1.11$	Scale factor for peak earthquake horizontal input compared to average of two horizontal components
$inp3 := 1.1$	Ductility increase factor which influences the vertical earthquake reduction in hold down forces

$$p := \delta_w \cdot h_w \cdot \left[1 - .4 \cdot inp1 \cdot inp3 \cdot ampv \cdot VOH \cdot \frac{a}{g}\right]$$ Water pressure at bottom of tank

$$w := p$$ Effective pressure on tank bottom

$$I := \left[\frac{1}{12}\right] \cdot \left[t^3\right]$$ Moment of inertia of bottom plate per length

$$P_{net} := -\left[P_{DL} - P_{EQ} \cdot .4 \cdot inp3 \cdot VOH \cdot \frac{a}{g}\right]$$ Net vertical force due to dead load and earthquake force positive up

$$M_{net} := 233800 \cdot inp1 \cdot inp2 \cdot k \cdot in$$ Net overturning moment due to earthquake (233800 k*in for 0.30g for 7% damping average of two horizontal components)

DETERMINE SHELL BUCKLING CAPACITY:

Tank buckling capacity based on "Seismic Design of Storage Tanks" by M. J. N. Priestley, J. H. Wood and B. J. Davidson, Bulletin of the New Zealand National Society For Earthquake Engineering, Vol. 19, No. 4, December 1986.

$$f_{c1} := 0.6 \cdot E_s \cdot \left[\frac{t_s}{R}\right]$$ Classical "perfect shell" buckling stress

BUCKLING IN MEMBRANE COMPRESSION (DIAMOND BUCKLING):

$$\sigma := 1 - \mu \cdot \left[\frac{\delta}{t_s}\right] \cdot \left[\left[1 + \frac{2}{\mu \cdot \left[\frac{\delta}{t_s}\right]}\right]^{\frac{1}{2}} - 1\right]$$ Imperfection reduction factor

$$\Gamma := \frac{f_y}{\sigma \cdot f_{c1}}$$ Factor to determine THICK or THIN wall

$$f_o := f_y \cdot \left[1 - \frac{\Gamma}{4}\right] \cdot (\Gamma < 2) + \sigma \cdot f_{c1} \cdot (\Gamma > 2)$$

elastic THICK wall   elastic THIN wall

$$P := \frac{p \cdot R}{t_s \cdot f_{c1}}$$

Normalized circumferential wall stress

$$P := P \cdot (P < 5) + 5 \cdot (P > 5)$$

P is Limited to 5

$$C := \sqrt{1 - \left[1 - \frac{P}{5}\right]^2 \cdot \left[1 - \left[\frac{f_o}{f_{c1}}\right]^2\right]}$$

Factor for wall thickness and pressure - not to exceed 1.0

$$f_p := f_{c1} \cdot C \cdot (C < 1) + f_{c1} \cdot (C > 1)$$

Stress limited by classical buckling stress

$$f_{md} := f_{c1} \cdot \left[0.19 + 0.81 \cdot \frac{f_p}{f_{c1}}\right]$$

$$f_{md} := f_{md} \left[f_{md} < f_y\right] + f_y \cdot \left[f_{md} > f_y\right]$$

Diamond buckling capacity limited by yield stress

ELASTIC PLASTIC COLLAPSE (ELEPHANT FOOT BUCKLING):

$$s := \frac{\frac{R}{t_s}}{400}$$

$$K := \left[1 - \left[\frac{p \cdot R}{t_s \cdot f_y}\right]^2\right] \cdot \left[1 - \frac{1}{1.12 + s^{1.5}}\right] \cdot \left[\frac{s + \frac{f_y}{250 \cdot MPa}}{s + 1}\right]$$

$f_{me} := f_{c1} \cdot K \cdot (K < 1) + f_{c1} \cdot (K > 1)$ Limit elephant foot buckling stress to classical buckling stress

MINIMUM BUCKLING STRESS:

$$f_m := f_{md} \cdot \left[f_{md} < f_{me} \right] + f_{me} \cdot \left[f_{me} < f_{md} \right]$$

This is the buckling stress used:

$\frac{R}{t_s} = 640$ $\frac{p \cdot R}{t_s} = 57.59 \cdot MPa$

$f_m = 17.183 \cdot ksi$ $f_{md} = 171.061 \cdot MPa$ $f_{me} = 119.325 \cdot MPa$

$f_{s\alpha} := f_m$ Allowable tank buckling stress

DEFINE FORCES AND MOMENTS AT TANK BOTTOM AT THE CENTER:

$j := 1 \,..3$

$\theta_{NA_j} := \theta_{st} + .01 \cdot (j - 1)$

Compressive force due to contact between tank shell and base, positive downward:

$$P_{s_j} := 2 \cdot f_{s\alpha} \cdot R \cdot \frac{t_s}{1 - \cos\left[\theta_{NA_j}\right]} \cdot \left[\theta_{NA_j} \cdot \cos\left[\theta_{NA_j}\right] - \sin\left[\theta_{NA_j}\right]\right]$$

$\frac{P_{s_j}}{k}$ where θ.NA is angle to neutral axis

-965.512
-985.896
$-1.006 \cdot 10^{3}$

Moment due to contact between tank shell and base, when resists earthquake overturning:

$$M_{s_j} := \frac{f_{s\alpha} \cdot R^2 \cdot t_s}{1 - \cos\left(\theta_{NA_j}\right)} \cdot \left[\theta_{NA_j} - \frac{1}{2} \cdot \sin\left(2 \cdot \theta_{NA_j}\right)\right]$$

$\frac{M_{s_j}}{1000 \cdot k \cdot in}$
226.666
231.232
235.782

Force due to a single bolt:

$$const_j := \frac{A_b \cdot f_{s\alpha}}{1 - \cos\left(\theta_{NA_j}\right)} \cdot \left(\frac{h_1}{h_2}\right) \cdot \left(\frac{E_b}{E_s}\right)$$

$$P2_j := const_j \cdot \left[\cos\left(\theta_{NA_j}\right) - \cos\left(\theta_2\right)\right]$$

$$P3_j := const_j \cdot \left[\cos\left(\theta_{NA_j}\right) - \cos\left(\theta_3\right)\right]$$

$$P4_j := const_j \cdot \left[\cos\left(\theta_{NA_j}\right) - \cos\left(\theta_4\right)\right]$$

$$P5_j := const_j \cdot \left[\cos\left(\theta_{NA_j}\right) - \cos\left(\theta_5\right)\right]$$

Modify bolt force to be greater than 0 but less than Py:

$$P2_j := P2_j \cdot \left(P2_j > 0 \cdot k\right) \cdot \left(P2_j < P_y\right) + P_y \cdot \left(P2_j > P_y\right)$$

$$P3_j := P3_j \cdot \left(P3_j > 0 \cdot k\right) \cdot \left(P3_j < P_y\right) + P_y \cdot \left(P3_j > P_y\right)$$

$$P4_j := P4_j \cdot \left[P4_j > 0 \cdot k\right] \cdot \left[P4_j < P_y\right] + P_y \cdot \left[P4_j > P_y\right]$$

$$P5_j := P5_j \cdot \left[P5_j > 0 \cdot k\right] \cdot \left[P5_j < P_y\right] + P_y \cdot \left[P5_j > P_y\right]$$

Force due to sum of all bolts, positive downward:

$$P_{b_j} := 2 \cdot \left[P2_j + P3_j + P4_j\right] + P5_j$$

$$\frac{P_{b_j}}{k}$$

769.191
763.138
757.452

Moment due to all bolts, positive when resists earthquake overturning:

$$M_{b_j} := -R \cdot \left[2 \cdot P2_j \cdot \cos\left[\theta_2\right] + 2 \cdot P3_j \cdot \cos\left[\theta_3\right] + 2 \cdot P4_j \cdot \cos\left[\theta_4\right] + P5_j \cdot \cos\left[\theta_5\right]\right]$$

$$\frac{M_{b_j}}{1000 \cdot k \cdot in}$$

62.236
63.263
64.228

Total hold down force from water from bottom plate, positive downward:
(This equation assumes that the there is no rotation at the tank shell/bottom plate interface as the plate picks up due to rocking.)

$$P_{L_j} := \frac{4 \cdot R}{3} \cdot \left[72 \cdot I \cdot w^3 \cdot f_{s\alpha} \cdot h_1\right]^{\frac{1}{4}} \cdot \int_{\theta_{NA_j}}^{\pi} \left[\frac{\cos\left[\theta_{NA_j}\right] - \cos(\theta)}{1 - \cos\left[\theta_{NA_j}\right]}\right]^{\frac{1}{4}} d\theta$$

Total resisting moment from water on bottom plate, positive when resists earthquake overturning:

$$M_{L_j} := \frac{-\left[4 \cdot R^2\right]}{3} \cdot \left[72 \cdot I \cdot w^3 \cdot f_{s\alpha} \cdot h_1\right]^{\frac{1}{4}} \cdot \int_{\theta_{NA_j}}^{\pi} \left[\frac{\cos\left[\theta_{NA_j}\right] - \cos(\theta)}{1 - \cos\left[\theta_{NA_j}\right]}\right]^{\frac{1}{4}} \cdot \cos(\theta)\, d\theta$$

$$w = 0.013 \cdot \frac{k}{in^2}$$

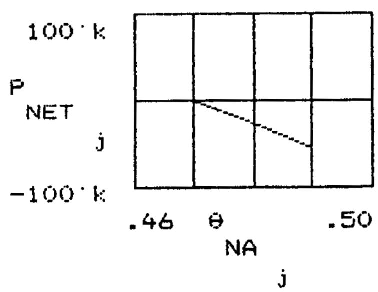
$\frac{P_{L_j}}{k}$
140.525
138.595
136.717

$\frac{M_{L_j}}{1000 \cdot k \cdot in}$
10.27
10.23
10.192

$$P_{NET_i} := P_{s_i} + P_{b_i} + P_{L_i} - P_{net}$$ Vertical equilibrium equation

θ_{NA_j}
0.47
0.48
0.49

$\frac{P_{NET_j}}{k}$
1.439
-26.927
-54.865



$$M_{u_j} := M_{b_j} + M_{L_j} + M_{s_j}$$

θ_{NA_j}	$\frac{M_{u_j}}{1000 \cdot k \cdot in}$
0.47	299.172
0.48	304.725
0.49	310.202

$$a_{CAP_j} := \frac{M_{u_j}}{M_{net}} \cdot (.30 \cdot g)$$

θ_{NA_j}	$\frac{a_{CAP_j}}{g}$
0.47	0.346
0.48	0.352
0.49	0.359

$i := 1$

$$a_{FIN} := a_{CAP_i} - P_{NET_i} \frac{a_{CAP_{i+1}} - a_{CAP_i}}{P_{NET_{i+1}} - P_{NET_i}}$$

$a_{FIN} = 0.346 \cdot g$

$$\theta_{FIN} := \theta_{NA_i} - P_{NET_i} \frac{\theta_{NA_{i+1}} - \theta_{NA_i}}{P_{NET_{i+1}} - P_{NET_i}}$$

$\theta_{FIN} = 0.471$

Some parameters:

These are the starting values: $\theta_{st} \equiv .47$ $a \equiv .346 \cdot g$

$f_{s\alpha} = 17.183 \cdot ksi$

$P_y = 135 \cdot k$

inp1 = 1

inp2 = 1.11

inp3 = 1.1

VOH = 0.667

ampv = 1.89

Develop Fragility Parameters, $\check{F}$, β_r, β_u & NCLPF

Capacity

Strength

$\check{F} = 0.346g / 0.346g = 1.0$ (see page 12)

$\beta_r = 0$

uncertainty

1. Bolt capacity ($-1\sigma = (44 \times 2.5) = 110^k$)

   TANK.HC2 gives pga = 0.302g

   $\beta_u = \ln \frac{.346}{.302} = \underline{0.14}$

2. Buckling capacity (imperfection (δ) = wall thickness (t_s) is -1σ)

   TANK.HC2 gives pga = .334g

   $\beta_u = \ln \frac{.346}{.334} = \underline{0.04}$

3. Holddown water force (-1σ is ½ median force)

   TANK.HC2 gives pga = .324g

   $\beta_u = \ln \frac{.346}{.324} = \underline{0.07}$

total

$\beta_u = (0.14^2 + 0.04^2 + .07)^{1/2} = \underline{0.16}$

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Inelastic Response

Bolts only yield 1 way (in tension). Assume 3 maximum cycles of motion to estimate

$$F_\mu = 1.1$$

$$\beta_c = \frac{1}{2.33} \ln\left(\frac{1.1}{1}\right) = 0.04$$

$$\beta_r = \beta_u = 0.03$$

Structure Response

Ground Motion This parameter includes:

1) Peak-to-peak spectrum variability
2) Horizontal Direction Variability
3) Equipment Horizontal Component Capa...

Tank capacity is controlled by maximum earthquake horizontal component. Factor of 1.11 to account for bias between average component (of 2 horiz dir) and median of maximum direction is included in analysis (see inp2 = 1.11 on pg 4)

Randomness for variability of maximum component (about its median is 0.12 (see Appendix A)

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Since 1 direction controls there is no horizontal direction coupling

For peak-to-peak response spectrum variability analyzed tank for $+1\sigma$ input, ie

$$\text{inp} 1 = e^{+.20} = 1.22$$

TANK.HC2 gives pga = .286,

$$\beta_r = \frac{1}{1} \ln \frac{.346}{.286} = 0.19$$

Combined

$$\check{F} = 1.0$$

$$\beta_u = 0$$

$$\beta_r = 0.22 \quad \left(\overline{0.12}^2 + \overline{0.19}^2\right)^{1/2}$$

Damping

Median damping is 7% (Analysis performed for 7%)
-1σ damping is 5%
from ground response spectra (freq on plateau of spectra)

$$Sa(7\%) = 1.89$$

$$Sa(5\%) = 2.12$$

$$\beta_u = \ln \frac{2.12}{1.89} = 0.11$$

$$\beta_r = 0$$

$$\check{F} = 1.0$$

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Modeling

frequency - since tank frequency ($\approx 6hz$) is in region of ground response spectrum plateau no variability

$\check{F} = 1.0$

$\beta_r = 0$

$\beta_u = 0$

Mode Shape - Assume controlled by first mode & analysis is rigorous

$\check{F} = 1.0$

$\beta_r = 0$

$\beta_u = 0$ (estimate)

Mode Combination

Assume no variability for mode effect

$\check{F} = 1.0$

$\beta_r = 0$

$\beta_u = 0$

Horizontal Component Phasing

Response controlled by 1 direction (i.e. max dir)

$\check{F} = 1.0$

$\beta_r = 0$

$\beta_u = 0$

SSI (Rock site)

$\check{F} = 1.0$ $\beta_r = 0$ $\beta_u = 0.05$ (estimate)

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Parameter	$\check{F}$	β_r	β_u
Capacity			
Strength	1.0	0	0.16
Inelastic Response	1.1	0.03	0.03
Equipment Response			
Spectral Shape	N.A.		
Damping			
Modeling – Frequency			
– Mode Shape			
Mode Combination			
Horiz. Component Phasing			
Structure Response			
Ground Motion	1.0	0.22	0
Damping	1.0	0	0.11
Modeling – Frequency	1.0	0	0
Mode Shape	1.0	0	0
Mode Combination	1.0	0	0
Horiz. Component Phasing	1.0	0	0
SSI	1.0	0	0.05
Inelastic Response	N/A	N/A	N/A
Combined:	1.1	0.22	0.20

$\check{a}_{pga} = (1.1)(0.346)g = 0.38g$ $\quad \check{a}_{pga(Peak\ 84\%)} = 1.4(.38) = 0.53g$

$HCLPF_m\ (0.38)\ e^{-1.65(.20+.20)} = 0.19g$

$HCLPF_{Peak\ 84\%}\ 1.4 \cdot (0.20) = 0.27g$ (See Appendix B)

B

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Compute HCLPF Directly (See Appendix C)

$\Sigma x \cdot \beta = 1.65(.22 + .20) = 0.0.69$

Parameter	Conservatism	$x \cdot \beta$	Deterministic Input
Strength	95% NEP	(1.65)(0.16) = .26	Bolt Cap = 110k −1σ = 0.14 Buckling Stts −1σ = 0.04 Holddown water = 1/2 −1σ = 0.07 0.25
Inelastic Response	ln 1.1/1 (ie ≅ −2.33 σ)	0.10	Fμ = 1.0 (INP3 = 1.0)
Ground Motion	+1σ	0.22	INP2 = 1.11 INP1 = $e^{.20}$ = 1.22
Damping	+1σ	0.11	increase INP1 to (1.22)(2.12/1.12) also ampv = 2.12
		Σ = 0.67	

Other Parameters the same (note these includes the peak Horizontal to - average of two horizontal components factor, INP 2 = 1.11)

See Analysis on pages 19 – 28

Median level $NCLPF_M$ = 0.20g

84% NEP Mac Input = (0.20)(1.4) = 0.28 g

↑ see Appendix B

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MCAD FILES VTANK.HC1

CONSISTENT SET OF UNITS:

BASE UNITS:

$$in \equiv 1L$$

$$m \equiv 1M$$

$$sec \equiv 1T$$

DERIVED UNITS:

$$ft \equiv 12 \cdot in$$

$$lb \equiv m \cdot \frac{in}{sec^2}$$

$$k \equiv 1000 \cdot lb$$

$$g \equiv 32.2 \cdot \frac{ft}{sec^2}$$

$$ksi \equiv \frac{k}{in^2}$$

$$MPa \equiv 0.144 \cdot ksi$$

DEFINE MEDIAN PROPERTIES OF TANK:

$R := 240 \cdot in$	Radius of tank
$t_s := 0.375 \cdot in$	Thickness of tank wall
$\mu := 1.24$	Shell imperfection factor - for buckling cap.
$\delta := t_s$	Shell imperfection size - assumed equal to shell thickness (normal construction)
$t := 0.25 \cdot in$	Thickness of tank bottom
$E_s := 28000. \cdot ksi$	Modulus of elasticity of tank steel
$A_b := 3.14 \cdot in^2$	Area of tank holddown anchor bolt for stiffness calculations - use 2.50 for strength

$E_b := 29000.\cdot ksi$	Modulus of elasticity of bolt steel
$P_y := 110\cdot k$	Yield strength of bolt: use effective yield stress equalto average 44 ksi times the net bolt area = 2.50 sq in. This produces total capacity = 110 k.
$h_1 := 27\cdot in$	Height of bolt from nut to bottom of tank
$h_2 := 54.5\cdot in$	Height of bolt from nut to embedment plate
$f_y := 37\cdot ksi$	Tank median yield stress
$P_{DL} := 63.7\cdot k$	Dead weight of tank shell and roof
$P_{EQ} := 63.7\cdot k$	Vertical earthquake force due to tank wall and roof calculated for a 1.0g vertical acceleration.
$\delta_w := 62.4\cdot \frac{lb}{ft^3}$	Density of water
$h_w := 37\cdot ft$	Height of water in tank
$n := 8$	Number of bolts
$i := 1 ..n$	Loop over all bolts to define angle from symmetric axis to each bolt:
$\theta_i := (i - 1)\cdot \frac{2\cdot \pi}{n}$	Angle from symmetric axis to each bolt, i
$VOH := \frac{2}{3}$	Ratio of vertical to horizontal earthquake components
$ampv := 2.12$	Vertical amplification for the fluid due to tank radial expansion (median at 7% damping)
$inp1 := 1.22\cdot 1.12$	Input scale factor for higher spectral amplification relative to median input at 7% damping.
$inp2 := 1.11$	Scale factor for peak earthquake horizontal input compared to average of two horizontal components
$inp3 := 1.0$	Ductility increase factor which influences the vertical earthquake reduction in hold down forces

$$p := \delta_w \cdot h_w \cdot \left[1 - .4 \cdot inp1 \cdot inp3 \cdot ampv \cdot VOH \cdot \frac{a}{g}\right]$$ Water pressure at bottom of tank

$$w := p$$ Effective pressure on tank bottom

$$I := \left[\frac{1}{12}\right] \cdot \left[t^3\right]$$ Moment of inertia of bottom plate per length

$$P_{net} := -\left[P_{DL} - P_{EQ} \cdot .4 \cdot inp3 \cdot VOH \cdot \frac{a}{g}\right]$$ Net vertical force due to dead load and earthquake force positive up

$$M_{net} := 233800 \cdot inp1 \cdot inp2 \cdot k \cdot in$$ Net overturning moment due to earthquake (233800 k*in for 0.30g for 7% damping average of two horizontal components)

DETERMINE SHELL BUCKLING CAPACITY:

Tank buckling capacity based on "Seismic Design of Storage Tanks" by M. J. N. Priestley, J. H. Wood and B. J. Davidson, Bulletin of the New Zealand National Society For Earthquake Engineering, Vol. 19, No. 4, December 1986.

$$f_{c1} := 0.6 \cdot E_s \cdot \left[\frac{t_s}{R}\right]$$ Classical "perfect shell" buckling stress

BUCKLING IN MEMBRANE COMPRESSION (DIAMOND BUCKLING):

$$\sigma := 1 - \mu \cdot \left[\frac{\delta}{t_s}\right] \cdot \left[\left[1 + \frac{2}{\mu \cdot \left[\frac{\delta}{t_s}\right]}\right]^{\frac{1}{2}} - 1\right]$$ Imperfection reduction factor

$$\Gamma := \frac{f_y}{\sigma \cdot f_{c1}}$$ Factor to determine THICK or THIN wall

$$f_o := f_y \cdot \left[1 - \frac{\Gamma}{4}\right] \cdot (\Gamma < 2) + \sigma \cdot f_{c1} \cdot (\Gamma > 2)$$

elastic THICK wall   elastic THIN wall

$$P := \frac{p \cdot R}{t_s \cdot f_{c1}}$$ Normalized circumferential wall stress

$$P := P \cdot (P < 5) + 5 \cdot (P > 5)$$ P is Limited to 5

$$C := \sqrt{1 - \left[1 - \frac{P}{5}\right]^2 \cdot \left[1 - \left[\frac{f_o}{f_{c1}}\right]^2\right]}$$ Factor for wall thickness and pressure - not to exceed 1.0

$$f_p := f_{c1} \cdot C \cdot (C < 1) + f_{c1} \cdot (C > 1)$$ Stress limited by classical buckling stress

$$f_{md} := f_{c1} \cdot \left[0.19 + 0.81 \cdot \frac{f_p}{f_{c1}}\right]$$

$$f_{md} := f_{md} \cdot \left[f_{md} < f_y\right] + f_y \cdot \left[f_{md} > f_y\right]$$ Diamond buckling capacity limited by yield stress

ELASTIC PLASTIC COLLAPSE (ELEPHANT FOOT BUCKLING):

$$s := \frac{\frac{R}{t_s}}{400}$$

$$K := \left[1 - \left[\frac{p \cdot R}{t_s \cdot f_y}\right]^2\right] \left[1 - \frac{1}{1.12 + s^{1.5}}\right] \cdot \left[\frac{s + \frac{f_y}{250 \cdot MPa}}{s + 1}\right]$$

$f_{me} := f_{c1} \cdot K \cdot (K < 1) + f_{c1} \cdot (K > 1)$ Limit elephant foot buckling stress to classical buckling stress

MINIMUM BUCKLING STRESS:

$$f_m := f_{md} \cdot \left[f_{md} < f_{me}\right] + f_{me} \cdot \left[f_{me} < f_{md}\right]$$

This is the buckling stress used:

$\frac{R}{t_s} = 640$ $\frac{p \cdot R}{t_s} = 60.36 \cdot MPa$

$f_m = 13.919 \cdot ksi$ $f_{md} = 96.662 \cdot MPa$ $f_{me} = 118.703 \cdot MPa$

$f_{s\alpha} := f_m$ Allowable tank buckling stress

DEFINE FORCES AND MOMENTS AT TANK BOTTOM AT THE CENTER:

$j := 1 \,..3$

$\theta_{NA_j} := \theta_{st} + .01 \cdot (j - 1)$

Compressive force due to contact between tank shell and base, positive downward:

$$P_{s_j} := 2 \cdot f_{s\alpha} \cdot R \cdot \frac{t_s}{1 - \cos\left[\theta_{NA_j}\right]} \cdot \left[\theta_{NA_j} \cdot \cos\left[\theta_{NA_j}\right] - \sin\left[\theta_{NA_j}\right]\right]$$

where θ.NA is angle to neutral axis

$\frac{P_{s_j}}{k}$
-749.089
-765.617
-782.138

Moment due to contact between tank shell and base, when resists earthquake overturning:

$$M_{s_j} := \frac{f_{s\alpha} \cdot R^2 \cdot t_s}{1 - \cos\left[\theta_{NA_j}\right]} \cdot \left[\theta_{NA_j} - \frac{1}{2} \cdot \sin\left[2 \cdot \theta_{NA_j}\right]\right]$$

$\frac{M_{s_j}}{1000 \cdot k \cdot in}$
176.181
179.905
183.616

Force due to a single bolt:

$$const_j := \frac{A_b \cdot f_{s\alpha}}{1 - \cos\left[\theta_{NA_j}\right]} \cdot \left[\frac{h_1}{h_2}\right] \cdot \left[\frac{E_b}{E_s}\right]$$

$$P2_j := const_j \cdot \left[\cos\left[\theta_{NA_j}\right] - \cos\left[\theta_2\right]\right]$$

$$P3_j := const_j \cdot \left[\cos\left[\theta_{NA_j}\right] - \cos\left[\theta_3\right]\right]$$

$$P4_j := const_j \cdot \left[\cos\left[\theta_{NA_j}\right] - \cos\left[\theta_4\right]\right]$$

$$P5_j := const_j \cdot \left[\cos\left[\theta_{NA_j}\right] - \cos\left[\theta_5\right]\right]$$

Modify bolt force to be greater than 0 but less than Py:

$$P2_j := P2_j \cdot \left[P2_j > 0 \cdot k\right] \cdot \left[P2_j < P_y\right] + P_y \cdot \left[P2_j > P_y\right]$$

$$P3_j := P3_j \cdot \left[P3_j > 0 \cdot k\right] \cdot \left[P3_j < P_y\right] + P_y \cdot \left[P3_j > P_y\right]$$

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$$P4_j := P4_j \cdot \left[P4_j > 0 \cdot k\right] \cdot \left[P4_j < P_y\right] + P_y \cdot \left[P4_j > P_y\right]$$

$$P5_j := P5_j \cdot \left[P5_j > 0 \cdot k\right] \cdot \left[P5_j < P_y\right] + P_y \cdot \left[P5_j > P_y\right]$$

Force due to sum of all bolts, positive downward:

$$P_{b_j} := 2 \cdot \left[P2_j + P3_j + P4_j\right] + P5_j$$

$$\frac{P_{b_j}}{k}$$

637.107
631.528
626.302

Moment due to all bolts, positive when resists earthquake overturning:

$$M_{b_j} := -R \cdot \left[2 \cdot P2_j \cdot \cos\left[\theta_2\right] + 2 \cdot P3_j \cdot \cos\left[\theta_3\right] + 2 \cdot P4_j \cdot \cos\left[\theta_4\right] + P5_j \cdot \cos\left[\theta_5\right]\right]$$

$$\frac{M_{b_j}}{1000 \cdot k \cdot in}$$

48.953
49.899
50.786

Total hold down force from water from bottom plate, positive downward:
(This equation assumes that the there is no rotation at the tank shell/bottom plate interface as the plate picks up due to rocking.)

$$P_{L_j} := \frac{4\cdot R}{3\cdot 2}\cdot\left[72\cdot I\cdot w^3\cdot f_{s\alpha}\cdot h_1\right]^{\frac{1}{4}}\int_{\theta_{NA_j}}^{\pi}\left[\frac{\cos\left[\theta_{NA_j}\right]-\cos(\theta)}{1-\cos\left[\theta_{NA_j}\right]}\right]^{\frac{1}{4}}d\theta$$

Total resisting moment from water on bottom plate, positive when resists earthquake overturning:

$$M_{L_j} := \frac{-\left[4\cdot R^2\right]}{3\cdot 2}\cdot\left[72\cdot I\cdot w^3\cdot f_{s\alpha}\cdot h_1\right]^{\frac{1}{4}}\cdot\int_{\theta_{NA_j}}^{\pi}\left[\frac{\cos\left[\theta_{NA_j}\right]-\cos(\theta)}{1-\cos\left[\theta_{NA_j}\right]}\right]^{\frac{1}{4}}\cdot\cos(\theta)\,d\theta$$

$$w = 0.014\cdot\frac{k}{in^2}$$

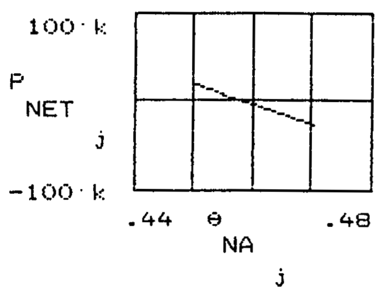
$\frac{P_{L_j}}{k}$
71.019
70.017
69.042

$\frac{M_{L_j}}{1000\cdot k\cdot in}$
5.089
5.068
5.047

$$P_{NET_j} := P_{s_j} + P_{b_j} + P_{L_j} - P_{net}$$ Vertical equilibrium equation

L7

θ_{NA_j}	$\frac{P_{NET_j}}{k}$
0.45	19.374
0.46	-3.736
0.47	-26.457



$$M_{u_j} := M_{b_j} + M_{L_j} + M_{s_j}$$

θ_{NA_j}	$\frac{M_{u_j}}{1000 \cdot k \cdot in}$
0.45	230.223
0.46	234.872
0.47	239.45

$$a_{CAP_j} := \frac{M_{u_j}}{M_{net}} \cdot (.30 \cdot g)$$

θ_{NA_j}	$\frac{a_{CAP_j}}{g}$
0.45	0.195
0.46	0.199
0.47	0.203

$i := 1$

$$a_{FIN} := a_{CAP_i} - P_{NET_i} \cdot \frac{a_{CAP_{i+1}} - a_{CAP_i}}{P_{NET_{i+1}} - P_{NET_i}} \qquad a_{FIN} = 0.198 \cdot g$$

$$\theta_{FIN} := \theta_{NA_i} - P_{NET_i} \cdot \frac{\theta_{NA_{i+1}} - \theta_{NA_i}}{P_{NET_{i+1}} - P_{NET_i}} \qquad \theta_{FIN} = 0.458$$

Some parameters:

These are the starting values: $\theta_{st} \equiv .45$ $\quad a \equiv .198 \cdot g$

$f_{s\alpha} = 13.919 \cdot ksi$

$P_y = 110 \cdot k$

inp1 = 1.366

inp2 = 1.11

inp3 = 1

VOH = 0.667

ampv = 2.12

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Revision to calculations Subsequent to Nov 12-13, 1987 Meeting

Include inertial water pressure at tank bottom

Allowable buckling stress decreases
Hold down force on bottom plate increases

Rerun TANK.HC2 (i.e. VTANK.HC2)

$$Capacity = 0.354g$$

$$\check{a}_{pga}\,(peak\ 84\%) = (1.4)(1.1)(.354) = 0.55g$$

$$HCLPF_{Peak\ 84\%} = 0.55\,e^{-1.65(.20+.24)} = 0.28g$$

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BY NRC DATE 10/4/87 PROJECT ______ PAGE 1 OF ______
CHKD. BY ML DATE 12/12/87 SUBJECT ______ JOB NO. 105-17

AUXILIARY CONTACTOR CHATTER AT GROUND

1. Capacity : EPRI GERS * (see page D-39)

   * "Generic Seismic Ruggedness of Power Plant Equipment," EPRI NP-5223, 1 May, 1987.

2. Failure (see page 2)

Approach	HCLPF Median Prob Ave Horiz Dir	HCLPF 84% NEP Max Horiz Dir	Median 84% NEP Max Horiz Dir
Fragility Analysis	0.34g	0.48g	1.20g
CDFM (back fit)	0.34g	0.48g	—

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See the Calculation for the MCC at Floor Level for discussion of general Assumption

The difference here is that the ground response spectrum (5% damped anchored to 1g) is the input

Failure is assumed to occur if the ground response spectral ordinate exceed the random GERS

See Program on page 3-9. The results are

$$\check{p}ga = 0.857g$$

$$\beta = 0.27$$

Note that this could have been obtained directly by seeing that the input would have to be scaled to the median GERS, i.e

$$\check{p}ga = \frac{1.5 \times 1.2}{2.12g} \times 1.0g = 0.849g$$

(1.5 × 1.2: median GERS; 2.12 g: 5% damped plateau)

$$\beta = 0.27 \text{ (from the GERs)}$$

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MCAD FILES MCC.GND

NORMAL DISTRIBUTION:

$b1 \equiv .319381530$

$b2 \equiv -.356563782$

$b3 \equiv 1.781477937$

$b4 \equiv -1.821255978$

$b5 \equiv 1.330274429$

$p \equiv .2316419$

$$Z(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION:

$c0 \equiv 2.515517$

$c1 \equiv .802853$

$c2 \equiv .010328$

$d1 \equiv 1.432788$

$d2 \equiv .189269$

$d3 \equiv .001308$

$$t(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$INOR(p) \equiv (-1)^{p < .5} \cdot \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3}\right]$$

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READ IN AND PLOT THE BASIC FLOOR RESPONSE SPECTRUM GERS:

$i := 0 \,..40$

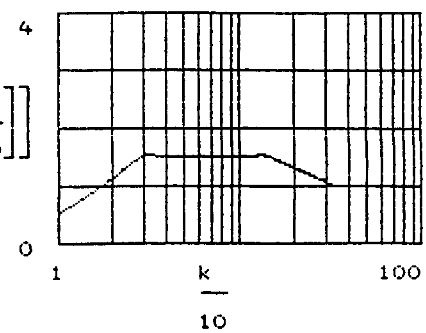
$BGER_i := READ(BGER)$ Median GERS values

$LFRE_i := READ(LFRE)$ Logarithms of the corresponding frequencies

$s1 := cspline(LFRE,BGER)$ This fits a cubic spline through the GERS values

$k := 1 \,..600$

$interp\left[s1,LFRE,BGER,\ln\left[\frac{k}{10}\right]\right]$



READ IN THE MEDIAN GROUND RESPONSE SPECTRUM NORMALIZED TO 1.0G:

$l := 0 \,..40$

$GRSP_l := READ(GRSP)$ Median ground response spectrum values

$LFRG_l := READ(LFRG)$ Logarithms of the corresponding frequencies

$s3 := cspline(LFRG,GRSP)$

This function computes 1 if either the Sa value is greater than the GERs, otherwise 0 and sums the values over all n trials:

$$TOTAL(c) := \sum_{k} \frac{c \cdot GSPEC_k > GF_k \; GERSP_k}{n}$$

$N := 10$

$j := 0 \,..N$

$pga_j := j \cdot .10 + .5$

$\%fail_j := TOTAL\left[pga_j\right]$

pga_j	$\%fail_j$
0.5	0.025
0.6	0.09
0.7	0.235
0.8	0.395
0.9	0.56
1	0.715
1.1	0.82
1.2	0.895
1.3	0.94
1.4	0.965
1.5	0.98

This calculation take the probabilities of failure and the associated peak ground acceleration values and performs a least squares fit in the log-probability domain to obtain the "best fit" median and β value.

$y1_j := \ln\left[pga_j\right]$

$x1_j := INOR\left[\%fail_j\right]$

$slope(x1,y1) = 0.271$ $\beta := slope(x1,y1)$

$intercept(x1,y1) = -0.154$ $a_{med} := \exp(intercept(x1,y1))$

$corr(x1,y1) = 1$

$a_{med} = 0.857$

$\beta = 0.271$

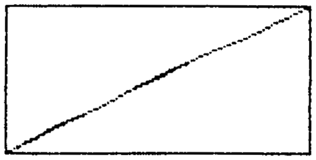
$z1_j := intercept(x1,y1) + slope(x1,y1) \cdot x1_j$

$max(y1) = 0.405$

$min(y1) = -0.693$

$max(x1) = 2.054$

$min(x1) = -1.96$

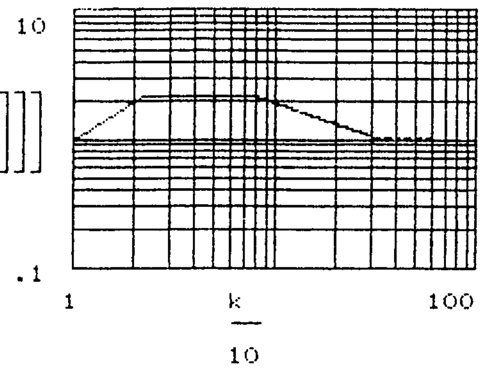


j	$x1_j$	$y1_j$	$z1_j$
5	−1.96	−0.693	−0.686
6	−1.341	−0.511	−0.518
7	−0.722	−0.357	−0.35
8	−0.266	−0.223	−0.227
9	0.151	−0.105	−0.113
10	0.568	0	$-3.099 \cdot 10^{-4}$
11	0.915	0.095	0.094
12	1.254	0.182	0.186
13	1.555	0.262	0.268
14	1.812	0.336	0.337
15	2.054	0.405	0.403
16			

5

This is the 5 % damped spectrum

$$\exp\left[\operatorname{interp}\left[s3, LFRG, GRSP, \ln\left[\frac{k}{10}\right]\right]\right]$$



DETERMINE POINTS ON FRAGILITY CURVE BASED ON LATIN HYPERCUBE SIMULATION CONSIDERING THE FOLLOWING VARIABLES BEING UNCERTAIN:

Equipment frequency:	$\beta = 0.20$	median = 6.5 hz
GERS:	$\beta = 0.27$	median = 1.2x

Total of n simulations are performed

$n := 200$

$k := 1 \,..n$

$$EF_k := 6.5 \cdot \exp\left[0.20 \cdot INOR\left[\frac{k - 1 + rnd(1)}{n}\right]\right]$$

This creates n equipment fre values - note it is based on a combined variability

$$GF_k := 1.2 \cdot \exp\left[0.27 \cdot INOR\left[\frac{k - 1 + rnd(1)}{n}\right]\right]$$

This creates n factors to scale the GERS capacity

$p := 1 .. 10$

p	EF_p	GF_p
1	3.727	0.556
2	4.079	0.61
3	4.09	0.657
4	4.22	0.68
5	4.355	0.701
6	4.423	0.709
7	4.468	0.729
8	4.539	0.747
9	4.625	0.756
10	4.642	0.769

$WRITE(EF) := EF_k$

$WRITE(GF) := GF_k$

The program is transferred to DOS where the files are randomly mixed using program CADMIX

$EF_k := READ(EF)$

$GF_k := READ(GF)$

$GSPEC_k := \exp\left[interp\left[s3, LFRG, GRSP, \ln\left[EF_k\right]\right]\right]$ Ground spectrum values at random frequencies

$GERSP_k := interp\left[s1, LFRE, BGER, \ln\left[EF_k\right]\right]$ GERs capacity values at random frequencies

p	EF_p	GF_p	$GSPEC_p$	$GF_p \cdot GERSP_p$
1	8.395	1.248	2.076	1.872
2	8.505	1.979	2.062	2.969
3	7.55	0.945	2.127	1.418
4	6.374	1.067	2.114	1.6
5	5.931	1.066	2.116	1.599
6	6.592	1.109	2.113	1.664
7	6.369	1.261	2.114	1.891
8	3.727	1.045	2.116	1.564
9	8.058	0.9	2.111	1.35
10	9.832	1.093	1.894	1.64

Tabulate probabilities to compare calculated and best fit values:

$a_j := \exp\left[y1_j\right]$ Peak ground accelerations

$p_j := \mathrm{NOR}\left[x1_j\right]$ Probability of failure values

$p_{best_j} := \mathrm{NOR}\left[\dfrac{\ln\left[\dfrac{a_j}{a_{med}}\right]}{\beta}\right]$ Best fit probability of failure values

j	a_j	p_j	p_{best_j}
5	0.5	0.025	0.012
6	0.6	0.09	0.059
7	0.7	0.235	0.166
8	0.8	0.395	0.323
9	0.9	0.56	0.497
10	1	0.715	0.654
11	1.1	0.82	0.776
12	1.2	0.895	0.863
13	1.3	0.94	0.919
14	1.4	0.965	0.954
15	1.5	0.98	0.974
16	1.6	0.985	0.986

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Develop Fragility Parameters $\check{F}$, β_r, β_u & HCLPF

Capacity

Strength (see page 2)

$\check{F} = 0.86g/0.86g = 1.0$

$\beta_u = 0.27$

$\beta_r = 0$

Inelastic Response

Assume this parameter is included in GERS

Response

Ground Motion This parameter includes

1) Peak-to-peak spectral variation
2) Horizontal direction variability
3) Equipment horizontal component coupl.

Assume critical element coupling is small

From Appendix A.

$\check{F} = 1.0$

$\beta_r = 0.25$

$\beta_u = 0$

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Damping:

		S_a (Ground at 6.5 hz)
Median	5%	2.12
-1σ	3%	2.46

$$\beta_U = \frac{1}{1} \ln \frac{2.46}{2.12} = 0.15$$

Modeling*

frequency - included in simulation (a small effect)

Mode shape - (assume included in SERs)

Mode Combination*

Assume included in SERs

Horizontal Component Phasing*

Assume included in SERs

SSI (Rock site)

$\check{F} = 1.0 \quad \beta_R = \beta_U = 0$

* $\tilde{F} = 1.0$

$\beta_R = 0$

$\beta_U = 0$

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Parameter	$\check{F}$	β_r	β_u
Capacity			
Strength	1.0	0	0.27
Inelastic Response	1.0	0	0
Equipment Response			
Spectral Shape			
Damping			
Modeling – Frequency		N/A	
– Mode Shape			
Mode Combination			
Horiz. Component Phasing			
Structure Response			
Ground Motion	1.0	0.25	0
Damping	1.0	0	0.15
Modeling – Frequency	1.0	0	0
Mode Shape	1.0	0	0
Mode Combination	1.0	0	0
Horiz. Component Phasing	1.0	0	0
SSI	1.0	0	0
Inelastic Response	N/A	N/A	N/A
Combined:	1.0	0.25	0.31

$\check{a}_{pga} = (1.0)(0.86) = 0.86g$ $\check{a}_{pga}(\text{Peak } 84\%) = 0.86 \times 1.4 = 1.20g$

$HCLPF_m \; (.86)\, e^{-1.65(.25+.31)} = 0.34g$

$HCLPF_{\text{Peak } 84\%} \; 1.4\,(.34) = 0.48g$ (See Appendix B)

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Compute HCLPF Directly (See Appendix C) $\Sigma X_i \beta = 1.65(.25+.31) = 0.92$

Parameter	Conservatism	$X_i \beta$	Deterministic Input
Strength	97.5% NEP	$(1.96)(.27) = 0.53$	$(1.5)(1.2)e^{-1.96(.27)} = 1.06$ (median stress)
Ground Motion	$+1\sigma$	0.25	$e^{.25} = 1.28$
Damping	-1σ	0.15	2.46 NCLPF (← 3% damping SA)
		0.93	

All other parameters at median value

Median level HCLPF $= \frac{1.06}{(2.46)(1.28)} g = 0.34g$

84% max component $= (0.34g)(1.4) = 0.48g$

← see Appendix B

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CHKD BY RK DATE 12/12/87 SUBJECT ______ JOB NO. 105-170

AUXILIARY CONTACTOR CHATTER AT FLOOR LEVEL

1. Capacity: EPRI GER's* (see pg D-39)

   * "Generic Seismic Ruggedness of Power Plant Equipment," EPRI NP-5223, May 1987;

2. Failure (see page 2)

Approach	NCLPF Median Prob Ave Hor. Dir	NCLPF 84% NEP Max Horiz Dir	Median 84% NEP Max Horiz Dir
Fragility Analysis	0.08g	0.11g	0.43g
CDFM (backfit)	0.07g	0.10g	–

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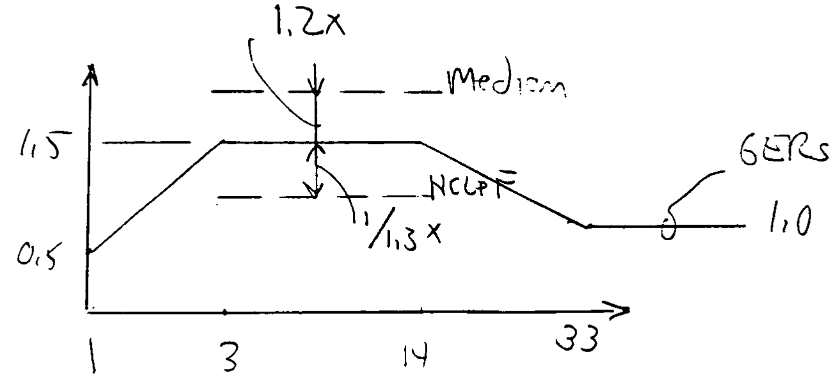
The median Fragility Curve was obtained by simulation

The GERs capacity Function was defined as follows:



$$\beta_U = \frac{\ln 1.2 \times 1.3}{1.65} = 0.27$$

Failure was considered to occur if either of the following occurs:

1) average floor response spectral ordinate (averaged over ~0.2 F) exceeds the random GERs

or 2) the floor response spectral ordinate exceeds 1.5 x the random GERs

Note: The spectra were pretty broad hence this definition could have used 1) with the average equal to the spectrum

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Simulation included

1. The ground response spectrum was varied randomly (up & down) due to shift in building frequency (a small effect !!).
2. The equipment frequency (at a combined β for variability in both the equipment and building frequency) was varied
3. The GERs were varied randomly (up & down)

The floor response spectrum was held constant at 5% damping

page 4–13 give the analysis and results

$$\check{p}ga = 0.31g$$

$$\beta_U = 0.53$$

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MCAD FILES MCC.FRA

NORMAL DISTRIBUTION:

$b1 \equiv .319381530$

$b2 \equiv -.356563782$

$b3 \equiv 1.781477937$

$b4 \equiv -1.821255978$

$b5 \equiv 1.330274429$

$p \equiv .2316419$

$$Z(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION:

$c0 \equiv 2.515517$

$c1 \equiv .802853$

$c2 \equiv .010328$

$d1 \equiv 1.432788$

$d2 \equiv .189269$

$d3 \equiv .001308$

$$t(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$INOR(p) \equiv (-1)^{p < .5} \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3} \right]$$

READ IN AND PLOT THE BASIC FLOOR RESPONSE SPECTRUM GERS:

$i := 0 \,..40$

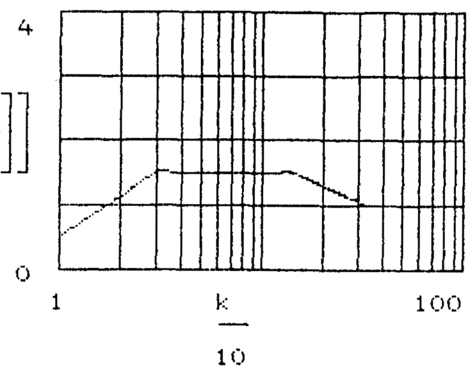
$BGER_i := READ(BGER)$ Median GERS values

$LFRE_i := READ(LFRE)$ Logarithms of the corresponding frequencies

$s1 := cspline(LFRE, BGER)$ This fits a cubic spline through the GERS values

$k := 1 \,..600$

$$interp\left[s1, LFRE, BGER, \ln\left[\frac{k}{10}\right]\right]$$



READ IN AND PLOT THE HORIZONTAL FLOOR RESPONSE SPECTRUM:

$j := 0 \,..45$

$FLSP_j := READ(FLSP)$ Floor response spectrum values

$LFRS_j := READ(LFRS)$ Logarithms of the corresponding frequencies

$s2 := cspline(LFRS, FLSP)$

READ IN THE HORIZONTAL FLOOR RESPONSE SPECTRUM AVERAGED OVER 0.2f FREQUENCY BAND

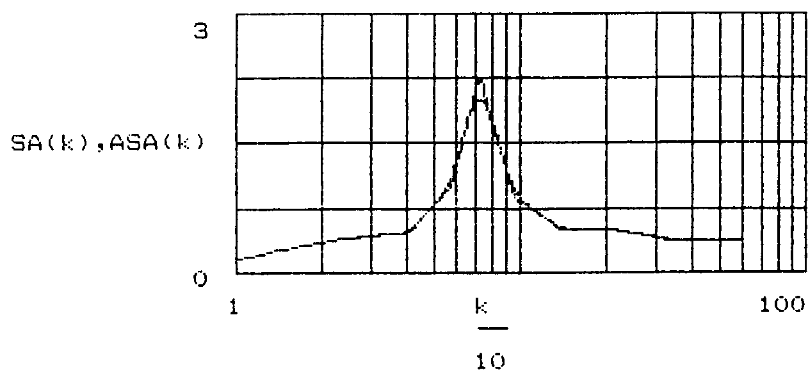
$AFLSP_j$:= READ(AFLSP) Average floor response spectrum values

s4 := cspline(LFRS,AFLSP)

$SA(x) := interp\left[s2,LFRS,FLSP,\ln\left[\frac{x}{10}\right]\right]$

$ASA(x) := interp\left[s4,LFRS,AFLSP,\ln\left[\frac{x}{10}\right]\right]$

Note: the average is not much different from spectrum



READ IN THE MEDIAN GROUND RESPONSE SPECTRUM NORMALIZED TO 1.0G:

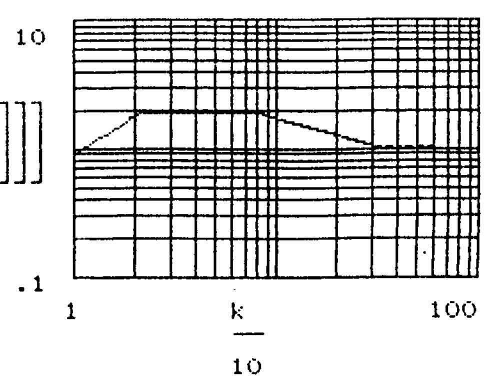
l := 0 ..40

$GRSP_l$:= READ(GRSP) Median ground response spectrum values

$LFRG_l$:= READ(LFRG) Logarithms of the corresponding frequencies

s3 := cspline(LFRG,GRSP)

$\exp\left[interp\left[s3,LFRG,GRSP,\ln\left[\frac{k}{10}\right]\right]\right]$



DETERMINE POINTS ON FRAGILITY CURVE BASED ON LATIN HYPERCUBE SIMULATION CONSIDERING THE FOLLOWING VARIABLES BEING UNCERTAIN:

Building frequency:	$\beta = 0.25$	median = 7.2 hz
Equipment frequency:	$\beta = 0.20$	median = 6.5 hz
Combined uncertainity:	$\beta = 0.32$	
GERS:	$\beta = 0.27$	median = 1.2x

Total of n simulations are performed

$n := 200$

$k := 1 \,..n$

$$BF_k := 7.2 \cdot \exp\left[0.25 \cdot INOR\left[\frac{k - 1 + rnd(1)}{n}\right]\right]$$

This creates n building freq values

CONST := exp(interp(s3,LFRG,GRSP,ln(7.2)))
CONST = 1.89

This constant is the ground spectrum amplification at the building frequency, 7.2hz

$$FF_k := \frac{\exp\left[interp\left[s3,LFRG,GRSP,\ln\left[BF_k\right]\right]\right]}{CONST}$$

This creates n factors to scale the floor spectra for different building frequencies

$$EF_k := 6.5 \cdot \exp\left[0.32 \cdot INOR\left[\frac{k - 1 + rnd(1)}{n}\right]\right]$$

This creates n equipment fr values - note it is based o a combined variability for building and equipment frequencies

$$GF_k := 1.2 \cdot \exp\left[0.27 \cdot INOR\left[\frac{k - 1 + rnd(1)}{n}\right]\right]$$

This creates n factors to scale the GERS capacity for the variability in the GERS

$p := 1 .. 10$ The first ten sets of samples before randomly mixing

p	FF_p	EF_p	GF_p
1	0.999	2.611	0.566
2	0.998	2.918	0.64
3	0.998	3.184	0.642
4	0.998	3.314	0.67
5	0.998	3.435	0.699
6	0.998	3.486	0.714
7	0.998	3.601	0.723
8	0.998	3.709	0.739
9	0.998	3.758	0.758
10	0.998	3.835	0.762

$WRITE(FF) := FF_k$

$WRITE(EF) := EF_k$

$WRITE(GF) := GF_k$

The program is transferred to DOS where the files are randomly mixed

$FF_k := READ(FF)$

$EF_k := READ(EF)$

$GF_k := READ(GF)$

$FSPEC_k := interp\left[s2, LFRS, FLSP, \ln\left[EF_k\right]\right]$ Floor spectral values at random frequencies

$ASPEC_k := interp\left[s4, LFRS, AFLSP, \ln\left[EF_k\right]\right]$ Average floor values at random frequencies

$GERSP_k := interp\left[s1, LFRE, BGER, \ln\left[EF_k\right]\right]$ GERs capacity values at random freq.

p	FF_p	EF_p	GF_p	$FF_p \cdot ASPEC_p$	$FF_p \cdot FSPEC_p$	$GF_p \cdot GERSP_p$
1	0.999	4.437	1.403	0.602	0.605	2.106
2	0.998	3.601	0.97	0.45	0.456	1.453
3	0.998	6.947	1.573	1.957	2.065	2.36
4	0.998	5.221	1.02	0.862	0.874	1.53
5	0.998	8.227	0.937	1.602	1.593	1.405
6	0.998	6.718	1.262	1.853	1.863	1.894
7	0.998	10.225	1.411	0.821	0.816	2.117
8	0.998	7.691	1.332	1.898	1.905	1.998
9	0.998	10.733	1.167	0.765	0.773	1.749
10	0.998	3.835	1.14	0.463	0.447	1.707

This function computes 1 if either the Sa value is greater than 1.5*GERS or if the average Sa valve is greater than 1.0*GERS, otherwise 0 and sums the values over all n trials:

$$TOT(c) := \sum_k \frac{1 - \left[c \cdot FF_k \cdot FSPEC_k < 1.5 \cdot GF_k \cdot GERSP_k\right] \cdot \left[c \cdot FF_k \cdot ASPEC_k < GF_k \cdot GERSP_k\right]}{n}$$

$a := 0.11$

$N := 10$

$j := 0 \,..N$

$$pga_j := \exp\left[\ln(a) + j \cdot \frac{\ln(N)}{N}\right]$$

Spaces a set of pga values evenly in the log domain

$$\%fail_j := TOT\left[\frac{pga_j}{0.18}\right]$$

This computes the fraction of failures (or fragility curve values) as a function of the pga values

pga_j	$\%fail_j$
0.11	0.03
0.138	0.05
0.174	0.15
0.219	0.315
0.276	0.46
0.348	0.585
0.438	0.74
0.551	0.84
0.694	0.925
0.874	0.975
1.1	0.995

$WRITE(PGA) := pga_j$

$WRITE(FAIL) := \%fail_j$

MCAD FILES LOGNORML.FIL

This program take the calculated probabilities of failure and the associated peak ground acceleration values and performs a least squares fit in the log-probability domain to obtain the "best fit" median and ß value.

NORMAL DISTRIBUTION:

$b1 \equiv .319381530$

$b2 \equiv -.356563782$

$b3 \equiv 1.781477937$

$b4 \equiv -1.821255978$

$b5 \equiv 1.330274429$

$p \equiv .2316419$

$$Z(x) \equiv \frac{\exp\left[\dfrac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION:

$c0 \equiv 2.515517$

$c1 \equiv .802853$

$c2 \equiv .010328$

$d1 \equiv 1.432788$

$d2 \equiv .189269$

$d3 \equiv .001308$

$$t(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$INOR(p) \equiv (-1)^{p < .5} \cdot \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3} \right]$$

$N := 10$

$j := 0 \,..N$

$y1_j := READ(PGA)$

$x1_j := READ(FAIL)$

j	$y1_j$	$x1_j$
0	0.11	0.03
1	0.138	0.05
2	0.174	0.15
3	0.219	0.315
4	0.276	0.46
5	0.348	0.585
6	0.438	0.74
7	0.551	0.84
8	0.694	0.925
9	0.874	0.975
10	1.1	0.995

$y1_j := \ln\left[y1_j\right]$

$x1_j := INOR\left[x1_j\right]$

$slope(x1,y1) = 0.527$ $\qquad \beta := slope(x1,y1)$

$intercept(x1,y1) = -1.185$ $\qquad a_{med} := \exp(intercept(x1,y1))$

$corr(x1,y1) = 0.997$

$a_{med} = 0.306$

$\beta = 0.527$

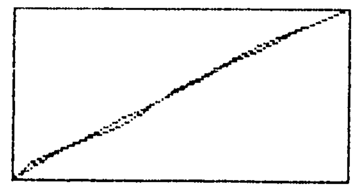
$$z1_j := \text{intercept}(x1,y1) + \text{slope}(x1,y1) \cdot x1_j$$

$\max(y1) = 0.095$

$\min(y1) = -2.207$

$\max(x1) = 2.576$

$\min(x1) = -1.881$



j	$x1_j$	$y1_j$	$z1_j$
0	-1.881	-2.207	-2.175
1	-1.645	-1.977	-2.051
2	-1.036	-1.747	-1.73
3	-0.481	-1.516	-1.438
4	-0.1	-1.286	-1.237
5	0.214	-1.056	-1.072
6	0.643	-0.826	-0.846
7	0.994	-0.595	-0.661
8	1.44	-0.365	-0.426
9	1.96	-0.135	-0.152
10	2.576	0.095	0.173

Tabulate probabilities to compare calculated and best fit values:

$a_j := \exp\left[y1_j\right]$ Peak ground accelerations

$p_j := \text{NOR}\left[x1_j\right]$ Probability of failure values

$$p_{best_j} := \text{NOR}\left[\frac{\ln\left[\dfrac{a_j}{a_{med}}\right]}{\beta}\right]$$ Best fit probability of failure values

j	a_j	p_j	p_{best_j}
0	0.11	0.03	0.026
1	0.138	0.05	0.066
2	0.174	0.15	0.143
3	0.219	0.315	0.264
4	0.276	0.46	0.423
5	0.348	0.585	0.596
6	0.438	0.74	0.752
7	0.551	0.84	0.868
8	0.694	0.925	0.94
9	0.874	0.975	0.977
10	1.1	0.995	0.992

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Develop Fragility Parameters, $\check{F}$, β_r, β_u & HCLPF

Capacity

Strength (see page 11)

$$\check{F} = 0.31g/0.31g = 1.0$$

$$\beta_u = 0.53$$

$$\beta_r = 0$$

Inelastic Response

Assume this parameter is included in GERs

$\check{F} = 1.0$ $\beta_r = \beta_u = 0$

Equipment Response

Spectral Shape (difference between smoothed spectra and raw spectra)

$F = 1.0$

$\beta_r = 0$

$\beta_u = 0.05$ (estimate)

Damping

Median damping = 5% (This is basis for calculation)

$\check{F} = 1.0$

$-1\sigma = 3.5\%$ damping

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Mean MCC.FRA 2 3.5% damping spectra

$$\check{S}_a = 0.27g$$

$$\beta_U = \frac{1}{1} \ln \frac{0.31}{0.27} = 0.14$$

$$\beta_R = 0$$

Modeling

Frequency & Mode Shape - Assume var is included in the GERS*

Mode Combination

Assume included in GERS*

Horizontal Component Phasing

Assume included in GERS*

*
$\check{F} = 1.0$
$\beta_R = 0$
$\beta_U = 0$

Structure Response

Ground Motion This Parameter includes:

1) Peak to Peak Spectrum Variation
2) Horizontal Direction Variability
3) Equipment Horizontal Component Coupling

Assume out-of-plane element Coupling small

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From Appendix A:

$\check{F} = 1.0$

$\beta_r = 0.25$

$\beta_u = 0$

<u>Damping</u> — <u>S_a (Ground at 7.2 hz)</u>

Median: 7% — 1.89 g

-1σ : 5% — 2.12 g

$$\beta_u = \frac{1}{1} \ln \frac{2.12}{1.89} = 0.11$$

$\beta_r = 0$

$\check{F} = 1.0$ (used 7% in strength calc)

<u>Modeling</u>

Frequency - Since building frequency is in region of ground spectrum plateau little change will occur (also see Equip. Modeling - Frequency)

$\beta_r = \beta_u = 0$ $\check{F} = 1.0$

Mode Shape - (controlled by first mode - estimate)

$\check{F} = 1.0$ $\beta_r = 0$ $\beta_u = 0.10$ (est)

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Mode Combination (controlled by first mode - est.)

$$\check{F} = 1.0 \quad \beta_r = \beta_u = 0$$

Horizontal Component Phasing

$$\check{F} = 1.0 \quad \beta_r = 0.10 \quad \beta_u = 0 \quad (\text{est})$$

SSI (Rock Site)

$$\check{F} = 1.0 \quad \beta_r = 0 \quad \beta_u = 0.05 \quad (\text{est.})$$

Inelastic Response (Structure assumed linear)

$$\check{F} = 1.0 \quad \beta_r = \beta_u = 0$$

BY ____ DATE ____	PROJECT MCC @ Floor level	PAGE 18 OF ____
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Parameter	$\check{F}$	β_r	β_u
Capacity			
Strength	1.0	0	0.53
Inelastic Response	1.0	0	0
Equipment Response			
Spectral Shape	1.0	0	0.05
Damping	1.0	0	0.14
Modeling – Frequency	1.0	0	0
– Mode Shape	1.0	0	0
Mode Combination	1.0	0	0
Horiz. Component Phasing	1.0	0	0
Structure Response			
Ground Motion	1.0	0.25	0
Damping	1.0	0	0.11
Modeling – Frequency	1.0	0	0
Mode Shape	1.0	0	0.10
Mode Combination	1.0	0	0
Horiz. Component Phasing	1.0	0.10	0
SSI	1.0	0	0.05
Inelastic Response	1.0	0	0
Combined:	1.0	0.27 ✓	0.57

$\check{a}_{pga} = (1.0)(0.31)g = 0.31g$ $\quad a_{pga}(\text{Peak } 84\%) = 1.4(.31g) = 0.43g$

$HCLPF_m\ (0.31)\ e^{-1.65(.27+.57)} = 0.08g$

$HCLPF_{Peak\ 84\%}\ 1.4(.08) = 0.11g$ (see Appendix B)

B

Compute HCLPF Directly (see Appendix C) $\Sigma X \cdot \beta = 1.65(.27+.57) = 1.3$

Parameter	Conservatism	$X \cdot \beta$	Deterministic Input
Strength	95% NEP	$(1.65)(.53) = 0.87$	See discussion below
Equip Damping	-1σ	0.14	Use 3.5% floor spect
Ground Motion	$+1\sigma$	0.25	$e^{.25} = 1.28$
Building Damping	-1σ	0.11	$\frac{2.12}{1.89} = 1.12$
		$\Sigma = 1.37$	

Keep other parameters at median level

Since the strength was determined by direct simulation (i.e counting failures & non failures in a Latin Hypercube simulation) an alternate means is need to back calculate the strength effect. Since the GERS is essentially flat in the ~6.5 hz ± region, one could get the 95% NEP capacity by averaging the scaled 3.5% damped floor spectra over the frequency variation due to both β_U for building & equip (i.e $\beta = 0.32$). Note the scaling is for the effect of ground motion and building damping (i.e., 1.28×1.12). Once the median capacity and β_U is found

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then the β_u for the GERs (i.e., 0.27) must be added to get a total variability. Then the input must be scaled (from 0.18g pga) so that there is only a 5% prob of exceeding the median GERs

Fig 1 thru 23 gave the simulation to obtain the median input (this is for 3.5% floor spectra corresponding to 7% building damping and median ground motion input scaled to 0.18g pga), ie

$$\check{S}_a = 1.29g$$
$$\beta_u = 0.50$$

scale for other effects:

$$\check{S}_a = (1.29)(1.28 \times 1.12) = 1.85g$$

(1.28: 1σ ground motion; 1.12: 5% building damping)

$$\beta_u = (0.50^2 + 0.27^2)^{1/2} = 0.57$$

(0.27: GERs)

Now compare to GERs distribution with

$$\check{S}_a = 1.5 \times 1.2 = 1.8g$$
$$\beta = 0.57$$

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MCAD FILES SPECT.SIM

NORMAL DISTRIBUTION

$b1 \equiv .319381530$

$b2 \equiv -.356563782$

$b3 \equiv 1.781477937$

$b4 \equiv -1.821255978$

$b5 \equiv 1.330274429$

$p \equiv .2316419$

$$Z(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION

$c0 \equiv 2.515517$

$c1 \equiv .802853$

$c2 \equiv .010328$

$d1 \equiv 1.432788$

$d2 \equiv .189269$

$d3 \equiv .001308$

$$t(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$INOR(p) \equiv (-1)^{p < .5} \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3}\right]$$

22

$Freq_{hat} := 6.5$

$\beta := 0.32$

$n := 199$

$i := 0 \,..n$

$$Freq_i := Freq_{hat} \cdot \exp\left[\beta \cdot INOR\left[\frac{i + rnd(1)}{n+1}\right]\right]$$

$LFreq_i := \ln\left[Freq_i\right]$

$m := 45$

$j := 0 \,..m$

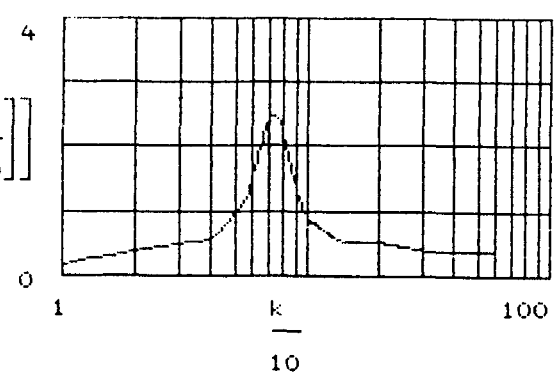
$XLFR_j := READ(XLFR)$

$XSA_j := READ(XSA)$ Read in the averaged 3.5% damped spectrum

$s := cspline(XLFR,XSA)$

$k := 1 \,..600$

$interp\left[s,XLFR,XSA,\ln\left[\frac{k}{10}\right]\right]$



$SA_i := interp\left[s,XLFR,XSA,LFreq_i\right]$

$LSA_i := \ln\left[SA_i\right]$

Median := exp(mean(LSA))

Beta := stdev(LSA)

Median = 1.287

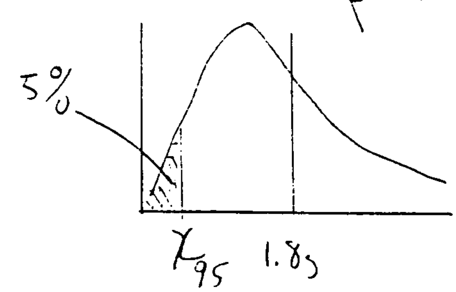
Beta = 0.498

Fmedian := exp(mean(LFreq))

Fbeta := stdev(LFreq)

Fmedian = 6.482

Fbeta = 0.327



$$x_{95} = 1.8\, e^{-(.57)(1.65)} = 0.70g$$

Thus the HCLPF is just:

Median Level: $HCLPF = \frac{0.70}{1.85} \times 0.18 = 0.07$

$1.85 = \check{S}_a$ (see page 20)

84% MAC component $= (0.07)(1.4) = 0.10$
↑ see Appendix B

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STARTING AIR TANK

Assumption (special)

1. Failure is 0.4 in displacement at C.G. of Tank (≈ 1% drift). This assumes connecting piping will fail at this displacement

Approach	HCLPF Median Prob. Avg Hor Dir	HCLPF 84% NEP Max Horiz Dir	Median 84% NEP Max Horiz Dir
Fragility Analysis	0.30 g	0.43 g	1.40 g
CDFM (backfit)	0.29 g	0.41 g	–

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Calculate Capacity of Connection

(code yield to median yield)

bolt: 3/8 ϕ $\sigma = 1.2 \times 36 = 43.2$ ksi

Area = .226 in² (across thread)

$F_{TEN} = (43.2)(.226) = \underline{9.8^k}$

Angle 2 × 2½ × 3" long ¼" thick

$M_{CAP} = S\sigma$ $S = \frac{(3.0)(.25)^2}{6} = 0.03125 \text{ in}^2$

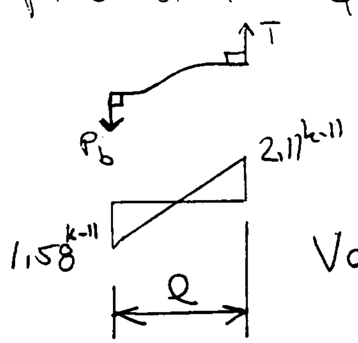
$\sigma = 1.25 \times 36 = 45$ ksi

Full plasticity across section ↓ ↑ code to median yield

at Tank: $M_{CAP} = (1.5)(.03125)(45) = 2.11^{k\text{-}"}$

at bolt $M_{CAP} = 2.11 \times \frac{3-3/4}{3} = 1.58^{k\text{-}"}$

Assume fixed at tank & bolt: $T = \frac{M_1+M_2}{\ell} = \frac{2.11+1.58}{1.75} = \underline{2.11^k}$



Hence bending of ∠ controls:

Variability 1.25 factor ∴ $\beta_U = \frac{1}{1.65} \ln\frac{1.25}{1.0} = .14$

1.5 factor $\beta_U \frac{1}{1.65} \ln\frac{1.5}{1.4} = .04$

$\beta_U = (.14^2 + .04^2)^{1/2} = 0.15$

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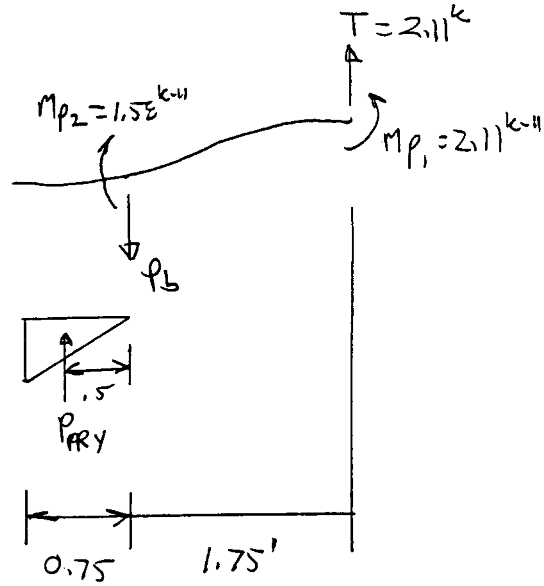
Check Prying Action:



$$P_{PRY} = \frac{1.58}{.5} = 3.16^k$$

$P_b = 2.11 + 3.16 = 5.25^k < 9.8^k$ tension capacity

This indicates that angle bottom plate will become a mechanism before bolt fails

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Calculate Frequency of Accumulator (horiz)

$$I_{tank} = \frac{\pi}{4}\left[12^4 - (12 - 3/8)^4\right] = 1942 \text{ in}^4$$

For 1^k load at C.G. displacement is: $\left(\Delta = \frac{PD^3}{3EI}\right)$

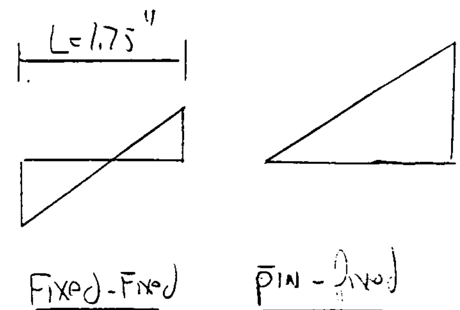
$$\Delta = \frac{(1)(41.5)^3}{(3)(29\times10^3)(1942)} \cong 4\times10^{-4} \text{ in}$$

(very small)

Most displacement is in support system

Support system Stiffness

Angle: Moment ranges between



$$I_x = (3)(.25)^3/12 = .003906 \text{ in}^4$$

$$k_{MAX} = \frac{(12)(29\times10^3)(.003906)}{(1.75)^3} = 253.6 \text{ k/in} \qquad K_{MAX} = \frac{12EI}{L^3} \qquad K_{MIN} = \frac{3EI}{L^3}$$

$$K_{MIN} = 1/4\,(253.6) = 63.4 \text{ k/in}$$

bolt: $k = AE/L$ $\qquad A = (5/8)^2\frac{\pi}{4} = .307 \text{ in}^2$

$$= \frac{(.307)(29\times10^3)}{6} \qquad \ell = 6''$$

$$= 1484 \text{ k/in}$$

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Combined Stiffness for Support (1 angle + bolt)

MIN: $\frac{1}{K} = \frac{1}{63.4} + \frac{1}{1484}$ $K_{MIN} = 60.8$ k/in

MAX: $\frac{1}{K} = \frac{1}{253.6} + \frac{1}{1484}$ $K_{MAX} = 216.6$ k/in

Accumulator Stiffness

From Force Analysis (base rotation stiffness, K_θ)

$$K_\theta = \frac{M}{\theta} = \frac{10.77^{\text{in-lbs}}}{\psi} K$$ (see page 20)

where K is single angle stiffness

ψ = Normalized displacement computed in Force analysis

X dir: $\psi = P1/_{X1} \sin\theta$

Y dir $\psi = P1/_{X1} \cos\theta$

Combined X & y values range from about .012 to .025 k/in (see pg 25)

K_θ ranges from $\frac{10.77}{.025}(60.8) = 26{,}193$ k-in/rad

to $\frac{10.77}{.012}(216.6) = 194{,}400$ k-in/rad

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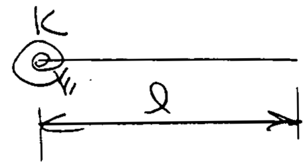
Using Table 8-10 pg 168 from
Formulas for natural frequency & mode shape,
by Blevins



$L = 91"$ $\quad m = 920/(386.4 \times 91)$

$I = 1942 \text{ in}^4$ $\quad = 2.62 \times 10^{-5} \ \frac{\text{k-sec}^2}{\text{in}^2}$

$E = 29 \times 10^3$ ksi

$K = 194{,}400,\ 26{,}193$ k-in/rad

$$\frac{Kl}{EI} = \frac{(194{,}400)(91)}{(29\times10^3)(1942)} \quad \text{or} \quad \frac{(26{,}193)(91)}{(29\times10^3)(1942)}$$

$= .314$ $\qquad\qquad = .042$

λ_1 (Table 8-10) $= .979$ $\qquad .581$ (see next page for interpolation)

$$f_1 = \frac{\lambda_1^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}$$

$$= \frac{(.979)^2}{2\pi(91^2)}\left(\frac{29\times10^3 \times 1942}{2.62\times10^{-5}}\right)^{1/2} \qquad = \frac{(.581)^2}{2\pi(91)^2}\left(\quad\right)^{1/2}$$

$= (.958)\ (28.2)$ Hz $\qquad (.338)\ (28.2)$ Hz

$= 27$ Hz $\qquad = 9.5$ Hz

Assume median $\tilde{f} = \sqrt{27 \times 9.5} = 16$ Hz

$$\beta = \frac{1}{4} \ln \frac{27}{9.5} = 0.26 \ (\text{use } 0.25)$$

MCADFILESBLEVINS.810 7

This interpolates first mode values for Table 8-10 in Blevins Book

$k_0 := .01$ $\mu_0 := .4159$

$k_1 := .1$ $\mu_1 := .7357$

$k_2 := 1$ $\mu_2 := 1.248$

$k_3 := 10$ $\mu_3 := 1.723$

$k_4 := 100$ $\mu_4 := 1.857$

$i := 0 ..4$

$Logk_i := \log\left[k_i\right]$

$s := \text{cspline}(Logk, \mu)$

$\text{interp}(s, Logk, \mu, \log(.314)) = 0.979$

$\text{interp}(s, Logk, \mu, \log(.042)) = 0.581$

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Check vertical frequency (consider only 1-angle effective)

$$f_{MIN} = \frac{1}{2\pi}\sqrt{\frac{K}{M}} = \frac{1}{2\pi}\sqrt{\frac{60.8}{(2.62\times10^{-5})91}}$$

(60.8 ← Min stiffness)

$$= 25.4 \text{ hz}$$

$$f_{MAX} = \sqrt{\frac{216.6}{60.8}}\,(25.4)$$

$$= 47.9$$

$$\breve{f} = \sqrt{(25.4)(47.9)} = 35 \text{ hz}$$

This frequency is likely to be conservative (i.e. low) since the capacity will corresponds to a vertical acceleration which is less than 1g; hence, the accumulator will not rise vertically

Hence Use $SA_v = 0.20g$ / 0.18g pga horiz

$$\beta_f = \frac{1}{1.25}\ln\left(\frac{47.9}{35}\right) = 0.25$$

↑ 90% confidence

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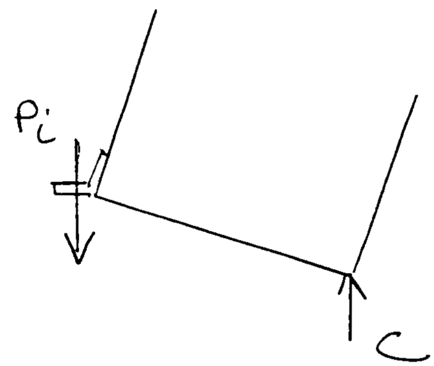
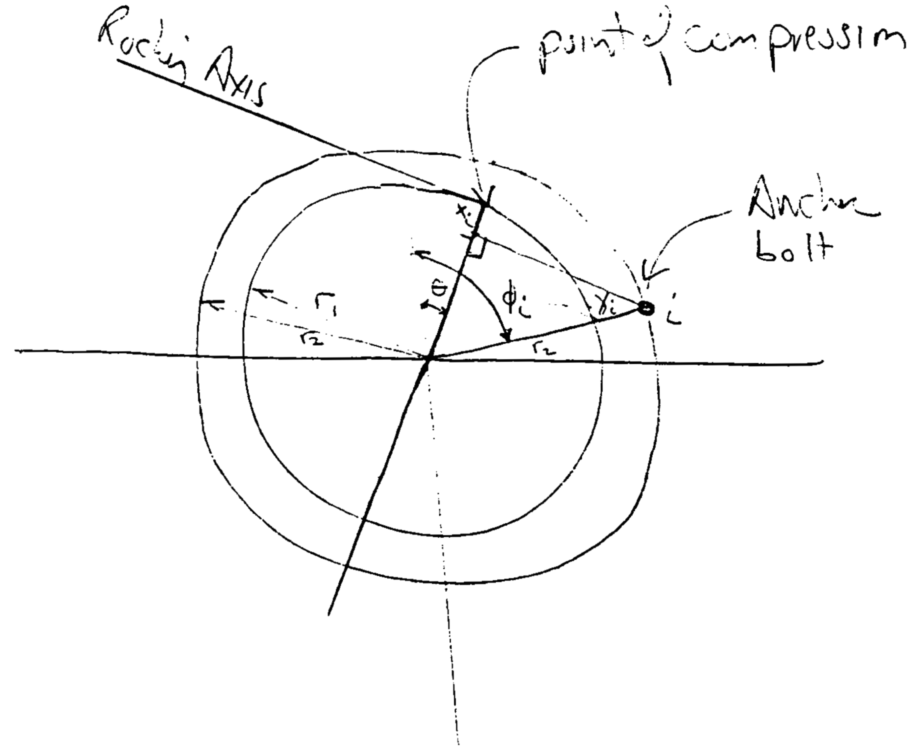
Analysis To Determine Force in Bolts

Assumption: The accumulator rocks as a rigid body with insignificant deformation relative to displacement in Anchors (bolt/angle). Note D/t is ≈64 for shell.



Following page gives derivation for calculating bolt forces

The derivation is followed by MATHCAD program for finding forces by trial & error (i.e. Θ, angle to compressive force, is unknown)

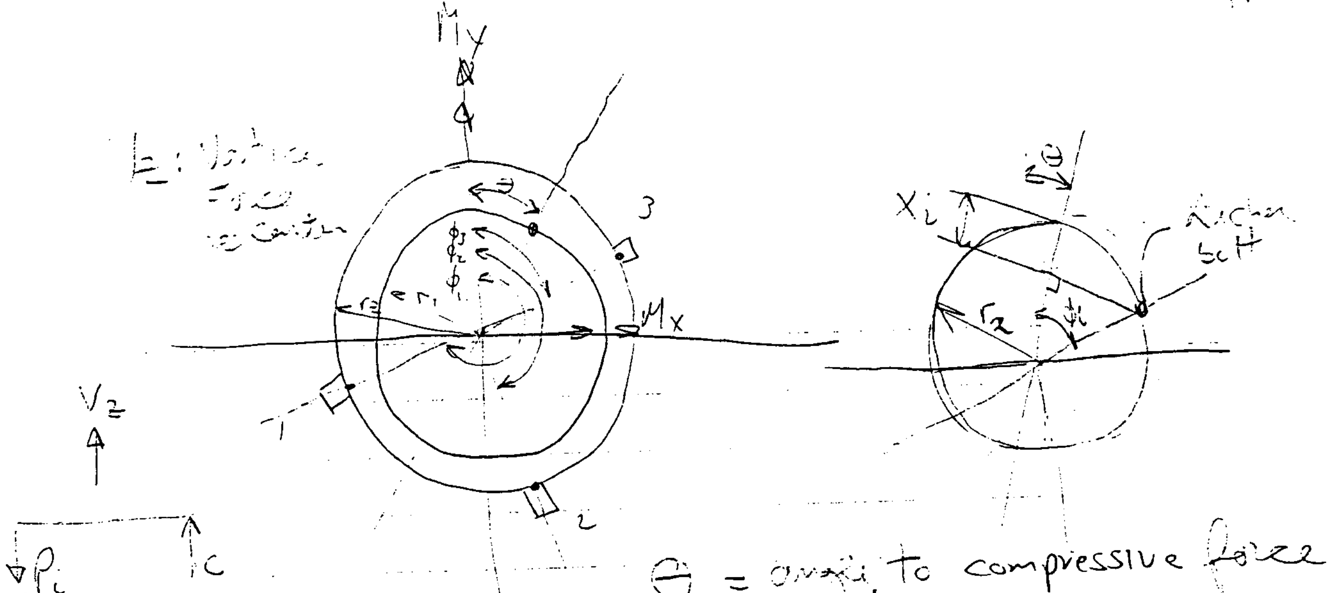


$$X_i = r_1 - r_2 \sin \gamma_i \qquad \gamma_i = \pi/2 - (\phi_i - \theta)$$

$$0 \le X_i \le r_1 + r_2$$

$$X_i = r_1 - r_2 \cos[\phi_i - \theta]$$

$$X_i = X_i * (X_i > 0)$$ (ie X_i must not be negative)



Θ = angle to compressive force location

— support stiffness

$P1 = X1 \alpha K_1$
$P2 = X2 \alpha K_2$
$P3 = X3 \alpha K_3$

$$P2 = P1 (X2/X1)(k2/k1)$$
$$P3 = P1 (X3/X1)(k3/k1)$$

C: positive ↑
Pi: positive ↓
Vz: positive ↑

$$= - P1\left(\frac{X1}{X1}\frac{K1}{K1} + \frac{X2}{X1}\frac{k2}{k1} + \frac{X3}{X1}\frac{k3}{K1}\right)_{\gamma_2} = 0$$

$$= r_1 \cos\theta - P1\, r_2 \left(\frac{X1}{X1}\frac{k2}{k1}\cos\phi_1 + \frac{X2}{X1}\frac{k2}{K1}\cdot\cos\phi_2 + \frac{X3}{X1}\frac{k3}{K1}\cos\phi_3\right) = -Mx$$

$$= r_1 \sin\theta - P1\, r_2 \underbrace{\left(\frac{X1}{X1}\frac{K1}{K1}\sin\phi_1 + \frac{X2}{X1}\frac{K2}{K1}\sin\phi_2 + \frac{X3}{X1}\frac{K3}{K1}\sin\phi_3\right)}_{\gamma_3} = My$$

$$C - \gamma_1 Pl = -V_z$$

$$\boxed{C = \gamma_1 Pl - V_z}$$

$$\gamma_1 = \Sigma x_i / Xl$$

$$\gamma_2 = \Sigma \frac{x_i \cos t_i}{Xl}$$

$$\gamma_3 = \Sigma \frac{x_i \sin t_i}{Xl}$$

$$(\gamma_1 Pl - V_z) r_1 \cos\theta - r_2 \gamma_2 Pl = -M_x \qquad (1)$$

$$(\gamma_1 Pl - V_z) r_1 \sin\theta - r_2 \gamma_3 Pl = M_y \qquad (2)$$

1) $$\cos\theta = \frac{-M_x + r_2\gamma_2 Pl}{r_1(\gamma_1 Pl - V_z)}$$

$$\cos\theta = \left[\frac{r_2\gamma_2 Pl - M_x}{r_1(\gamma_1 Pl - V_z)}\right]$$

$$\sin\theta = \left[\frac{+ r_2\gamma_3 Pl + M_y}{r_1(\gamma_1 Pl - V_z)}\right]$$

$$\sin^2\theta + \cos^2\theta = 1$$

$$(r_2\gamma_2 P1 - M_x)^2 + (+r_2\gamma_3 P1 + M_y)^2 = r_1^2(\gamma_1 P1 - V_z)^2$$

$$r_2^2\gamma_2^2 \; P1^2 \; - 2M_x r_2\gamma_2 P1 + M_x^2$$

$$r_2^2\gamma_3^2 P1^2 \; + 2M_y r_2\gamma_3 P1 + M_y^2$$

$$- r_1^2\gamma_1^2 P1^2 \; + 2V_z r_1^2\gamma_1 P1 - r_1^2 V_z^2 = 0$$

$$a P1^2 + b P1 + c = 0$$

$$a = r_2^2\gamma_2^2 + r_2^2\gamma_3^2 - r_1^2\gamma_1^2$$

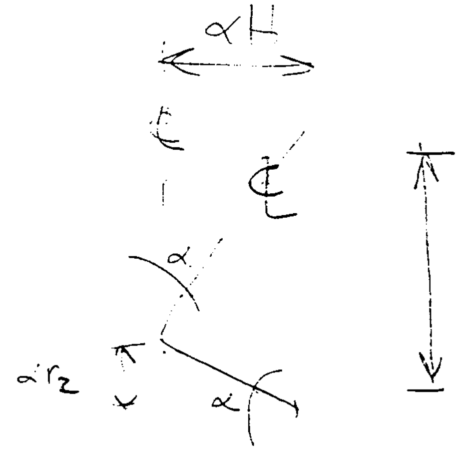
$$b = -2M_x r_2\gamma_2 + 2M_y r_2\gamma_3 + 2V_z r_1^2\gamma_1$$

$$c = M_x^2 + M_y^2 - r_1^2 V_z^2$$

$$P1 = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \qquad (P1 \text{ must be } +)$$

$$\Theta = \cos^{-1}\left[\frac{r_2\gamma_2 P1 - M_x}{r_1(\gamma_1 P1 - V_z)}\right]$$

displacement at C.G.



$$\alpha = \frac{Pl}{K \cdot Xl}$$

$$X_{disp} = \alpha H \sin\theta$$
$$y_{disp} = \alpha H \cos\theta$$

$$X_{disp} = \frac{Pl\,H}{K \cdot Xl} \sin\theta$$

$$X_{nor} = \frac{X_{disp} \cdot K}{H} = \frac{Pl}{Xl} \sin\theta$$

$$y_{nor} = \frac{y_{disp}\, K}{H} = \frac{Pl}{Xl} \cos\theta$$

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This program calculates the reactions of a three support circular vertical tank where the tank rotates as a rigid body and the location of the compressive reaction must be obtained by trial and error. The loading consists of a moment loading with a vector of 10 at an angle of A with the reference axis and a second moment of 4 at 90 degrees to the vector of 10.

$deg \equiv \frac{\pi}{180}$

This establishes the angle A at which the bolt forces will be calculated

$i := 0 ..15$

$A_i := 22.5 \cdot i \cdot deg$

$M := 10$

$V := 0$

$kr := 1.1$

kr is the relative stiffness of the support in compression to the support in tension. It is stiffer in compression since the bolt does not resist any compression

$r_1 := 12$

$r_2 := 14.25$

$\phi1 := 55 \cdot deg$

$\phi2 := 215 \cdot deg$

$\phi3 := 315 \cdot deg$

$M_x := M \cdot \sin\left[A_k\right] + .4 \cdot M \cdot \sin\left[A_k - \frac{\pi}{2}\right]$

$M_y := M \cdot \cos\left[A_k\right] + .4 \cdot M \cdot \cos\left[A_k - \frac{\pi}{2}\right]$

$V_z := V$

$n := 9$

$range_2 := range_1 + 10 \cdot deg$

Note: Do not calculate at exactly 180 degree

$i := 0 ..n$

$\theta_i := range_1 + \frac{range_2 - range_1}{n + 1} \cdot i$

$$x1_i := r_1 - r_2 \cdot \cos\left[\phi 1 - \theta_i\right]$$

$$x2_i := r_1 - r_2 \cdot \cos\left[\phi 2 - \theta_i\right]$$

$$x3_i := r_1 - r_2 \cdot \cos\left[\phi 3 - \theta_i\right]$$

$$k1_i := 1 \cdot \left[x1_i > 0\right] + kr \cdot \left[x1_i < 0\right]$$

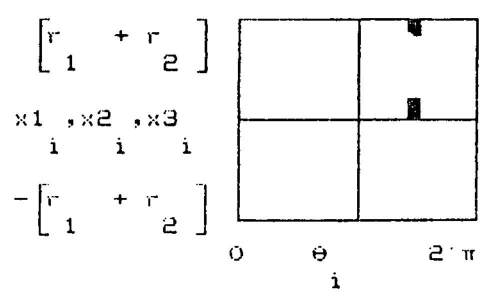
This step tests for the support being in tension or in compression and sets the relative stiffness values

$$k2_i := 1 \cdot \left[x2_i > 0\right] + kr \cdot \left[x2_i < 0\right]$$

$$k3_i := 1 \cdot \left[x3_i > 0\right] + kr \cdot \left[x3_i < 0\right]$$

$$x2_i := \left[\frac{k2_i}{k1_i}\right] \cdot x2_i$$

$$x3_i := \left[\frac{k3_i}{k1_i}\right] \cdot x3_i$$



$$\delta_{1_i} := \frac{x1_i + x2_i + x3_i}{x1_i}$$

$$\delta_{2_i} := \frac{x1_i \cdot \cos(\phi 1) + x2_i \cdot \cos(\phi 2) + x3_i \cdot \cos(\phi 3)}{x1_i}$$

$$\delta_{3_i} := \frac{x1_i \cdot \sin(\phi 1) + x2_i \cdot \sin(\phi 2) + x3_i \cdot \sin(\phi 3)}{x1_i}$$

$$a_i := r_2^2 \cdot {\delta_2}_i^2 + r_2^2 \cdot {\delta_3}_i^2 - r_1^2 \cdot {\delta_1}_i^2$$

$$b_i := -2 \cdot M_x \cdot r_2 \cdot {\delta_2}_i + 2 \cdot M_y \cdot r_2 \cdot {\delta_3}_i + 2 \cdot V_z \cdot r_1^2 \cdot {\delta_1}_i$$

$$c := M_X^2 + M_y^2 - r_1^2 \cdot V_z^2$$

$$P1_i := \left[\frac{1}{2 \cdot a_i}\right] \cdot \left[-b_i + \left[\left[b_i > 0\right] - \left[b_i < 0\right]\right] \sqrt{b_i^2 - 4 \cdot a_i \cdot c}\right]$$

$$P2_i := P1_i \cdot \left[\frac{x2_i}{x1_i}\right]$$

$$P3_i := P1_i \cdot \left[\frac{x3_i}{x1_i}\right]$$

$$C_i := P1_i \cdot {\delta_1}_i - V_z$$

$$\theta_{cal_i} := \operatorname{acos}\left[\frac{r_2 \cdot {\delta_2}_i \cdot P1_i - M_x}{r_1 \cdot \left[{\delta_1}_i \cdot P1_i - V_z\right]}\right]$$

$$\theta_{cal_i} := \theta_{cal_i} \cdot \left[\theta_i < \pi\right] + \left[2 \cdot \pi - \theta_{cal_i}\right] \cdot \left[\theta_i > \pi\right]$$

$$M_{xcal_i} := -C_i \cdot r_1 \cdot \cos\left[\theta_{cal_i}\right] + P1_i \cdot r_2 \cdot \delta_{2_i}$$

$$M_{ycal_i} := C_i \cdot r_1 \cdot \sin\left[\theta_{cal_i}\right] - P1_i \cdot r_2 \cdot \delta_{3_i}$$

$$V_{zcal_i} := P1_i + P2_i + P3_i - C_i$$

$$\theta_{del_i} := \theta_{cal_i} - \theta_i$$

θ_i	θ_{cal_i}	θ_{del_i}	M_{ycal_i}	M_{xcal_i}	V_{zcal_i}
4.538	4.653	0.115	−10	4	0
4.555	4.642	0.087	−10	4	0
4.573	4.631	0.058	−10	4	0
4.59	4.619	0.029	−10	4	0
4.608	4.608	$3.333 \cdot 10^{-4}$	−10	4	0
4.625	4.597	−0.029	−10	4	0
4.643	4.585	−0.058	−10	4	0
4.66	4.573	−0.087	−10	4	0
4.677	4.562	−0.116	−10	4	0
4.695	4.55	−0.145	−10	4	0

$$t_i := i$$

$$s_{cal} := \mathrm{cspline}\left[t, \theta_{cal}\right]$$

$$s := \mathrm{cspline}(t, \theta)$$

$x := 5$

$\mathrm{root}\left[\mathrm{interp}\left[s_{cal}, t, \theta_{cal}, x\right] - \mathrm{interp}(s,t,\theta,x), x\right] = 4.013$

$\theta_{opt_k} := \mathrm{interp}\left[s,t,\theta,\mathrm{root}\left[\mathrm{interp}\left[s_{cal}, t, \theta_{cal}, x\right] - \mathrm{interp}(s,t,\theta,x), x\right]\right]$

$y_{NOR_i} := \dfrac{P1_i}{x1_i} \cdot \cos\left[\theta_i\right]$ $\qquad$ $x_{NOR_i} := \dfrac{P1_i}{x1_i} \cdot \sin\left[\theta_i\right]$

$j := 1$

$A_k = 180 \cdot deg$ $\qquad$ $\dfrac{\theta_{opt_k}}{deg} = 264.013$

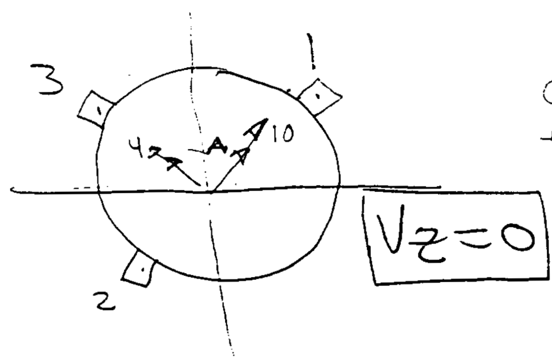
$\frac{P1_i}{j}$	$\frac{P2_i}{j}$	$\frac{P3_i}{j}$	$\frac{C_i}{j}$	$\frac{\theta_i}{deg}$	$\frac{x_{NOR_i}}{j}$	$\frac{y_{NOR_i}}{j}$
0.413	0.032	0.063	0.508	260	-0.016	-0.003
0.413	0.035	0.06	0.508	261	-0.016	-0.003
0.412	0.038	0.057	0.508	262	-0.017	-0.002
0.412	0.041	0.054	0.508	263	-0.017	-0.002
0.412	0.045	0.051	0.508	264	-0.017	-0.002
0.412	0.048	0.048	0.509	265	-0.017	-0.001
0.412	0.052	0.045	0.509	266	-0.017	-0.001
0.413	0.055	0.042	0.51	267	-0.017	$-8.965 \cdot 10^{-4}$
0.413	0.059	0.039	0.511	268	-0.017	$-6.012 \cdot 10^{-4}$
0.413	0.063	0.036	0.512	269	-0.017	$-3.025 \cdot 10^{-4}$

$k \equiv 8$

$range_1 \equiv 260 \cdot deg$

Results For Case

4 k ↑ → 10 k



Compression force: $\sqrt{10^2+4^2}$ → $\frac{10.77}{(14.25)(12)} = .410$

Max	Angle, A		P_1	P_2	P_3	C	θ	$\frac{P1}{X1}\sin\theta$	$\frac{P1}{X1}\cos\theta$
.302	0	0	0*	.302	.202	.504	64.7	.011	.006
.270	22.5	1	.007	.238	.270	.516	91.6	.012	0
.311	45	2	.059	.179	.311	.548	114.5°	.011	−.005
.325	67.5	3	.121	.115	.325	.560	135.7	.009	−.009
.309	90	4	.183	.054	.309	.546	156.9	.005	−.011
.266	112.5	5	.243	.004	.266	.513	180	0	−.012
.302	135	6	.302	0	.202	.504	207.0	−.006	−.011
.358	157.5	7	.358	0	.130	.489	235.6	−.011	−.008
.412	180.0	8	.412	.045	.051	.508	264.0	−.017	−.002
.441	202.5	9	.441	.155	0	.596	285.8	−.02	.006
.430	225	10	.430	.275	0	.705	302.3	−.021	.013
.375	247.5	11	.369	.375	0	.744	315.5	−.018	.018
.431	270.0	12	.269	.431	0	.700	328.6	−.013	.021
.441	292.5	13	.145	.441	0	.586	345.5	−.005	.020
.410	315	14	.038	.410	.056	.504	7.6	.002	.017
.355	337.5	15	0	.355	.132	.487	36.2	.008	.011

* this run did not allow the supports to go in compression – difference is insignificant

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Preliminary Capacity Analysis

$4^{k\text{-}''}$ ↑ → $10^{k\text{-}''}$

From Force analysis:

Median force / $10^{k\text{-}''}$ input $\cong 0.441^k$ (in bolt)

Median allowable angle cap $= 2.11^k$

⇒ Corresponding Moment at base in 1 direction $= (2.11 / .441) 10 = 47.84^{k\text{-}''}$

Force at C.g. $= \dfrac{47.84}{41.5} = 1.15^k$

$Sa = \dfrac{1.15}{.920} g = 1.25 g$

Increase for inelastic response

Allow 0.4 in @ Cg ($\approx 1\%$ drift)

Yield Force in support $\approx 2.11 \times \dfrac{1}{1.5} = 1.41^k$ (Force in bolt)

Moment at base of tank $= \dfrac{1.41}{.441} \times 10^{k\text{-}''} = 31.9^{k\text{-}''}$

See page 6

$F = 9.5\ hz$: Rotation at base $\Theta = \dfrac{31.9}{26,193} \cong 1.2 \times 10^{-3}$ rad.
min stiffness → 26,193

$F = 27\ hz$: Rotation at base $\Theta = \dfrac{31.9}{194,400} = 1.6 \times 10^{-4}$ rad
max stiffness → 194,400

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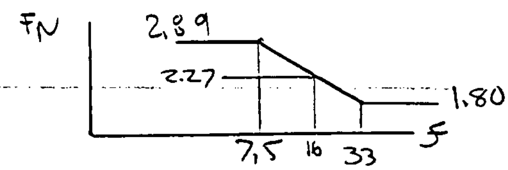
<u>ductility</u>: $F = 9.5\,hz$: $\mu = 0.4/(1.2\times10^{-3}\times 41) = 8.1$

$F = 27\,hz$: $\mu = 0.4/(1.6\times10^{-4}\times 41) = 61.0$

<u>Convert to 7.5 hz & 33 hz</u> <u>Riddell/Newmark</u>

@ 7.5 hz: $\mu = 8.1 \times \left(\frac{7.5}{9.5}\right)^2 = 5.0$ $F_\mu = [2.851(5.0) - 1.851]^{.422} = 2.89$

@ 33 hz: $\mu = 61 \times \left(\frac{33}{27}\right)^2 = 91.1$ $F_\mu = (91.1)^{.13} = 1.80$



@ Median: f = 16 hz:

$$\ln F_\mu = \ln 2.89 - \frac{\ln 16 - \ln 7.5}{\ln 33 - \ln 7.5}(\ln 2.89 - \ln 1.80)$$

$$F_\mu = 2.27$$

$\beta_r = 0.11\,[F_\mu - \tfrac{1}{2}] = 0.14$ Fits Riddell/Newmark Criteria

$\beta_u = 0.25$ (estimate)

<u>Note</u>: Since f = 16 hz is on stiff side of spectral peak, $F_\mu = 2.27$ is too optimistic. see effect on page 22A–C
JRB 11/13/88

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Reanalysis Performed After Completion of Calculation
(see pgs 23+ for rest of original calculation)

The better approach is to combine S_A & F_μ into single term S_A/F_μ. A time history was developed by fitting real earthquake record to 5% damped floor spectrum and a nonlinear analysis was performed to obtain F_μ values, i.e.

freq (hz)	μ*	F_μ
4.0	1.44	1.50
6.0	3.23	4.70
7.2	4.65	5.20
8.0	5.75	4.40
9.0	7.27	2.90
10.0	8.97	2.10
15.0	20.19	1.45
20.0	35.90	1.40
30.0	80.78	1.35

* μ to obtain 0.4 in displacement at C.G of tank

Pages 22B and 22C compute S_A/F_μ values and statistics are obtained for uncertainty in frequency [i.e. LN(16 hz, 0.35)]. ~~S_A/F_μ median is 0.35g and β_U = 0.07.~~ see pg 25

The $\check{S}_A/F_\mu$ implied for F_μ = 2.27 is .567/2.27 = 0.25g. Thus, the final median capacity (see p.34) should be 0.25/0.35 x 1.4g = 1.0g

However, the β's need to be modified:

1) β_U due to freq is 0.07 not 0.30

2) β_U due to inelastic response is estimated to be 0.15 not 0.25

3) β_R due to inelastic response is estimated to be:

F_μ = .25/.35 x 2.27 = 1.62

& β_R = .111[1.62 - ½] = .12

However, based on recent work the β_R does not combine by SRSS to β_R total. Total β_R is only slightly increased.

Based on these changes:

$\check{a}$ = 1.0g β_R = .24 β_U = 0.28

HCLPF = $1.0e^{-1.65(.24+.28)}$ = .42g which is slightly smaller than 0.43g from before (see p 34)

B

ORIGIN ≡ 0

$y \equiv 0$

$inorm(x) \equiv root(cnorm(y) - x, y)$ Inverse normal distribution

$Freq_{hat} := 16$

$\beta := 0.35$

$n := 199$

$i := 0 .. n$

$$Freq_i := Freq_{hat} \cdot exp\left[\beta \cdot inorm\left[\frac{i + rnd(1)}{n + 1}\right]\right]$$

$LFreq_i := ln\left[Freq_i\right]$

$Freq_0 = 5.176$

$Freq_{199} = 40.227$

$m := 45$

$j := 0 .. m$

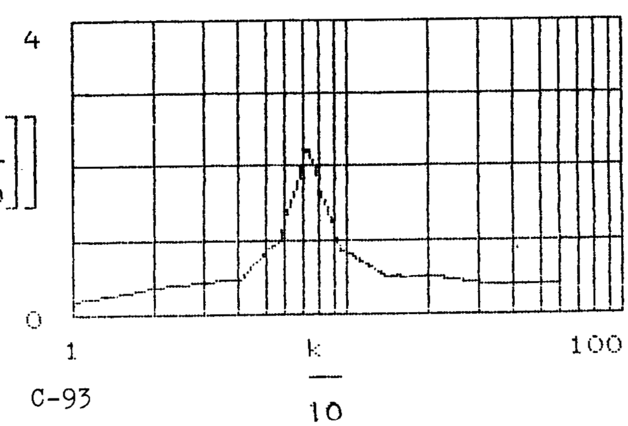
$XLFR_j := READ(HLFR)$

$XSA5_j := READ(HSA5)$

$s5 := cspline(XLFR, XSA5)$

$k := 1 .. 600$

$$interp\left[s5, XLFR, XSA5, ln\left[\frac{k}{10}\right]\right]$$

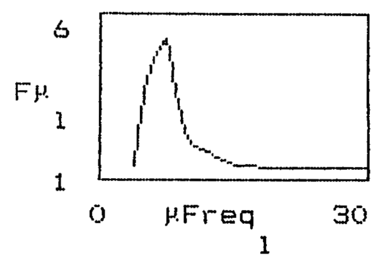


$$SA := \overrightarrow{(\mathrm{interp}(s5, XLFR, XSA5, LFreq))}$$

$$\mu Freq := \begin{bmatrix} 4.0 \\ 6.0 \\ 7.2 \\ 8.0 \\ 9.0 \\ 10.0 \\ 15.0 \\ 20.0 \\ 30.0 \end{bmatrix} \qquad F\mu := \begin{bmatrix} 1.50 \\ 4.70 \\ 5.20 \\ 4.40 \\ 2.90 \\ 2.10 \\ 1.45 \\ 1.40 \\ 1.35 \end{bmatrix}$$

$$SaeF\mu := \overrightarrow{\left[\frac{SA}{\mathrm{linterp}(\mu Freq, F\mu, Freq)}\right]}$$

$$l := 0 \,..8$$

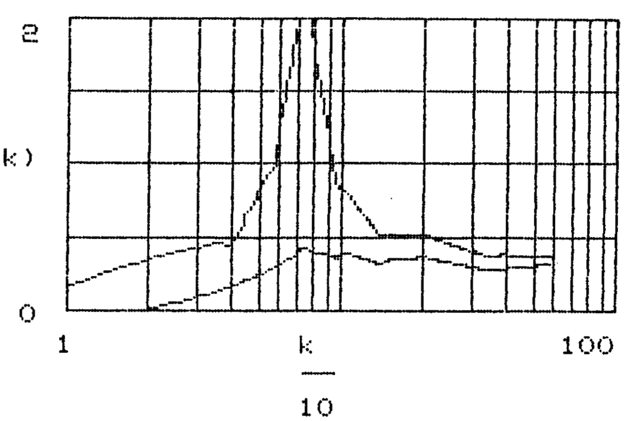


$$sp(k) := \mathrm{interp}\left[s5, XLFR, XSA5, \ln\left[\frac{k}{10}\right]\right]$$

$$s\mu(k) := \mathrm{linterp}\left[Freq, SaeF\mu, \frac{k}{10}\right]$$

The peaked curve is Sa

The flat curve is Sa/Fμ



$$LSA := \overrightarrow{\ln(SaeF\mu)}$$

$$Median := \exp(\mathrm{mean}(LSA)) \qquad Fmedian := \exp(\mathrm{mean}(LFreq))$$

$$Beta := \mathrm{stdev}(LSA) \qquad Fbeta := \mathrm{stdev}(LFreq)$$

$$Median = 0.35 \qquad Fmedian = 15.978$$

$$Beta = 0.073 \qquad Fbeta = 0.35$$

Sa capacity $= (2.27)(1.25g) = 2.84\,g$

Floor capacity $= \left(\frac{2.84}{.567}\right) .38g = 1.90\,g$ (← ZPA)

└ $\tilde{S}_a$ @ 5% damping for 0.18g pga norm (see pg 30)

Ground capacity $\left(\frac{2.84}{.567}\right)(.18g) = 0.90\,g$

Corresponding vertical acceleration @ floor

$$S_a = \frac{0.90g}{0.18} \times 0.20 = 1.00g$$

Net downward force using 100-40-40 rule is $920[1-(0.4)(1.00)] \cong \underline{552\ lbs}$

The corresponding Moment @ base is $47.84^{k\text{-}ft}$

Check of forces shows the median capacity goes up by a factor of $\cong 1.11$ (due to help from dead load)

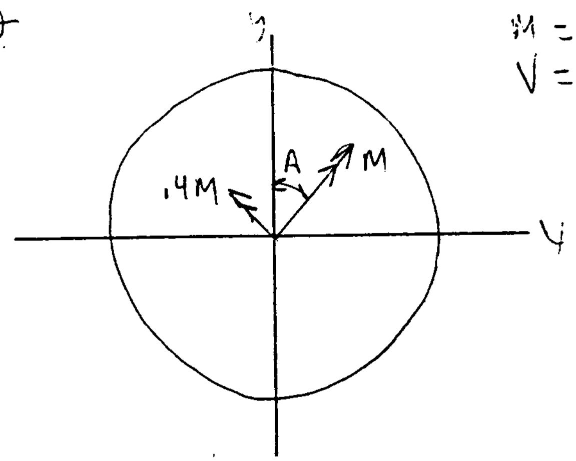
<u>Final Capacity Analysis</u>

Revised Net Downward Force
$920[[1-(0.4)(1.0)(1.11)] \cong -511\ lbs$ ← Use in final calc

Revised Moment $= (1.11)(47.84$
$= 53.10$ k-ft

Recompute Maximum Force in Anchor For Revised Force Set

$M = 53.0^{k\text{-}in}$
$V = .511^{k}$



From page 20 Angle 202.5° and 292.5° have maximum force in angle:

k	Angle A	Peak Force in bolt
9	202.5	2.10^k
13	292.5	2.09^k

$F_{MAX} = 2.10^k$

$$\text{Force at C.G} = \frac{53.0}{41.0''} \times \frac{2.11}{2.10} = 1.30^k$$

$$Sa = \frac{1.30}{.92} = 1.41g$$

↑ wt

pga capacity (w/o ductility factor)

$$pga = \frac{1.41\,g}{.567} \times .18g = 0.45g$$

↑ $\bar{S}a$ for 0.18g pga horiz (5% damping)
(see $\bar{S}a$ calc on page 30)

Check vertical floor acceleration

pga w/ ductility $= (.45)(2.27) = 1.01g$

$$Sav = \frac{1.01}{0.18} \times .20 = 1.12g \quad (\text{assumed } (1.11)(1.0) = 1.11g)$$

↑ Spa for 0.18g pga horiz

Hence assumed vertical load is ok

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Develop Fragility Parameters $\check{F}$, β_r, β_u & HCLPF

Capacity

Strength: $\check{F} = 0.45_g / 0.45_g = 1.0$ (see page 25)

$\beta_r = 0$

$\beta_u = 0.15$ (angle capacity – see page 2)

Inelastic Response:

$\check{F} = 2.27$ (see page 22)

$\beta_r = 0.14$

$\beta_u = 0.25$

Equipment Response

Spectral Shape (difference between smoothed spectra and linear spectra)

$\check{F} = 1.0$

$\beta_r = 0$

$\beta_u = 0.05$ (estimate)

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Damping

Median damping = 5% (this is basis for calculation)

$\tilde{F} = 1.0$

-1σ damping = 3.5%

freq	Sa(5%)	Sa(3.5%)	(0.18g hory pga)
16 hz	.512g	.525g	

$\beta_U = \frac{1}{1} \ln \frac{.525}{.512} = 0.03$

$\beta_R = 0$

Modeling

Frequency: Combine both equipment and structure freq variation

$\beta_U = 0.25$ building (est.)

$\beta_U = 0.25$ equipment

Combined $\beta_U = 0.35$

freq	Sa(5%)	(0.18g hory pga)
16 hz	0.512g	
$16e^{-.35} = 11.2$ hz	0.724	

$\beta_U = \frac{1}{1} \ln \frac{.724}{.512} = .35$ (too high!)

looking at floor response spectrum shape
this value is high
Do simulation (see SPECT, SIM following pages

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MCAD FILES SPECT.SIM

This program calculates distribution of Sa for random variation of frequency

NORMAL DISTRIBUTION

$b1 \equiv .319381530$

$b2 \equiv -.356563782$

$b3 \equiv 1.781477937$

$b4 \equiv -1.821255978$

$b5 \equiv 1.330274429$

$p \equiv .2316419$

$$Z(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION

$c0 \equiv 2.515517$

$c1 \equiv .802853$

$c2 \equiv .010328$

$d1 \equiv 1.432788$

$d2 \equiv .189269$

$d3 \equiv .001308$

$$t(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$INOR(p) \equiv (-1)^{p < .5} \cdot \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3}\right]$$

$Freq_{hat} := 16$

$\beta := 0.35$

$n := 199$

$i := 0 \,..\, n$

$$Freq_i := Freq_{hat} \cdot \exp\left[\beta \cdot INOR\left[\frac{i + rnd(1)}{n + 1}\right]\right]$$

$LFreq_i := \ln\left[Freq_i\right]$

$m := 45$

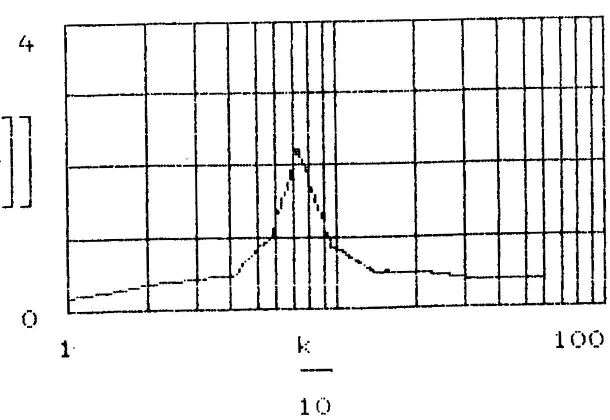
$j := 0 \,..\, m$

$XLFR_j := READ(XLFR)$

$XSA5_j := READ(XSA5)$

$s5 := cspline(XLFR, XSA5)$

$k := 1 \,..\, 600$



$SA_i := interp\left[s5, XLFR, XSA5, LFreq_i\right]$

$LSA_i := \ln\left[SA_i\right]$

```
Median := exp(mean(LSA))

Beta := stdev(LSA)

    Median = 0.567
```

← This is $\check{S}_a$

```
    Beta = 0.298
```

← This β_v

```
Fmedian := exp(mean(LFreq))

Fbeta := stdev(LFreq)

    Fmedian = 15.951

    Fbeta = 0.358
```

Modeling (Frequency) Continued

$\beta_U = 0.30$

$\beta_R = 0$

$\check{F} = 1.0$ (0.567g used in strength analysis – see pg 23)

Mode shape: simple equipment in high frequency domain

$\check{F} = 1.0 \quad \beta_R = \beta_U = 0$

Mode Combination simple structure in high frequency domain (also linear first mode shape with OTM at base failure mode)

$\check{F} = 1.0 \quad \beta_R = \beta_U = 0$

Horizontal Component Phasing

Bolt Force w/ 100-40 = 2.10 (median)
Bolt Force w/ 100-100 = 2.617 * (3β)

$$\beta_R = \frac{1}{3}\left|\ln \frac{2.617}{2.10}\right| = 0.07$$

$\beta_U = 0$

$\check{F} = 1.0$

*Reran ACCUM.FIN (pg 20)

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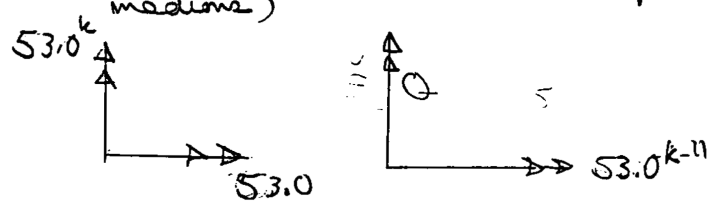
Structure Response

Ground Motion This parameter includes:

1) Peak-to-Peak Spectrum Variation
2) Horizontal Direction Variability
3) Equipment Horizontal Component Coupling

See Appendix A

C = ratio of minor horizontal direction response to combined response (i.e. ratio of medians)



$\check{P}_b = 2.617^k$ $\check{P}_b = 1.875^k$

$$C = \frac{2.617 - 1.875}{1.875} = 0.40$$

From DIR.FAT Run (see Appendix A)

$\tilde{F} = 1/1.02 \doteq 0.98$

$\beta_r = 0.29$

$\beta_u = 0$

Damping:

		S_A (Ground @ 7.2 Hz)
Median	: 7%	1.89g
-1σ	: 5%	2.12

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Structure Response (Damping) Continued

$\beta_U = \frac{1}{1} \ln \frac{2.12}{1.89} = 0.11$

$\beta_R = 0$

$\check{F} = 1.0$ (used 7% in strength calc)

Modeling

Frequency - Since building frequency in region of ground spectra plateau little change will occur (also see Equip. Modeling Frequency)

$\beta_R = \beta_U = 0 \quad \check{F} = 1.0$

Mode Shape - Assume controlled by first mode

$\check{F} = 1.0 \quad \beta_R = 0 \quad \beta_U = 0.10$ (estimate)

Mode Combination (controlled by first mode - estimate)

$\check{F} = 1.0 \quad \beta_R = \beta_U = 0$

Horizontal Component Phasing

$\check{F} = 1.0 \quad \beta_R = 0.10 \quad \beta_U = 0$ (estimate)

SSI (Rock site)

$\check{F} = 1.0 \quad \beta_R = 0 \quad \beta_U = 0.05$ (est)

Inelastic Response (structure assumed linear) $\check{F} = 1.0 \quad \beta_R = \beta_U = 0$

Parameter	$\check{F}$	β_r	β_u
Capacity			
Strength	1.0	0	0.15
Inelastic Response	2.27	0.14	0.25
Equipment Response			
Spectral Shape	1.0	0	0.05
Damping	1.0	0	0.03
Modeling - Frequency	1.0	0	0.30
- Mode Shape	1.0	0	0
Mode Combination	1.0	0	0
Horiz Component Phasing	1.0	0.07	0
Structure Response			
Ground Motion	0.98	0.19	0
Damping	1.0	0	0.11
Modeling - Frequency	1.0	0	0
- Mode Shape	1.0	0	0.10
Mode Combination	1.0	0	0
Horizontal Component Phasing	1.0	0.10	0
SSI	1.0	0	0.05
Inelastic Response	1.0	0	0
Combined	2.22	0.27	0.45

$\check{a}_{pga} = (2.22)(0.45) = 1.00g$ $\check{a}_{pga}$(Peak 84%) $= 1.0 \times 1.4 = 1.4g$

$HCLPF_m = (1.00)e^{-1.65(.27+.45)} = 0.30g$ (Median input)

Bordin → $HCLPF_{Peak\,84\%} = 1.4\ (0.30) = .0.43g$ (Peak Component 84% NEP)

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Compute HCLPF Directly $\sum x \cdot \beta = 1.65(.27+.45) = 1.19$
(See Appendix C)

Parameter	Conservatism	x, β	Deterministic Input
Strength	96% NEP	(1.8)(.15) = .27	Code yield 0.14 (-1.65σ) 1.4 plasticity 0.13 (-3.25σ) 0.27
Inelastic Response	-1.93 σ on both randomness & uncertainty ($\gamma = .56/.29 = 1.93$)	$\ln \frac{2.27}{1.3} = 0.56$	$F_\mu = 1.3$
Equip Frequency	-0.5 σ ($\gamma = .16/.30 = .5$)	0.16	$S_a = .567 e^{.16} \left(\frac{.30}{.18}\right) = 1.11g$ (Assume pga capacity)
Ground Motion	+1.0 σ	0.20	$e^{.20} = 1.22$
		$\sum = 1.19$	

Keep other parameters at median level

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Develop input to tank model assuming 0.30 HCLPF

at 0.5 T frequency (↓ median) $Sa = 0.567\, e^{+.16} \left(\frac{.30}{.18}\right) = 1.11g$

increase for +1.0σ spread spectrum

$Sa = (1.22)(1.11) = 1.35g$

Moment at base of tank, M

$M = \frac{1.35}{1.41} \times \frac{53^{k\text{-}ft}}{1.3} = \underline{39.1^{k\text{-}ft}}$

(1.3 ← ductility factor)

Vertical force in tank, V

$ZPA \text{ for vert} = \frac{.30}{.18} \times .20 \times 1.22 = 0.41g$

$V = .920[1 - 0.40(.41)] = \underline{.770^k}$

Develop Capacity of tank, P_b [illegible]

Angle Cap, $M_{cap} = 1.32\, S\sigma$ (↓ plastic factor (−3.25σ))
@ Tank

$= (1.32)(.03125)(36) = 1.49^{k\text{-}ft}$ (↓ code yield (−1.65σ))

Angle Cap at bolt $M_{cap} = 1.49 \times \frac{3 - 3/4}{3} = 1.12^{k\text{-}ft}$

$T = (1.49 + 1.12)/1.75 = \underline{1.31^k}$

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Analysis for Tank $M = 39.1^{k-ft}$, $V = -.770^{k}$ $\Rightarrow P_{b_{MAX}} = 1.35^{k} > 1.31$

Median level HCLPF pga $\cong \frac{1.31}{1.35} \times .30 = \underline{0.29g}$

84% MAX Component pga $= (0.29g)(1.4) = \underline{0.41g}$

↑ See Appendix B

BY SDR	DATE 10/2/87	PROJECT	PAGE 1 OF
CHKD. BY JR	DATE 1/3/88	SUBJECT	JOB NO. 105-170

BLOCK WALL

Assumption (special)

1. No equipment attached to wall. Max displacement at top allowed = 6"
2. Bending at base is failure mode

Approach	HCLPF Median Prob Ave Horiz Dir	HCLPF 84% NEP Max Horiz Dir	Median 84% NEP Max Horiz D
Fragility Analysis	0.27g	0.38g	1.41g
CDFM (backfit)	0.27	0.38	—
Revised Fragility*	0.36	0.52	1.96

* after 11/12-13/87 Meeting

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Calculate Strength of wall

Median Capacity (bending at base is failure mode)

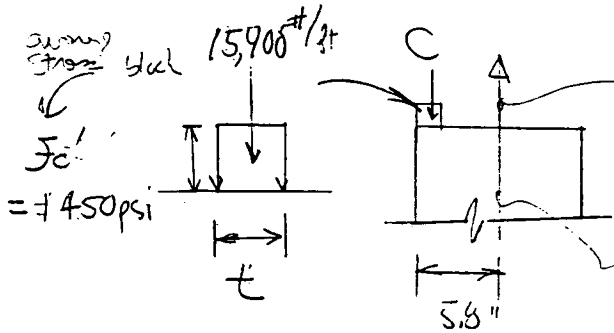
Use ultimate strength approach:

$$f_s = (1.1)(60,000) = 66,000 \text{ psi}$$

↑ code to median

compare to 28 day compressive stress of 1800psi for mortar (type S)

$$f_c' = .85 f_m' = .85(1700\text{psi}) = 1450 \text{ psi}$$



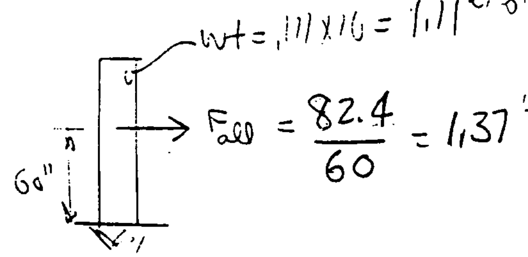
T/12" = .233 X 66,000 = 15,400 #/ft

↑ A_s/ft (#5@16")

$$t = \frac{15,400}{(12)(1430)} = 0.90'' < 1\tfrac{1}{2}'' \text{ hence within outside shell of block}$$

$$M_{cap} = 15.4^{k}(5.8 - 0.90/2) = 82.4^{k\text{-}''}/ft$$

Find Sa allowable



$$F_{all} = \frac{82.4}{60} = 1.37^{k} \qquad S_a = \frac{1.37}{1.11} = 1.24g$$

displacement at C.G is ≈ $\frac{1.5}{2}\frac{S_a}{\omega^2}$

Variability

1. Placement of reinforcement dowels at bottom of wall (assume ±1" is ±2σ - special inspection)

For 1" offset:

$$S_a = \frac{5.8 - 1 - 0.90/2}{5.8 - 0.90/2} \times 1.24 = 1.01 g$$

(ratio of lever arms)

$$\beta_U = \frac{1}{2} \ln \frac{1.24}{1.01} = \underline{0.10}$$

2. For ultimate compressive strength (assume factor of ½ is −2σ, special inspection)

$$t = (0.90) \times 2 = 1.80 \quad \text{(into grouted area revise } t\text{)}$$

$$t = \frac{(1.80)(16) - (1.5)(16)}{8} + 1.5 = 2.10"$$

$$S_a = \frac{5.8 - 2.10/2}{5.8 - 0.90/2} \times 1.24 = 1.10 g$$

$$\beta_U = \frac{1}{2} \ln \frac{1.24}{1.10} = \underline{0.06}$$

3. For reinforcement yield stress (assume factor of 1.0 is −1σ)

$$\beta_U = \ln \frac{1.1}{1.0} = \underline{0.10}$$

Combined β_U

$$\beta_U = \left(0.10^2 + 0.06^2 + 0.10^2 \right)^{1/2} = \underline{0.15}$$

Ductility

Allow about 3" drift to C.g (≈ 1.7% drift)
(corresponds to 6" at top)

yield: $\Delta_{C.g} = 1.5/2 \frac{S_a}{\omega^2} = \frac{0.75}{(2\pi)^2} \frac{S_a}{f^2} = \frac{0.75(386.4)}{(2\pi)^2} \frac{S_a/g}{f^2}$ (g units)

$= \frac{7.34}{f^2}\left(\frac{S_a}{g}\right) = \frac{7.34}{f^2}(1.24) = \frac{9.10}{f^2}$ (↑ approx yield S_a)

median freq $\cong \sqrt{(3)(12)} = 6\,hz$ (lower bound ↑, ↑ upper bound) (use $\beta_f = 0.25$ see pg 11)

$S_a(\text{yield}) \approx 1.24g$

Freq	yield $\Delta_{C.g}$	μ_{eff}	F_μ
3hz	1.0 IN	3	2.10
6	0.25	12	4.05
12	0.06	48	7.28

Use Riddell/Newmark for F_μ → upper limit of accel region

use 7% damping*

$F_\mu = [(q+1)\mu_{eff} - q]^r$ (acceleration region)

$q = 3.0(7)^{-.3} = 1.67$

$r = 0.48(7)^{-.08} = .41$

$F_\mu = [(2.67)\mu_{eff} - 1.67]^{.41}$ (↑ failure disp)

$\mu_{eff} = 3/\left(\frac{9.10}{f^2}\right) = 0.330 f^2$

by simulation F_μ (see pages 5-10 for DUCTR.SIM)

$\check{F} = \underline{3.92}$ $\beta_u = 0.24$

Variability

$\beta_u = \underline{0.24}$

$\beta_r = 0.11\left[F_\mu - \frac{1}{2}\right] = \underline{0.37}$ ← too big! (use 0.22 (Fits Riddell/Newmark data)

* 10% is better for this failure mode but 7% is max spectrum available

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S

MCADFILESDUCTR.SIM

This program calculates the distribution on ductility factor for variability in frequency of equipment

NORMAL DISTRIBUTION

$b1 \equiv .319381530$

$b2 \equiv -.356563782$

$b3 \equiv 1.781477937$

$b4 \equiv -1.821255978$

$b5 \equiv 1.330274429$

$p \equiv .2316419$

$$Z(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION

$c0 \equiv 2.515517$

$c1 \equiv .802853$

$c2 \equiv .010328$

$d1 \equiv 1.432788$

$d2 \equiv .189269$

$d3 \equiv .001308$

$$t(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

6

$$INOR(p) \equiv (-1)^{p < .5}\left[t(p) - \frac{c0 + c1\cdot t(p) + c2\cdot t(p)^2}{1 + d1\cdot t(p) + d2\cdot t(p)^2 + d3\cdot t(p)^3}\right]$$

$n := 199$

$i := 0 \,..n$

$$dir_i := 6\cdot exp\left[0.25\cdot INOR\left[\frac{i + rnd(1)}{n + 1}\right]\right]$$

Latin hypercube simulation of equipment frequency

$min(dir) = 2.014$

$max(dir) = 11.682$

$$\mu_i := .330\cdot dir_i^2$$

Calculate ductility from frequency

$$\mu_i := \mu_i\left[\mu_i > 1\right] + 1\cdot\left[\mu_i < 1\right]$$

Ductility is constrained to be > 1

$$dir_i := \left[2.67\cdot\mu_i - 1.67\right]^{.41}$$

Calculate ductility factor

$min(\mu) = 1.338$

$max(\mu) = 45.033$

$min(dir) = 1.302$

$max(dir) = 7.085$

7

$m := 10$

$j := 0 .. m$

$spc_j := (\min(dir) - .0001) + \frac{j}{m} \cdot (\max(dir) - \min(dir) + .0002)$

$freq := hist(spc, dir)$

$k := 0 .. (m - 1)$

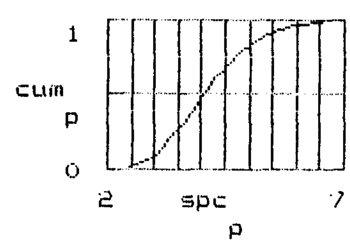
$\sum_k freq_k = 200$

$cum_0 := 0$

$p := 1 .. m$

$cum_p := cum_{p-1} + freq_{p-1}$

$cum_p := \frac{cum_p}{n + 1}$



$s := lspline(spc, cum)$

Find the median value:

$x := 4.0$

$root(interp(s, spc, cum, x) - .5, x) = 4.034$

Find the + 1σ value:

$root(interp(s, spc, cum, x) - .84134, x) = 5$

j	cum_j	spc_j
0	0	1.302
1	0.005	1.88
2	0.015	2.458
3	0.1	3.037
4	0.31	3.615
5	0.57	4.193
6	0.78	4.772
7	0.91	5.35
8	0.965	5.928
9	0.99	6.507
10	1	7.085

Find the − 1σ value:

root(interp(s,spc,cum,x) − .15866,x) = 3.232

$tcum_j := cum_j$

$tspc_j := spc_j$

$m := m - 2$

$s := 1$

$j := 0 .. m$

$cum_j := tcum_{j+1}$

$spc_j := spc_{j+1}$

j	cum_j	spc_j
0	0.005	1.88
1	0.015	2.458
2	0.1	3.037
3	0.31	3.615
4	0.57	4.193
5	0.78	4.772
6	0.91	5.35
7	0.965	5.928
8	0.99	6.507

This program take the calculated probabilities of failure and the associated peak ground acceleration values and performs a least squares fit in the log-probability domain to obtain the "best fit" median and ß value.

$x1_j := cum_j$

$y1_j := spc_j$

$y1_j := \ln\left[y1_j\right]$

$x1_j := INOR\left[x1_j\right]$

$slope(x1,y1) = 0.237$ $\qquad \beta := slope(x1,y1)$

$intercept(x1,y1) = 1.364$ $\qquad a_{med} := \exp(intercept(x1,y1))$

$corr(x1,y1) = 0.991$

$$a_{med} = 3.912$$
$$\beta = 0.237$$

$$z1_j := intercept(x1,y1) + slope(x1,y1) \cdot x1_j$$

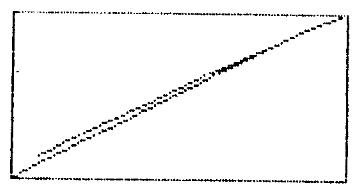
$max(y1) = 1.873$

$min(y1) = 0.631$

$min(z1) = 0.752$

$max(x1) = 2.327$

$min(x1) = -2.576$



j	$x1_j$	$y1_j$	$z1_j$
0	-2.576	0.631	0.752
1	-2.171	0.9	0.849
2	-1.282	1.111	1.06
3	-0.495	1.285	1.246
4	0.176	1.434	1.406
5	0.772	1.563	1.547
6	1.341	1.677	1.682
7	1.812	1.78	1.794
8	2.327	1.873	1.916

Tabulate probabilities to compare calculated and best fit values:

$a_j := \exp\left[y1_j\right]$ Peak ground accelerations

$p_j := NOR\left[x1_j\right]$ Probability of failure values

$$p_{best_j} := NOR\left[\frac{\ln\left[\frac{a_j}{a_{med}}\right]}{\beta}\right]$$

Best fit probability of failure values

j	a_j	p_j	p_{best_j}
0	1.88	0.005	0.001
1	2.458	0.015	0.025
2	3.037	0.1	0.143
3	3.615	0.31	0.37
4	4.193	0.57	0.615
5	4.772	0.78	0.799
6	5.35	0.91	0.906
7	5.928	0.965	0.96
8	6.507	0.99	0.984

Find Peak Ground Acceleration Capacity

- frequency of wall assumed to range 3 to 12 hz
(assume this is ± 3σ since wide range

$$\beta_U = \frac{1}{6} \ln \frac{12}{3} = 0.23 \quad \text{Use } 0.25$$

freq of building is 7.2 hz - assume $\beta_U = 0.25$ also

combined freq variation is $\sqrt{2}(.25) = 0.35$

Simulate randomly the frequency variation to obtain median Sa for 7% damping
(see SPECT.SIM on next 3 pages)

$\check{S}a = .855g$ ← corresponds to 0.18g pga input

$\beta_U = 0.30$

(Note if had used Sa value at median freq of 6 hz $\check{S}a$ would be 1.1g - error factor of 1.29)

← see pg 2

$$\check{pga} = \frac{1.24g}{0.855g} \times 0.18 = \underline{0.26g}$$

MCAD FILES SPECT.SIM

12

NORMAL DISTRIBUTION

$$b1 \equiv .319381530$$

$$b2 \equiv -.356563782$$

$$b3 \equiv 1.781477937$$

$$b4 \equiv -1.821255978$$

$$b5 \equiv 1.330274429$$

$$p \equiv .2316419$$

$$Z(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION

$$c0 \equiv 2.515517$$

$$c1 \equiv .802853$$

$$c2 \equiv .010328$$

$$d1 \equiv 1.432788$$

$$d2 \equiv .189269$$

$$d3 \equiv .001308$$

$$t(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$INOR(p) \equiv (-1)^{p < .5} \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3}\right]$$

$Freq_{hat} := 6$

$\beta := 0.35$

$n := 199$

$i := 0 .. n$

$$Freq_i := Freq_{hat} \cdot \exp\left[\beta \cdot INOR\left[\frac{i + rnd(1)}{n + 1}\right]\right]$$

$LFreq_i := \ln\left[Freq_i\right]$

$m := 45$

$j := 0 .. m$

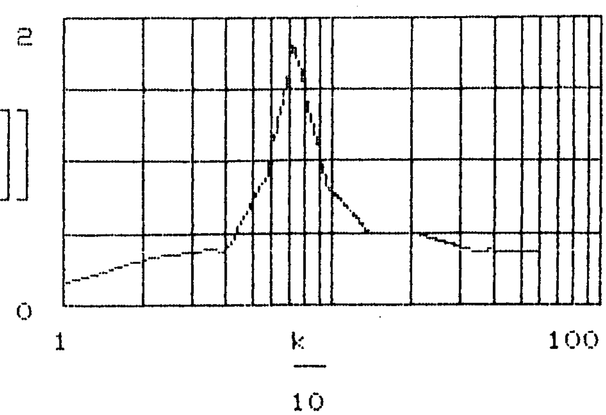
$XLFR_j := READ(XLFR)$

$XSA7_j := READ(XSA7)$

$s7 := cspline(XLFR, XSA7)$

$k := 1 .. 600$

$interp\left[s7, XLFR, XSA7, \ln\left[\frac{k}{10}\right]\right]$



$SA_i := interp\left[s7, XLFR, XSA7, LFreq_i\right]$

$LSA_i := \ln\left[SA_i\right]$

Median := exp(mean(LSA))

Beta := stdev(LSA)

Median = 0.855

Beta = 0.298

Fmedian := exp(mean(LFreq))

Fbeta := stdev(LFreq)

Fmedian = 5.998

Fbeta = 0.358

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Develop Fragility Parameters, $\check{F}$, β_r, β_u & HCLPF

Capacity

Strength

$\check{F} = 0.26/0.26 = 1.0$ (see page 11)

$\beta_r = 0$

$\beta_u = 0.15$ (see page 3)

Inelastic Response: (see page 4)

$\check{F} = 3.9$

$\beta_r = 0.22$

$\beta_u = 0.24$

Equipment Response

Spectral Shape (difference between smooth spectra to raw spectra

$\check{F} = 1.0$

$\beta_r = 0$

$\beta_u = 0.05$ (estimate)

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Damping

Median damping $= 7\%$ (used in calculation — note: 10% would have been better)

$\check{F} = 1.0$

-1σ damping $= 5\%$

$7\%\ \check{S}_a = 0.855\,g$ (see page 11)

$5\%\ \check{S}_a = 0.984\,g$ (run on SPECT.SIM)

$\beta_U = \frac{1}{1} \ln \frac{.984}{.855} = 0.14$

$\beta_R = 0$

note: if used spectral ordinate at median freq $= 6\,hz$: $\beta = \ln \frac{1.42}{1.1} = 0.26$ (Big difference!)

Modeling

frequency: combined both equip β = & structure β = in strength calc. (see page 11)

$\check{F} = 1.0$

$\beta_R = 0$

$\beta_U = 0.30$ (see page 11)

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Mode shape: simple equipment with linear mode shape – 1st mode controls since failure mode is OTM at base (all higher modes contribute nothing)

$\check{F} = 1.0 \quad \beta_r = \beta_u = 0$

Mode Combination (see mode shape discussion above)

$\check{F} = 1.0 \quad \beta_r = \beta_u = 0$

Horizontal Component Phasing

Controlled by 1 direction only. No contribution from other direction

$\check{F} = 1.0 \quad \beta_r = \beta_u = 0$

Structure Response

Ground Motion This parameter includes:

1) Peak-to-Peak Spectrum Variation
2) Horizontal Direction Variability
3) Equipment Horizontal Component Coupling

See Appendix A

C = ratio of minor horizontal direction response to combined response (ie ratio of medians)

C = 0 (only one direction controls)

From DIR.FAT: $\check{F} = 1.0$ $\beta_r = 0.24$

This can be obtained directly: $\beta_r = (0.12^2 + 0.20^2)^{1/2} = .23$

use $\beta_r = 0.24$ ← Appendix B

<u>Damping</u>: Median: 7% — Sa (ground at 7.2 hz) 1.89

-1σ 5% — 2.12

$\beta_u = \frac{1}{1} \ln \frac{2.12}{1.89} = 0.11$

$\beta_r = 0$

$\check{F} = 1.0$ (used 7% in strength calc)

<u>Modeling</u>:

frequency - since building frequency is in region of ground spectrum plateau little change will occur (also see Equip. Modeling)

$\check{F} = 1.0$

$\beta_u = \beta_r = 0$

Mode shape - Assume controlled by first mode

$\check{F} = 1.0 \quad \beta_r = 0 \quad \beta_u = 0.10$ (estimate)

Mode Combination (Controlled by first mode - estimate)

$\check{F} = 1.0 \quad \beta_r = \beta_u = 0$

Horizontal Component Phasing

$\check{F} = 1.0 \quad \beta_r = 0.10 \quad \beta_u = 0$ (estimate)

SSI (Rock site)

$\check{F} = 1.0 \quad \beta_r = 0 \quad \beta_u = 0.05$ (estimate)

Inelastic Response (structure assumed to be linear)

$\check{F} = 1.0 \quad \beta_r = \beta_u = 0$

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Parameter	$\check{F}$	β_r	β_u
Capacity			
Strength	1.0	0	0.15
Inelastic Response	3.9	0.22	0.24
Equipment Response			
Spectral Shape	1.0	0	0.05
Damping	1.0	0	0.14
Modeling – Frequency	1.0	0	0.30
– Mode Shape	1.0	0	0
Mode Combination	1.0	0	0
Horiz. Component Phasing	1.0	0	0
Structure Response			
Ground Motion	1.0	0.24	0
Damping	1.0	0	0.11
Modeling – Frequency	1.0	0	0
Mode Shape	1.0	0	0.10
Mode Combination	1.0	0	0
Horiz. Component Phasing	1.0	0.10	0
SSI	1.0	0	0.05
Inelastic Response	1.0	0	0
Combined:	3.9	0.34	0.47

$\check{A}_{pga} = (3.9)(0.26)g = 1.01g$ $\quad a_{psi}(\text{peak } 84\%) = 1.4(1.01) = 1.41g$

$HCLPF_m \ (1.01)\ e^{-1.65(.34+.47)} = 0.27$

$HCLPF_{\text{Peak } 84\%}\ 1.4\ (.27) = 0.38$ (See Appendix B)

Compute HCLPF Directly (See Appendix C)

$\sum x\beta = 1.65(.35+.47) = 1.35$

Parameter	Conservatism	$x\cdot\beta$	Deterministic Input
Capacity	95% NEP	$(1.65)(.15) = .25$	$Sa_{cap} = 1.24 e^{-1.65(.15)} = .97_g$
Inelastic Response	-1.8σ $1.8[.22^2+.24^2]^{1/2}$	0.59	$F_\mu = 3.9 e^{-.59} = 2.16$
Equip Freq –	-1σ	0.30	$Sa = .855 e^{.30}\left(\frac{HCLPF}{.18}\right) = 6.41\ HCLPF$
Ground Motion	$+1\sigma$	0.20	$e^{.20} = 1.22$
		$\Sigma = 1.34$	

Keep other parameters at median level

input: $(1.22)(6.41\ HCLPF) = 7.82\ HCLPF$

Capacity $(0.97_5)(2.16) = 2.10\ g$

at median level: $HCLPF = 2.10/7.82 = \underline{0.27\ g}$

at 84% Max Component HCLPF $= 0.27 \times 1.4 = \underline{0.38\ g}$

↑ see Appendix B

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Revised Analysis – Define failure (when displacement at Top = 12". After this displacement wall likely to collapse)

Revise page 4 of Calc

← disp at C.G. of wall (failure)

$$N_{eff} = 6'' / \left(\frac{9.10}{f^2}\right)$$

↑ yield displacement at C.G. of wall as function of equip freq

Freq	Yield $A_{c.g.}$	N_{eff}	F_μ
3 hz	1.0	5.9	3.0
6 hz	0.25	23.7	5.42
12 hz	0.06	94.9	9.64

By simulation (using DUCTR.SIM – similar to pages 5-10)

$$\check{F} = 5.4 \qquad \beta_U = 0.21 \ (\text{use } 0.24 \text{ as before})$$

Revised Fragility Parameters (see pg 20)

$$\check{a}_{pga} = (5.4)(.26) = 1.40\,g \qquad \beta_r = 0.34 \qquad \beta_u = 0.47$$

$$HCLPF_m = (1.40)e^{-1.65(.34+.47)} = 0.37\,g$$

$$HCLPF_{Pol\ 84\%} = 1.4(.37) = 0.52\,g$$

$$\check{a}_{pga}\ (Pol\ 84\%) = (1.4)(1.40) = 1.96\,g$$

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CHKD. BY NR DATE 12/17/87 SUBJECT ______ JOB NO. 105-170

COMPONENT COOLING HEAT EXCHANGER

Assumptions (special)

1. No nozzle loads considered in analysis (not given). Normally nozzle loads would be present.

2. Assume bolts are weakest link. There may be a weaker link present (no time to check).

   Bending of base plate at bottom of support looks like next candidate to check.

3. Ignore flexibility of support frame (normally this effect would be included)

4. Failure of bolts is assumed to be in the plane of the threads

Approach	HCLPF Median Prob Ave Horiz Dir	HCLPF 84% NEP Max Horiz Dir	Median 84% NEP Max Horiz Dir
Fragility Anal.	0.28 g	0.39 g	1.0 g
CDFM (backfit)	0.26	0.36	—

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Strength

Freq in long dir = 33hz (It is likely much less because of steel frame support — Normally this would be investigated).

Freq in transverse direction – assumed also equal to 33 hz

Consider $\beta_U = 0.30$ for equip freq variability
$\beta_U = 0.25$ for building freq variability

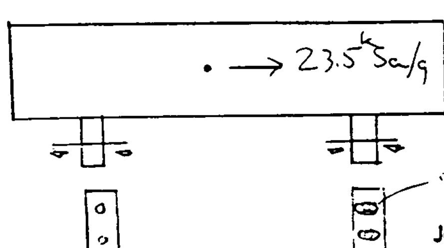
Combined $\beta_U = 0.39$

For 5% damping the median: $Sa = 0.42g$
(see pages 3-5 for SPECT.SIM analysis) $\beta = 0.14$

This corresponds to a 0.18g avg pga

Initially estimate bolt capacity with no tension in bolts (then check assumption)

Wt = 23.5^k



7/8" ϕ A-307 bolts

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MCAD FILES SPECT.SIM 3

NORMAL DISTRIBUTION

$b1 \equiv .319381530$

$b2 \equiv -.356563782$

$b3 \equiv 1.781477937$

$b4 \equiv -1.821255978$

$b5 \equiv 1.330274429$

$p \equiv .2316419$

$$Z(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION

$c0 \equiv 2.515517$

$c1 \equiv .802853$

$c2 \equiv .010328$

$d1 \equiv 1.432788$

$d2 \equiv .189269$

$d3 \equiv .001308$

$$t(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$INOR(p) \equiv (-1)^{p < .5} \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3}\right]$$

$Freq_{hat} := 33$

$\beta := 0.39$

$n := 199$

$i := 0 .. n$

$Freq_i := Freq_{hat} \cdot \exp\left[\beta \cdot INOR\left[\frac{i + rnd(1)}{n + 1}\right]\right]$

$LFreq_i := \ln\left[Freq_i\right]$

$m := 45$

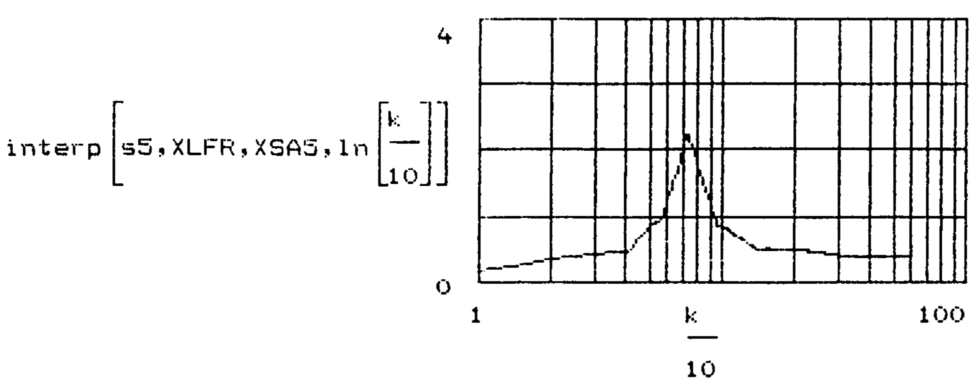
$j := 0 .. m$

$XLFR_j := READ(XLFR)$

$XSA5_j := READ(XSA5)$

$s5 := cspline(XLFR, XSA5)$

$k := 1 .. 600$



$SA_i := interp\left[s5, XLFR, XSA5, LFreq_i\right]$

$LSA_i := \ln\left[SA_i\right]$

5

Median := exp(mean(LSA))

Beta := stdev(LSA)

Median = 0.416

Beta = 0.137

Fmedian := exp(mean(LFreq))

Fbeta := stdev(LFreq)

Fmedian = 32.888

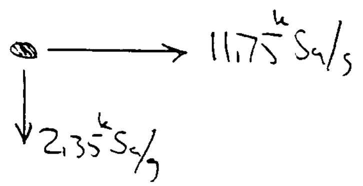
Fbeta = 0.399

Controlling direction is longitudinal (check assumption later)

Shear force on back bolt:

Longitudinal component $\frac{23.5^k \, Sa/g}{2} = 11.75^k \, Sa/g$

Transverse component $\frac{23.5^k}{4} \times 0.4 \, Sa/g = 2.35^k \, Sa/g$



Vector $= \sqrt{11.75^2 + 2.35^2} = \underline{11.98^k \, Sa/g}$

7/8 ϕ A-307 bolt ultimate capacity

Note: include factor to failure in inelastic response parameter

$\check{\tau}_v = (60)(1.2)(0.60) = 43.2$ ksi

(σ_u, Median/code, τ_u/σ_u)

$\check{F}_v = A \tau_v$

↑ use area across thread. Note that AISC code values have been modified for gross section area, but really based on net area

(e.g. $\frac{.6013}{.4617} \times 10 \times 1.67 = 21.75$ ksi)

(↑ AISC, 4-6; FS; ↑ .6 × 36 ksi)

$\check{F}_v = (.4617)(43.2) = 19.95^K$

Spectral Acceleration, Sa_H Capacity

$Sa_H = \frac{19.95}{11.98} = 1.67g$

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Check tension in bolts

$$\text{pga capacity} = \frac{1.67}{0.42} \times .18_g = 0.72_g$$

$$\check{S}a_v = \frac{0.72_g}{0.18_g} \times \check{S}a_v(a_g = 0.18)$$

↑ Run SPECTSIM for vertical direction

5% damping

freq = 33hz

$\beta_v = 0.39$

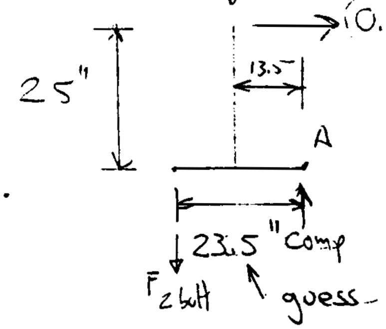
$\check{S}a_v(a = 0.18) = 0.26_g \quad \beta = 0.40$

$$\check{S}a_v = \frac{0.72}{0.18} \times 0.26_g = 1.04_g$$

Moment in transverse direction

$23.5^k[1-(0.4)(1.04)] = 13.72^k$

$(0.4)(1.67)(23.5)^k = 15.70$



$\Sigma M_A = 0$

$23.5 F_{2bolt} = (15.70)(25) - 13.72(13.5)$

$F_{2bolt} = 8.82^k$ tension for two bolt

$F_{1bolt} = 4.41^k$ tension for one bolt

Need to check for shear/tension interaction

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Bolt Stress due to
Moment in Longitudinal direction (compression by inspection)
Recompute capacity considering tension in bolt

$$\left(\frac{f_t}{F_T}\right)^2 + \left(\frac{f_v}{F_v}\right)^2 = 1$$

Assume median strength interaction eqn.

$$F_T = (1.2)(60)(.4617) = 33.24^k$$

$$f_T = \frac{0.4\, Sa_H (23.5)(25) - 23.5\left[1 - 0.4\left(\frac{1.07}{1.72}\right) Sa_H\right] 13.5}{23.5 \times 2}$$

$$f_T = 5.00 - 6.75(1 - .249\, Sa_H)$$

$$f_T = 6.68\, Sa_H - 6.75$$

$$\left(\frac{6.68\, Sa_H - 6.75}{33.24}\right)^2 + \left(\frac{11.98\, Sa}{19.95}\right)^2 = 1$$

$$0.0404\, Sa_H^2 - .0816\, Sa_H + .0412 + .4010\, Sa^2 - 1 = 0$$

$$379\, Sa_H^2 - .0816\, Sa_H - .9588$$

$$Sa_H = \frac{.0816 \pm \sqrt{(.0816)^2 + (4)(.4010)(.9588)}}{(2)(.4010)}$$

$$Sa_H = 1.65g \quad (\approx 1.67 \text{ slight interaction})$$

$$Sa_V = \frac{1.65}{1.67} \times 1.04g = 1.03g$$

$$pga = \frac{1.65}{1.67} \times .72g = 0.71g$$

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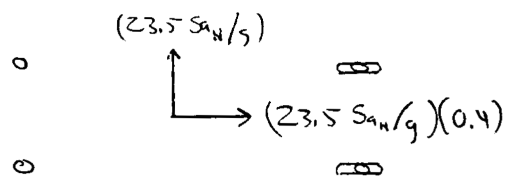
Check possibility that 100% Horiz in transverse direction controls



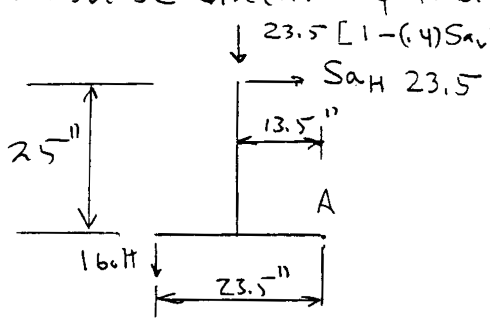
Shear on left bolt:

$$\frac{23.5}{2}(.4) = 4.7^k\, Sa/g$$

$$\frac{23.5}{4} = 5.88^k\, Sa/g$$

$$\text{Vector} = \sqrt{(4.7)^2 + 5.88)^2} = 7.53^k\, Sa/g$$

Moment in transverse direction & tension on 1 bolt:



$$F_T = \frac{Sa_H(23.5)(25) - 23.5\left[1 - 0.4\left(\frac{1.07}{1.72} Sa_H\right]13.5\right.}{23.5 \times 2}$$

$$F_T = 12.5\, Sa_H - 6.75(1 - .249\, Sa_H)$$

$$F_T = 14.18 - 6.75$$

$$\left(\frac{14.18\, Sa_H - 6.75}{33.24}\right)^2 + \left(\frac{7.53\, Sa_H}{19.95}\right)^2 = 1.0$$

try $Sa_H = 1.65g$ $\quad \Sigma = .64 < 1.0$

hence capacity for this loading exceeds 1.70 - other direction controls

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Check case for earthquake component phasing where both horizontal vectors are 100% in phase

(to develop β_r)

$$\frac{23.5}{2} = 11.75^k\ Sa/g$$

$$\frac{23.5}{4} = 5.875^k\ Sa/g \qquad \text{Vector} = 13.14^k\ Sa/g$$

$$F_T = \frac{Sa_H(23.5)(25) - 23.5\left[1-(0.4)\left(\frac{1.07}{1.72}Sa_H\right)\right]13.5}{23.5 \cdot \frac{2}{2} \cdot 2\ \text{bolts}}$$

$$F_T = 12.5\,Sa_H - 6.75(1 - .249\,Sa_H)$$

$$F_T = 14.18\,Sa_H - 6.75$$

$$\left(\frac{14.18\,Sa_H - 6.75}{33.24}\right)^2 + \left(\frac{13.14\,Sa_H}{19.95}\right)^2 = 1$$

$$0.1820\,Sa_H^2 - .1733\,Sa_H + .0412 + .4338\,Sa^2 - 1 = 0$$

$$.6158\,Sa_H^2 - .1733\,Sa_H - .9588$$

$$Sa_H = \frac{0.1733 \pm \sqrt{(.1733)^2 + (4)(.6158)(.9588)}}{(2)(.6158)}$$

$$Sa_H = 1.40\ g$$

consider this to be a 3σ range

$$\beta_r = \frac{1}{3}\ln\frac{1.67}{1.40} = 0.06$$

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Develop Fragility Parameters, $\check{F}$, β_r, β_u, & HCLPF

Capacity

Strength $\check{F} = 0.71g / 0.71g = 1.0$ (see page 8)

$\beta_r = 0$

$\beta_u = \frac{1}{1.65} \ln \frac{1.2}{1.0} = 0.11$ code to median yield

$\oplus \frac{1}{1.65} \ln \frac{0.60}{0.5} = 0.11$ Shear F_y / Tension F_y

$\beta_u = 0.16$

Inelastic Response:

Capacity based on ultimate strength of bolt which is predominately controlled by shear failure Thus no ductility assumed

$\check{F} = 1.0 \quad \beta_r = \beta_u = 0$

Equipment Response

Spectral Shape (difference between smoothed spectra and raw spectra)

$\check{F} = 1.0$

$\beta_r = 0$

$\beta_u = 0.05$ (estimate)

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<u>Damping</u> (high frequency response, thus damping effect neglible)

$\tilde{F} = 1.0$

$\beta_r = 0$

$\beta_u = 0$

<u>Modeling</u>

Frequency: From simulation on frequency (horiz)

$\beta_u = 0.14$

$\tilde{F} = 1.0$

$\beta_r = 0$

$\beta_u = 0.14$ (see page 5)

Mode Shape: Primarily single mode no variability

$\tilde{F} = 1.0$

$\beta_r = 0$

$\beta_u = 0$

<u>Mode Combination</u> Simple structure in high frequency domain

$\tilde{F} = 1.0 \quad \beta_r = \beta_u = 0$

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Horizontal Component Phasing

$\tilde{F} = 1.0$

$\beta_r = 0.06$ (see page 10)

$\beta_u = 0$

Structure Response

Ground Motion: This parameter includes:

1) Peak-to-Peak Spectrum variation
2) Horizontal Direction Variability
3) Equipment Horizontal Component Coupling

See Appendix A – Coupling is small ($C < 0.1$)

$\tilde{F} = 1.0$

$\beta_r = 0.25$

$\beta_u = 0$

Damping Median: 7% — Sa (Ground at 7.2hz) 1.89g

-1σ 5% — 2.12

$\beta_u = \frac{1}{1} \ln \frac{2.12}{1.89} = 0.11$

$\beta_r = 0$

$\tilde{F} = 1.0$ (used 7% in strength calc)

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Modeling

Frequency - since building frequency in region of ground spectrum plateau little change will occur. See equipment modeling frequency, which included the effect of building frequency variability on shifting floor response spectra

$\tilde{F} = 1.0 \quad \beta_r = \beta_u = 0$

Mode shape - Assume controlled by first mode

$\tilde{F} = 1.0 \quad \beta_r = 0 \quad \beta_u = 0.10$ (estimate)

Mode Combination (controlled by first mode - estimate)

$\tilde{F} = 1.0 \quad \beta_r = \beta_u = 0$

Horizontal Component Phasing

$\tilde{F} = 1.0 \quad \beta_r = 0.10 \quad \beta_u = 0$ (estimate)

SSI (Rock site)

$\tilde{F} = 1.0 \quad \beta_r = 0 \quad \beta_u = 0.05$

Inelastic Response (structure assumed to be linear)

$\tilde{F} = 1.0 \quad \beta_r = \beta_u = 0$

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Parameter	$\check{F}$	β_r	β_u
Capacity			
Strength	1.0	0	0.16
Inelastic Response	1.0	0	0
Equipment Response			
Spectral Shape	1.0	0	0.05
Damping	1.0	0	0
Modeling – Frequency	1.0	0	0.14
– Mode Shape	1.0	0	0
Mode Combination	1.0	0	0
Horiz. Component Phasing	1.0	0.06	0
Structure Response			
Ground Motion	1.0	0.25	0
Damping	1.0	0	0.11
Modeling – Frequency	1.0	0	0
Mode Shape	1.0	0	0.10
Mode Combination	1.0	0	0
Horiz. Component Phasing	1.0	0.10	0
SSI	1.0	0	0.05
Inelastic Response	1.0	0	0
Combined:	1.0	0.28	0.27

$\check{a}_{pga} = (1.0)(0.71)g = 0.71g$ $\quad \check{a}_{Peak\ 84\%} = (0.71)(1.4) = 1.0g$

$HCLPF_m \ (0.71)\ e^{-1.65(.27+.28)} = 0.28g$

$HCLPF_{Peak\ 84\%} \ 1.4\ (0.28)g = 0.39g$ (see Appendix B)

B

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Compute HCLPF Directly (See Appendix C)

$\Sigma X \cdot \beta = 1.65(.28+.27) = 0.91$

Parameter	Conservatism	X·β	Deterministic Input
Strength	99% NEP 2.88 x	.46	Code to median $+1.65\sigma = 1.65(.11) = .18$ Shear F_y/Tension F_y $-2.54\sigma = 2.54(.11) = .28$ $.46$ $\tau_u = (0.6e^{.28})(60) = 27.2$ ksi $F_V = (27.2)(.4617) = 10.92$k
Equip Frequency	−1 σ	0.14	$Sa_R = 0.42g\, e^{.11}$ (for 0.18g pga) $= 0.47g$
Ground Motion	+1 σ	0.20	$e^{.20} = 1.22$
Building Damping	+1 σ	0.11	use 7% $F = \frac{2.12}{1.89} = 1.12$
		.91	$Sa_H = (0.47)(1.22)(1.12) = 0.64g$ (for 0.18g pga)

Use other parameters at median values

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Capacity

$F_V = 10.92^k$

$F_T = (60)(.4617) = 27.70^k$

Input: $Sa_H = 0.64_g$ (for 0.18 $_g$ pga)

$S_V = 0.40_g$ (0.64/1.67 × 1.04) (see p 7)

↑ keep scaling to Sa_H

Solution:

$$\left(\frac{S_T}{F_T}\right)^2 + \left(\frac{S_V}{F_V}\right)^2 = 1 \quad \text{(see page 8)}$$

$$\left(\frac{6.68\,Sa_H - 6.75}{27.70}\right)^2 + \left(\frac{11.98\,Sa_H}{10.92}\right)^2 = 1$$

$$0.0582\,Sa^2 - .1175\,Sa + .0594 + 1.204\,Sa^2 - 1 = 0$$

$$1.262\,Sa^2 - .1175\,Sa - .9406 = 0$$

$$Sa_H = \frac{.1175 \pm \sqrt{(.1175)^2 + (4)(1.262)(.9406)}}{(2)(1.262)}$$

$Sa_H = 0.91_g$ (bolt in comp)

$$Sa_H = \frac{10.92}{11.98} = 0.91_g$$

Median HCLPF = $\frac{0.91}{0.64} \times .18 = \underline{\underline{0.26_g}}$

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84% NEP Max or HCLPF = 0.26 × 1.4 = $\underline{0.36_g}$ (see Appendix B for 1.4)

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APPENDIX A

This appendix gives the median factor and β_F values for the effect of:

1. coupling of earthquake directions on the total response of a critical element in a component (e.g. a bolt holding down a tank)

2. peak-to-peak ground response variability. Assumed to be lognormal with $\breve{F} = 1.0$ $\beta = 0.20$

3. Component-to Component horizontal earthquake direction variability. Assumed to be lognormal with $\breve{F} = 1.0$ $\beta = 0.15$

The input to the critical element in the analysis is assumed to be based on median ground spectra input. The ratio of the response due to 1 horizontal direction to the response from the other horizontal direction is factor "C". (you find this out when you do the analysis)

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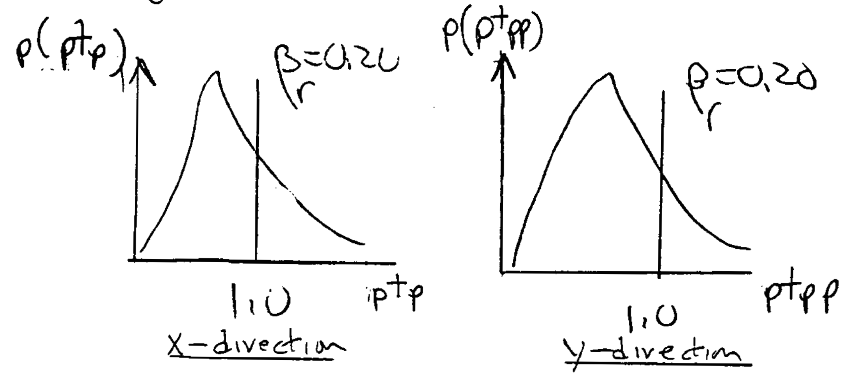
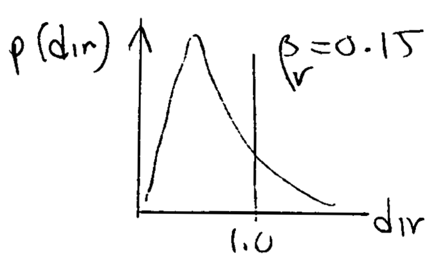
The peak-to-peak distribution for the two directions are assumed to be independent (i.e. just because one spectral ordinate is high doesn't mean the corresponding one at 90° is also high)



Each direction component relative to the median is also independent of ptp and ptpp:



Now there are two rules for combining response SRSS and 100-40. (Note that it is assumed that the vertical earthquake component is not significant; if it is then a similar approach can be used)

The *actual* combined response relative to the *calculated* combined response (i.e the analysis you perform using the median ground input for the two horizontal directions) is seen directly:

$$\text{Relative Response} = \sqrt{\frac{(dir \cdot ptp)^2 + \left(C \cdot \frac{ptpp}{dir}\right)^2}{1 + C^2}}$$

SRSS rule

$$\text{Relative Response} = \frac{dir \cdot ptp + .40\,C\,\frac{ptpp}{dir}}{1 + .40C}$$

100-40 rule

Following are the two MATHCAD programs for obtaining the distributions (assumed to be lognormal) parameters: $\check{F}$ and β_r for each of the two rules, i.e.

DIR.FAT: 100-40 rule
DIR.FAC: SRSS rule

On the last page is the median factors, $\check{F}$ and associated β_r values for different values of C. Thus, knowing C the correct $\check{F}$ and β_r can be used to reflect the variability in total response due to variability in the ground motion.

MCAD FILES DIR.FAT

This program calculates the distribution on response for peak-to-peak and direction variability when the combination method is the 100 40 rule

NORMAL DISTRIBUTION

$b1 \equiv .319381530$

$b2 \equiv -.356563782$

$b3 \equiv 1.781477937$

$b4 \equiv -1.821255978$

$b5 \equiv 1.330274429$

$p \equiv .2316419$

$$Z(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION

$c0 \equiv 2.515517$

$c1 \equiv .802853$

$c2 \equiv .010328$

$d1 \equiv 1.432788$

$d2 \equiv .189269$

$d3 \equiv .001308$

$$t(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$INOR(p) \equiv (-1)^{p < .5} \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3} \right]$$

$n := 199$

$i := 0 \,..n$

$$ptp_i := 1.0 \cdot \exp\left[0.20 \cdot INOR\left[\frac{i + rnd(1)}{n + 1}\right]\right] \quad \square$$

$$dir_i := 1.0 \cdot \exp\left[0.15 \cdot INOR\left[\frac{i + rnd(1)}{n + 1}\right]\right] \quad \square$$

$min(dir) = 0.659$ □

$max(dir) = 1.555$ □

$WRITE(DIR) := dir_i$ □

$WRITE(PTP) := ptp_i$ □

$WRITE(PTPP) := ptp_i$ □

Randomly mix both the direction and the two peak-to-peak arrays EXIT TO DOS and run CADMIX

$dir_i := READ(DIR)$

$ptp_i := READ(PTP)$

$ptpp_i := READ(PTPP)$

$c := 0 \,..4$

dir_c	ptp_c	$ptpp_c$
1.087	1.033	0.796
0.936	0.922	0.907
1.189	1.013	1.643
1.236	0.975	1.108
1.194	1.026	1.346

$$dir_i := \frac{dir_i \cdot ptp_i + .40 \cdot C \cdot \dfrac{ptpp_i}{dir_i}}{1 + .40 \cdot C}$$

$min(dir) = 0.459$

$max(dir) = 1.835$

$m := 10$

$j := 0 \,..m$

$$spc_j := (min(dir) - .0001) + \frac{j}{m} \cdot (max(dir) - min(dir) + .0002)$$

$freq := hist(spc, dir)$

$k := 0 \,..(m - 1)$

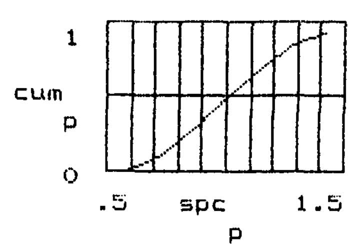
$$\sum_k freq_k = 200$$

$cum_0 := 0$

$p := 1 \,..m$

$cum_p := cum_{p-1} + freq_{p-1}$

$$cum_p := \frac{cum_p}{n + 1}$$



s := lspline(spc,cum)

Find the median value:

x := 1.0

j	cum_j	spc_j
0	0	0.458
1	0.01	0.596
2	0.105	0.734
3	0.295	0.871
4	0.515	1.009
5	0.69	1.147
6	0.84	1.284
7	0.925	1.422
8	0.96	1.56
9	0.99	1.697
10	1	1.835

root(interp(s,spc,cum,x) − .5,x) = 0.999

Find the + 1σ value:

root(interp(s,spc,cum,x) − .84134,x) = 1.286

Find the − 1σ value:

root(interp(s,spc,cum,x) − .15866,x) = 0.779

$tcum_j := cum_j$

$tspc_j := spc_j$

m := m − 2

s := 1

j := 0 ..m

$cum_j := tcum_{j+1}$

$spc_j := spc_{j+1}$

j	cum_j	spc_j
0	0.01	0.596
1	0.105	0.734
2	0.295	0.871
3	0.515	1.009
4	0.69	1.147
5	0.84	1.284
6	0.925	1.422
7	0.96	1.56
8	0.99	1.697

This program take the calculated probabilities of failure and the associated peak ground acceleration values and performs a least squares fit in the log-probability domain to obtain the "best fit" median and ß value.

$x1_j := cum_j$

$y1_j := spc_j$

$y1_j := \ln\left[y1_j\right]$

$x1_j := INOR\left[x1_j\right]$

slope(x1,y1) = 0.235

intercept(x1,y1) = 0.008

corr(x1,y1) = 0.999

$\beta := slope(x1,y1)$

$a_{med} := \exp(intercept(x1,y1))$

C ≡ 0

$a_{med} = 1.008$

$\beta = 0.235$

$z1_j := intercept(x1,y1) + slope(x1,y1) \cdot x1_j$

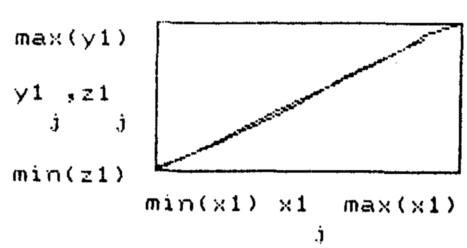
max(y1) = 0.529

min(y1) = −0.517

min(z1) = −0.539

max(x1) = 2.327

min(x1) = −2.327



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j	$x1_j$	$y1_j$	$z1_j$
0	-2.327	-0.517	-0.539
1	-1.254	-0.31	-0.287
2	-0.538	-0.138	-0.119
3	0.038	0.009	0.017
4	0.495	0.137	0.124
5	0.994	0.25	0.241
6	1.44	0.352	0.346
7	1.751	0.444	0.419
8	2.327	0.529	0.554

Tabulate probabilities to compare calculated and best fit values:

$a_j := \exp\left[y1_j\right]$ Peak ground accelerations

$p_j := NOR\left[x1_j\right]$ Probability of failure values

$$p_{best_j} := NOR\left[\frac{\ln\left[\frac{a_j}{a_{med}}\right]}{\beta}\right]$$ Best fit probability of failure values

j	a_j	p_j	p_{best_j}
0	0.596	0.01	0.013
1	0.734	0.105	0.088
2	0.871	0.295	0.268
3	1.009	0.515	0.502
4	1.147	0.69	0.709
5	1.284	0.84	0.849
6	1.422	0.925	0.929
7	1.56	0.96	0.969
8	1.697	0.99	0.987

MCAD FILES DIR.FAC

This program calculates the distribution on response for peak-to-peak and direction variability when the combination method is the SRSS rule

NORMAL DISTRIBUTION

$b1 \equiv .319381530$

$b2 \equiv -.356563782$

$b3 \equiv 1.781477937$

$b4 \equiv -1.821255978$

$b5 \equiv 1.330274429$

$p \equiv .2316419$

$$Z(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION

$c0 \equiv 2.515517$

$c1 \equiv .802853$

$c2 \equiv .010328$

$d1 \equiv 1.432788$

$d2 \equiv .189269$

$d3 \equiv .001308$

$$t(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$INOR(p) \equiv (-1)^{p < .5} \cdot \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3} \right]$$

$n := 199$

$i := 0 \,..n$

$$ptp_i := 1.0 \cdot exp\left[0.20 \cdot INOR\left[\frac{i + rnd(1)}{n + 1}\right]\right]$$ ▫

$$dir_i := 1.0 \cdot exp\left[0.15 \cdot INOR\left[\frac{i + rnd(1)}{n + 1}\right]\right]$$ ▫

$min(dir) = 0.659$ ▫

$max(dir) = 1.555$ ▫

$WRITE(DIR) := dir_i$ ▫

$WRITE(PTP) := ptp_i$ ▫

$WRITE(PTPP) := ptp_i$ ▫

Randomly mix both the direction and the two peak-to-peak arrays EXIT TO DOS and run CADMIX

$dir_i := READ(DIR)$

$ptp_i := READ(PTP)$

$ptpp_i := READ(PTPP)$

$c := 0 \,..4$

dir_c	ptp_c	$ptpp_c$
1.087	1.033	0.796
0.936	0.922	0.907
1.189	1.013	1.643
1.236	0.975	1.108
1.194	1.026	1.346

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$$dir_i := \frac{\sqrt{\left[dir_i \cdot ptp_i\right]^2 + \left[C \cdot \frac{ptpp_i}{dir_i}\right]^2}}{\sqrt{1 + C^2}}$$

min(dir) = 0.459

max(dir) = 1.835

m := 10

j := 0 ..m

$$spc_j := (\min(dir) - .0001) + \frac{j}{m} \cdot (\max(dir) - \min(dir) + .0002)$$

freq := hist(spc,dir)

k := 0 ..(m − 1)

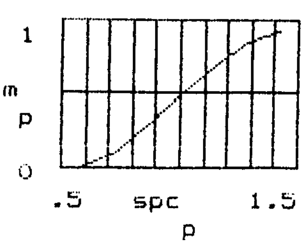
$$\sum_k freq_k = 200$$

$cum_0 := 0$

p := 1 ..m

$$cum_p := cum_{p-1} + freq_{p-1}$$

$$cum_p := \frac{cum_p}{n + 1}$$



```
s := lspline(spc,cum)
```

Find the median value:

```
x := 1.0

root(interp(s,spc,cum,x) - .5,x) = 0.999
```

Find the + 1σ value:

```
root(interp(s,spc,cum,x) - .84134,x) = 1.286
```

Find the - 1σ value:

```
root(interp(s,spc,cum,x) - .15866,x) = 0.779
```

j	cum_j	spc_j
0	0	0.458
1	0.01	0.596
2	0.105	0.734
3	0.295	0.871
4	0.515	1.009
5	0.69	1.147
6	0.84	1.284
7	0.925	1.422
8	0.96	1.56
9	0.99	1.697
10	1	1.835

$tcum_j := cum_j$

$tspc_j := spc_j$

$m := m - 2$

$s := 1$

$j := 0 .. m$

$cum_j := tcum_{j+1}$

$spc_j := spc_{j+1}$

j	cum_j	spc_j
0	0.01	0.596
1	0.105	0.734
2	0.295	0.871
3	0.515	1.009
4	0.69	1.147
5	0.84	1.284
6	0.925	1.422
7	0.96	1.56
8	0.99	1.697

This program take the calculated probabilities of failure and the associated peak ground acceleration values and performs a least squares fit in the log-probability domain to obtain the "best fit" median and ß value.

$x1_j := cum_j$

$y1_j := spc_j$

$y1_j := \ln\left[y1_j\right]$

$x1_j := INOR\left[x1_j\right]$

slope(x1,y1) = 0.235 $\qquad$ β := slope(x1,y1)

intercept(x1,y1) = 0.008 $\qquad$ a_{med} := exp(intercept(x1,y1))

corr(x1,y1) = 0.999

C ≡ 0	a_{med} = 1.008 β = 0.235

$z1_j$:= intercept(x1,y1) + slope(x1,y1)·$x1_j$

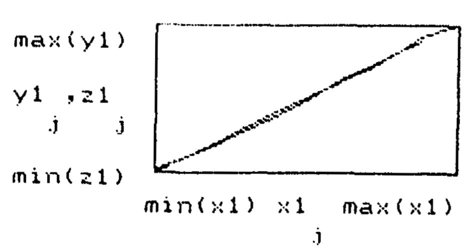
max(y1) = 0.529

min(y1) = -0.517

min(z1) = -0.539

max(x1) = 2.327

min(x1) = -2.327



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j	$x1_j$	$y1_j$	$z1_j$
0	-2.327	-0.517	-0.539
1	-1.254	-0.31	-0.287
2	-0.538	-0.138	-0.119
3	0.038	0.009	0.017
4	0.495	0.137	0.124
5	0.994	0.25	0.241
6	1.44	0.352	0.346
7	1.751	0.444	0.419
8	2.327	0.529	0.554

Tabulate probabilities to compare calculated and best fit values:

$a_j := \exp\left[y1_j\right]$ Peak ground accelerations

$p_j := NOR\left[x1_j\right]$ Probability of failure values

$$p_{best_j} := NOR\left[\frac{\ln\left[\frac{a_j}{a_{med}}\right]}{\beta}\right]$$ Best fit probability of failure values

j	a_j	p_j	p_{best_j}
0	0.596	0.01	0.013
1	0.734	0.105	0.088
2	0.871	0.295	0.268
3	1.009	0.515	0.502
4	1.147	0.69	0.709
5	1.284	0.84	0.849
6	1.422	0.925	0.929
7	1.56	0.96	0.969
8	1.697	0.99	0.987

For 100-40 Rule

C	$\check{F}$	β_r
0	1.01	0.24
0.1	1.01	0.23
0.2	1.02	0.21
0.3	1.02	0.20
0.4	1.02	0.19
0.5	1.02	0.19
0.6	* 1.02	0.18
0.7	1.02	0.18
0.8	1.02	0.17
0.9	1.02	0.17
1.0	1.02	0.16

← For this case use $\check{F}=1.0$ $\beta=0.25$

* Caution: These values are slightly low because of the possibility the other direction may control, i.e.,

C	$\check{F}$
0.7	1.03
1.0	1.09

For SRSS Rule

C	$\check{F}$	β_r
0	1.01	0.24
0.1	1.01	0.23
0.2	1.02	0.21
0.3	1.03	0.20
0.4	1.04	0.18
0.5	1.04	0.18
0.6	1.04	0.17
0.7	1.04	0.17
0.8	1.04	0.16
0.9	1.04	0.15
1.0	1.04	0.15

← For this case use $\check{F}=1.0$ $\beta=0.25$

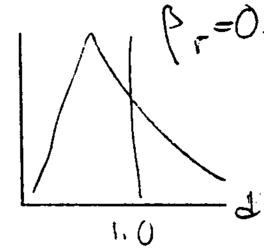
Note: These factors are for the case where a single direction controls the capacity. They do not apply to case of capacity controlled by all directions (e.g. circular tank with bolts)

Appendix B

This Appendix gives the factor necessary to convert a NCLPF spectrum calculated based on the assumption that the ground motion horizontal inputs are median, both in the sense of peak-to-peak variability and in the sense of a geometric average of the two horizontal components. In other words, a NCLPF spectrum calculated on this basis would be compared to a real earthquake by first averaging (geometrically) the two horizontal direction spectra and then comparing it to the HCLPF spectrum. In the frequency range of interest, if 50% of the ordinates are less than the NCLPF spectrum, then it is concluded that the NCLPF has not been exceeded.

To convert to a 84% NEP maximum component spectrum the variabilities due to both peak-to-peak and horizontal component-to-component effects must be considered. Note that the comparison of the NCLPF given this way is to take maximum earthquake component and to compare the corresponding response spectrum to the HCLPF spectrum in the frequency range of interest, if 84 % of the ordinates of the maximum component spectrum are less than the HCLPF spectrum, then it is concluded that the HCLPF has not been exceeded.

MATHCAD program PEAK.COM calculates first a distribution for only the direction-to-direction variability to find the median and β_r for the maximum component. This is simply done by sampling from the direction distribution dir:



If the value is less than 1.0 the inverse is used. The resulting distribution is fit to a lognormal (it fits pretty good!) and the parameters are:

$$\check{F} = 1.11$$

$$\beta_r = 0.12$$

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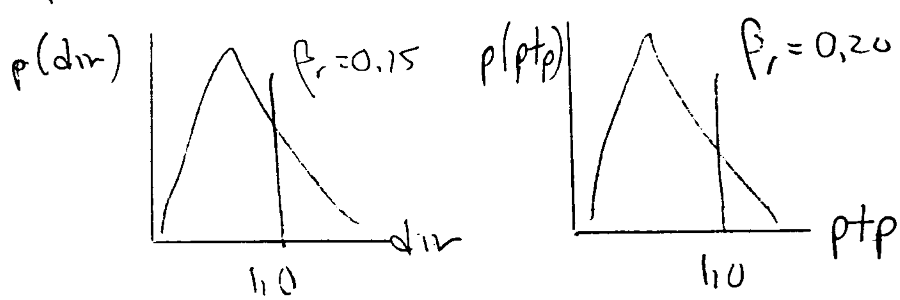
This was done as a side step for use in the Tank analysis

The second program PEAKEDIR.MCD calculates the distribution when both direction-to-direction and peak-to-peak variabilities are present



The combined parameter M(dir) • ptp has a lognormal (good fit) distribution with parameters $\tilde{F} = 1.11$, $\beta = 0.24$

where: M(dir) = dir, dir > 1
= 1/dir, dir < 1

Thus, the 84 NEP maximum component factor is just: $F_{84,MAX} = 1.11e^{.24} = 1.4$

Therefore to convert a HCLPF spectrum from a median level to a 84 NEP maximum component HCLPF spectrum, just multiply the former by 1.4.

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MCAD FILES PEAK.COM

his program calculates the distribution for the maximum of two horizontal
omponents where only the maximum component variability is included

ORMAL DISTRIBUTION

$1 \equiv .319381530$

$2 \equiv -.356563782$

$3 \equiv 1.781477937$

$4 \equiv -1.821255978$

$5 \equiv 1.330274429$

$p \equiv .2316419$

$$(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$(x) \equiv \frac{1}{1 + p\cdot|x|}$$

$$OR(x) \equiv \left|(x > 0) - Z(x)\cdot\left[b1\cdot t(x) + b2\cdot t(x)^2 + b3\cdot t(x)^3 + b4\cdot t(x)^4 + b5\cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION

$0 \equiv 2.515517$

$1 \equiv .802853$

$2 \equiv .010328$

$1 \equiv 1.432788$

$2 \equiv .189269$

$3 \equiv .001308$

$$(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$NOR(p) \equiv (-1)^{p < .5} \cdot \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3} \right]$$

$$n := 199$$

$$i := 0 \, .. n$$

$$dir_i := 1.0 \cdot \exp\left[0.15 \cdot INOR\left[\frac{i + rnd(1)}{n + 1} \right] \right]$$

$$dir_i := dir_i \cdot \left[dir_i > 1 \right] + \frac{1}{dir_i} \cdot \left[dir_i < 1 \right]$$

This step makes all direction values equal to or greater than 1 (i.e., peak component)

$$min(dir) = 1.001$$

$$max(dir) = 1.534$$

$$m := 10$$

$$j := 0 \, .. m$$

$$spc_j := (min(dir) - .0001) + \frac{j}{m} \cdot (max(dir) - min(dir) + .0002)$$

$$freq := hist(spc, dir)$$

$$k := 0 \, ..(m - 1)$$

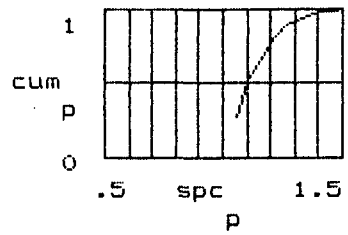
$$\sum_k freq_k = 200$$

$cum_0 := 0$

$p := 1 ..m$

$cum_p := cum_{p-1} + freq_{p-1}$

$$cum_p := \frac{cum_p}{n + 1}$$



s := lspline(spc,cum)

j	cum_j	spc_j
0	0	1.001
1	0.275	1.054
2	0.5	1.107
3	0.68	1.161
4	0.8	1.214
5	0.885	1.267
6	0.935	1.321
7	0.97	1.374
8	0.98	1.427
9	0.99	1.48
10	1	1.534

Find the median value:

x := 1.0

root(interp(s,spc,cum,x) − .5,x) = 1.107

Find the + 1σ value:

root(interp(s,spc,cum,x) − .84134,x) = 1.237

Find the − 1σ value:

root(interp(s,spc,cum,x) − .15866,x) = 1.031

$cum_j := cum_j$

$spc_j := spc_j$

$:= m - 2$

$s := 1$

$:= 0 .. m$

$cum_j := tcum_{j+1}$

$spc_j := spc_{j+1}$

j	cum_j	spc_j
0	0.275	1.054
1	0.5	1.107
2	0.68	1.161
3	0.8	1.214
4	0.885	1.267
5	0.935	1.321
6	0.97	1.374
7	0.98	1.427
8	0.99	1.48

This program take the calculated probabilities of failure and the associated peak ground acceleration values and performs a least squares fit in the log-probability domain to obtain the "best fit" median and ß value.

$x1_j := cum_j$

$y1_j := spc_j$

$y1_j := \ln\left[y1_j\right]$

$x1_j := INOR\left[x1_j\right]$

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slope(x1,y1) = 0.117 $\qquad$ ß := slope(x1,y1)

intercept(x1,y1) = 0.105 $\qquad$ a_{med} := exp(intercept(x1,y1))

corr(x1,y1) = 0.995

a_{med} = 1.11
ß = 0.117

$z1_j$:= intercept(x1,y1) + slope(x1,y1)·$x1_j$

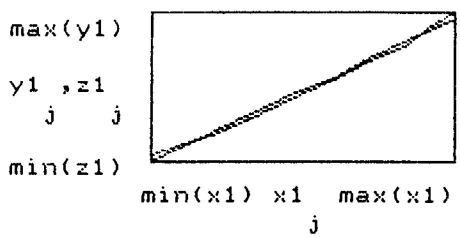
nax(y1) = 0.392

nin(y1) = 0.053

nin(z1) = 0.034

nax(x1) = 2.327

iin(x1) = -1.514



j	$x1_j$	$y1_j$	$z1_j$
0	-1.514	-0.235	-0.241
1	-0.659	-0.036	-0.039
2	0.088	0.129	0.136
3	0.643	0.272	0.267
4	1.2	0.396	0.398
5	1.751	0.507	0.527
6	2.171	0.606	0.626
7	2.576	0.697	0.721
8	2.576	0.78	0.721

Tabulate probabilities to compare calculated and best fit values:

$_j$:= exp$\left[y1_j\right]$ $\qquad$ Peak ground accelerations

$_j$:= NOR$\left[x1_j\right]$ $\qquad$ Probability of failure values

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$$p_{best_j} := \mathrm{NOR}\left[\frac{\ln\left[\frac{a_j}{a_{med}}\right]}{\beta}\right]$$

Best fit probability of failure values

j	a_j	p_j	p_{best_j}
0	0.791	0.065	0.068
1	0.964	0.255	0.259
2	1.138	0.535	0.524
3	1.312	0.74	0.747
4	1.486	0.885	0.884
5	1.66	0.96	0.952
6	1.833	0.985	0.982
7	2.007	0.995	0.993
8	2.181	0.995	0.998

MCAD FILES DIR&PEAK.MCD

This program calculates the distribution for the maximum of two horizontal components where both the peak-to-peak and maximum component variability are included

NORMAL DISTRIBUTION

$$b1 \equiv .319381530$$

$$b2 \equiv -.356563782$$

$$b3 \equiv 1.781477937$$

$$b4 \equiv -1.821255978$$

$$b5 \equiv 1.330274429$$

$$p \equiv .2316419$$

$$Z(x) \equiv \frac{\exp\left[\frac{-x^2}{2}\right]}{2.506628275}$$

$$t(x) \equiv \frac{1}{1 + p \cdot |x|}$$

$$NOR(x) \equiv \left|(x > 0) - Z(x) \cdot \left[b1 \cdot t(x) + b2 \cdot t(x)^2 + b3 \cdot t(x)^3 + b4 \cdot t(x)^4 + b5 \cdot t(x)^5\right]\right|$$

INVERSE NORMAL DISTRIBUTION

$$c0 \equiv 2.515517$$

$$c1 \equiv .802853$$

$$c2 \equiv .010328$$

$$d1 \equiv 1.432788$$

$$d2 \equiv .189269$$

$$d3 \equiv .001308$$

$$(p) \equiv \sqrt{\ln\left[\frac{1}{((p > .5) - p)^2}\right]}$$

$$NOR(p) \equiv (-1)^{p < .5} \cdot \left[t(p) - \frac{c0 + c1 \cdot t(p) + c2 \cdot t(p)^2}{1 + d1 \cdot t(p) + d2 \cdot t(p)^2 + d3 \cdot t(p)^3} \right]$$

:= 199

:= 0 ..n

$$tp_i := 1.0 \cdot exp\left[0.20 \cdot INOR\left[\frac{i + rnd(1)}{n + 1}\right]\right]$$

$$dir_i := 1.0 \cdot exp\left[0.15 \cdot INOR\left[\frac{i + rnd(1)}{n + 1}\right]\right]$$

$$dir_i := dir_i \cdot \left[dir_i > 1\right] + \frac{1}{dir_i} \cdot \left[dir_i < 1\right]$$

This step makes all direction values equal to or greater than 1 (i.e., peak component)

ıin(dir) = 1.001

ıax(dir) = 1.534

$WRITE(DIR) := dir_i$ ▫

$WRITE(PTP) := ptp_i$ ▫

Randomly mix both the direction and the two peak-to-peak arrays EXIT TO DOS and run CADMIX

$dir_i := READ(DIR)$

$ptp_i := READ(PTP)$

$c := 0 ..4$

dir_c	ptp_c
1.24	1.131
1.01	0.903
1.181	1.12
1.17	0.8
1.032	0.814

$dir_i := dir_i \cdot ptp_i$

min(dir) = 0.617

max(dir) = 2.354

m := 10

j := 0 ..m

$$spc_j := (\min(dir) - .0001) + \frac{j}{m} \cdot (\max(dir) - \min(dir) + .0002)$$

freq := hist(spc,dir)

k := 0 ..(m − 1)

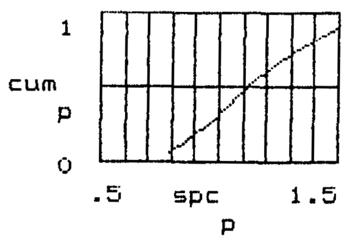
$$\sum_k freq_k = 200$$

$cum_0 := 0$

p := 1 ..m

$cum_p := cum_{p-1} + freq_{p-1}$

$$cum_p := \frac{cum_p}{n + 1}$$



s := lspline(spc,cum)

j	cum_j	spc_j
0	0	0.617
1	0.065	0.791
2	0.255	0.964
3	0.535	1.138
4	0.74	1.312
5	0.885	1.486
6	0.96	1.66
7	0.985	1.833
8	0.995	2.007
9	0.995	2.181
10	1	2.355

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Find the median value:

$x := 1.0$

root(interp(s,spc,cum,x) - .5,x) = 1.115

Find the + 1σ value:

root(interp(s,spc,cum,x) - .84134,x) = 1.425

$\rho = \frac{1}{2} \ln \frac{1.425}{.892} = 0.23$

Find the - 1σ value:

root(interp(s,spc,cum,x) - .15866,x) = 0.892

$tcum_j := cum_j$

$tspc_j := spc_j$

$m := m - 2$

$s := 1$

$j := 0 ..m$

$cum_j := tcum_{j+1}$

$spc_j := spc_{j+1}$

j	cum_j	spc_j
0	0.065	0.791
1	0.255	0.964
2	0.535	1.138
3	0.74	1.312
4	0.885	1.486
5	0.96	1.66
6	0.985	1.833
7	0.995	2.007
8	0.995	2.181

This program take the calculated probabilities of failure and the associated peak ground acceleration values and performs a least squares fit in the log-probability domain to obtain the "best fit" median and ß value.

$x1_j := cum_j$

$y1_j := spc_j$

$y1_j := \ln\left[y1_j\right]$

$x1_j := INOR\left[x1_j\right]$

$slope(x1,y1) = 0.235$

$ß := slope(x1,y1)$

$intercept(x1,y1) = 0.115$

$a_{med} := exp(intercept(x1,y1))$

$corr(x1,y1) = 0.997$

$a_{med} = 1.122$

$ß = 0.235$

$z1_j := intercept(x1,y1) + slope(x1,y1) \cdot x1_j$

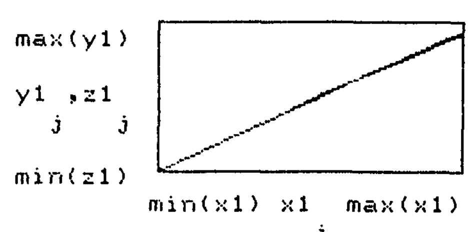
$max(y1) = 0.78$

$min(y1) = -0.235$

$min(z1) = -0.241$

$max(x1) = 2.576$

$min(x1) = -1.514$



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j	$x1_j$	$y1_j$	$z1_j$
0	-1.514	-0.235	-0.241
1	-0.659	-0.036	-0.039
2	0.088	0.129	0.136
3	0.643	0.272	0.267
4	1.2	0.396	0.398
5	1.751	0.507	0.527
6	2.171	0.606	0.626
7	2.576	0.697	0.721
8	2.576	0.78	0.721

Tabulate probabilities to compare calculated and best fit values:

$a_j := \exp\left[y1_j\right]$ Peak ground accelerations

$p_j := NOR\left[x1_j\right]$ Probability of failure values

$$p_{best_j} := NOR\left[\frac{\ln\left[\frac{a_j}{a_{med}}\right]}{\beta}\right]$$ Best fit probability of failure values

j	a_j	p_j	p_{best_j}
0	0.791	0.065	0.068
1	0.964	0.255	0.259
2	1.138	0.535	0.524
3	1.312	0.74	0.747
4	1.486	0.885	0.884
5	1.66	0.96	0.952
6	1.833	0.985	0.982
7	2.007	0.995	0.993
8	2.181	0.995	0.998

Appendix C

This appendix describes a simple procedure for using the results of the fragility analysis to calibrate a deterministic analysis which produces the same NCLPF as obtained using the fragility approach. This is a backfit technique. "Garbage in gives garbage back" The procedure simply says that if you have the results of a fragility analysis you can use them to decide what conservatism to put in each parameter (e.g. response spectrum NEP level, allowable stress, damping, etc.) in order to perform a single deterministic analysis to produce the same NCLPF. This is a formal approach for reconciling the CDFM and FA procedures.

A proof that the procedure should work is given following the procedure steps

PROCEDURE STEPS

The following steps are performed to obtain the parameter values to perform a single deterministic analysis to obtain the same HCLPF as calculated from the FA approach:

1. Calculate $1.65(\beta_r + \beta_u)$. This is the target conservatism.

2. For each of the response categories (e.g. strength, ductility, building damping, equipment damping, etc) assign a part of the target conservatism.

   Note: give the most to those categories that have the biggest category β's

3. For each of the individual category parts divide the portion of the target conservatism given to that part by the corresponding response β e.g. if the conservatism for strength is assigned 0.33 and the corresponding β is 0.20 then, γ is just:

$$\gamma = \frac{0.33}{20} = 1.65$$

4. Simply go to each of the corresponding parameters and select the same level of conservatism given by the γ's

e.g. for strength, select the allowable stress at the 1.65 σ level, i.e.

$$\text{stress} = \check{\text{stress}}\, e^{-1.65 \beta_{\text{stress}}}$$

5. Then perform the deterministic analysis using the calculated parameter values and you will get the same HCLPF (or nearly so)

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PROOF

Response, R

Parameters, x_i

(Vector notation)

$$R = F(x_1, x_2, \ldots, x_n) = F(\underline{X})$$

$$\check{R} \cong F(\check{x}_1, \check{x}_2, \ldots, \check{x}_n) = F(\check{\underline{X}})$$

Now calculate the response at some confidence level, i.e.

$$\check{R}\, e^{-\gamma_R \beta_R} \cong F(\check{x}_1 e^{-\gamma_1 \beta_1}, \check{x}_2 e^{-\gamma_2 \beta_2}, \ldots, \check{x}_N e^{-\gamma_N \beta_N}) \quad (1)$$

The question at this point is: what are the γ_i's?

Define: $\gamma_R \beta_R = \gamma_{R_1} \beta_{R_1} + \gamma_{R_2} \beta_{R_2} + \ldots + \gamma_{R_N} \beta_{R_N}$

where β_{R_j} is the β associated with response type j (e.g. strength and building damping)

γ_{R_j} is the associated level of conservatism

Note: It is clear that there is a large number of combinations of γ_{R_j} that can satisfy the definition

Remember: $\beta_e = \left[\sum \beta_{R_j}^2\right]^{1/2}$

Now expanding (1) by substituting the definition into the left side of eqn 1 and making an approximation to the right side (typically what is done in Seismic PRA calculations).

$$F(\check{\underline{X}})\, e^{-\sum \gamma_{R_j} \beta_{R_j}} = F(\check{\underline{X}}) \left[\frac{F(\check{X}_1 e^{-\gamma_1 \beta_1}, \check{X}_2, \ldots \check{X}_N)}{F(\check{\underline{X}})} , \frac{F(\check{X}_1, \check{X}_2 e^{-\gamma_2 \beta_2}, \ldots \check{X}_N)}{F(\check{\underline{X}})} , \ldots \frac{F(\check{X}_1, \check{X}_2, \ldots \check{X}_N e^{-\gamma_N \beta_N})}{F(\check{\underline{X}})} \right]$$

but:

$$F(\check{X}_1, \check{X}_2, \ldots \check{X}_i e^{-\gamma_i \beta_i}, \ldots, \check{X}_N) \cong F(\check{\underline{X}})\, e^{-\gamma_i \beta_{R_i}}$$

(this how you get β_{R_i})

hence

$$\sum \gamma_{R_j} \beta_{R_j} = \sum \gamma_i \beta_{R_i}$$

one solution is simply $\gamma_i = \gamma_{R_i}$

Thus the definition $\gamma_R \beta_R = \sum \gamma_{R_i} \beta_{R_i}$ and using $\gamma_i = \gamma_{R_i}$ the procedure steps follow

An Approach to Comparing the CDFM and FA Methods

by John W. Reed

An approach is presented to compare and reconcile the results from the HCLPF calculations using the CDFM and FA methods. The purpose for performing a reconciliation is to identify where conservatism is placed in the CDFM analysis and to obtain a measure of influence of each of the basic parameters relative to the conservatism in the HCLPF capacity. The approach is based on the procedure that J. W. Reed used to obtain backfit CDFM results from the FA analysis, which is documented earlier in this appendix.

The approach is based on two fundamental results. First, the number of standard deviations, γ, between the value of a basic parameter (e.g., damping, response spectrum input, or member strength) used in the CDFM analysis and the corresponding median value (in a multiplicative sense, since the underlying distribution is assumed to be lognormal) is the same γ conservatism in the HCLPF capacity due to <u>that</u> basic parameter.

Second, the following relationship is true when the HCLPF values from the CDFM and FA methods are the same:

$$\sum \gamma_i \beta_{CAPi} = 1.65(\beta_r + \beta_u) \qquad (1)$$

where:

γ_i: Number of standard deviations between the basic parameter i and its median value (in a multiplicative sense, since the underlying distribution is lognormal).

β_{CAPi}: the logarithmic standard deviation of the final capacity due to the variability in basic parameter i.

β_r, β_u: The total randomness and uncertainty logarithmic standard deviations for the final capacity, respectively.

The procedure to reconciling the results from the CDFM and FA methods consist of the following steps:

1. Identify the equation which relates the basic parameters to the final capacity of the component. Note that in using this approach the reconciliation can be performed only if the same equation is used in both the CDFM and FA calculations.

2. Using a Taylor series expansion of the final capacity equation, and a second moment approximation, obtain the median and logarithmic standard deviation values of the final capacity in terms of the statistics of the basic parameters (i.e., median and β).

3. For each basic parameter value, $CDFM_i$, assumed in the CDFM analysis, and the corresponding statistics for that basic parameter obtained in the FA analysis, calculate the γ_i values as follows:

$$\gamma_i = \left| \frac{1}{\beta_i} \ln \left(\frac{CDFM_i}{Median_i} \right) \right| \tag{2}$$

4. From the equation for the logarithmic standard deviation of the capacity, β_{CAP}, in terms of the basic parameter statistics, calculate β_{CAPi} which is the β in capacity due to the variability in basic parameter i. This step can be performed in terms of the combined basic parameter variability or in terms of the separate randomness and uncertainty parts (i.e., β_{r_i} and β_{u_i}). The only constraint is that the variability of all the basic parameters, either combined or separate, must be accounted for.

5. As a check, $\Sigma\gamma_i\beta_{CAPi}$ should be essentially equal to 1.65 (β_r and β_u) as described above (see equation 1). In general the two parts will not be exactly equal because the second moment approach is only approximate.

From these results, the γ_i values indicate the level of conservatism assumed in the CDFM analysis for each of the basic parameters. The ratio $\gamma_i\beta_{CAPi}/1.65(\beta_r+\beta_u)$ is the fraction of the total conservatism contributed by the ith parameter value used in the CDFM method.

Example

The calculations by R. P. Kennedy for the Starting Air Tank are used as an example to demonstrate a reconciliation between the CDFM and FA approaches for calculating the HCLPF capacity (see Appendix A for Kennedy's calculations). The governing equation for the capacity of the Starting Air Tank follows:

$$CAP = \frac{28.8\ Tu + 11.3}{38.2\ AF_H + 11.3\ F_V} \quad \text{(units: g)} \tag{3}$$

where Tu = Angle capacity (units: kips)

AF_H = Horizontal response factor (unitless)

F_V = Combined vertical response factor (unitless)

Note that the constants in the equation contain the appropriate units so that the capacity is given in terms of acceleration (i.e., gravity units). Using a Taylor series expansion and the second moment approach, the median and logarithmic standard deviation for capacity in terms of the basic parameters follow:

Median:

$$\check{CAP} = \frac{28.8\ Tu + 11.3}{38.2\ AF_H + 11.3\ F_V} \tag{4}$$

Logarithmic Standard Deviation (squared):

$$\beta^2_{CAP} = \left(\frac{28.8\ \check{Tu}}{28.8\ \check{Tu} + 11.3}\right)^2 \beta^2_{Tu} + \left(\frac{38.2\ \check{AF}_H}{38.2\ \check{AF}_H + 11.3\ \check{F}_v}\right)^2 \beta^2_{AF_H} + \left(\frac{11.3\ \check{F}_v}{38.2\ \check{AF}_H + 11.3\ \check{F}_v}\right)^2 \beta^2_{F_v} \tag{5}$$

Using the fragility parameters obtained by Kennedy and the above equations the final results are summarized below for the fragility analysis.

Parameter	Median	β_r	β_u	β_c
Tu	3.34	-	0.16	0.16
AF_H	2.51	0.22	0.22	0.31
F_v	0.32	0.53	0.19	0.56
CAP	1.08g	0.21	0.26	0.33

Note that the capacity values differ slightly from Kennedy's results since he obtained the capacity values by direct simulation rather than the second moment method. The HCLPF capacities obtained by the two approaches are shown below.

Method	HCLPF	
CDFM	0.48	
FA	0.50	$1.08e^{-1.65(0.21+0.26)}$

Since the HCLPF values by the two approaches differ slightly the $\Sigma_{\gamma\beta}$ value must be adjusted before reconciling the results as follows:

$$1.08^{-\sum\gamma\beta} = 0.48 \text{ ----> } \sum\gamma\beta = 0.81$$

Table 1 gives the results of reconciling the CDFM and FA approaches. The first three columns give the basic parameters and the results Kennedy obtained in his analyses using the two approaches. The β_C values are obtained by the SRSS combination of the β_r and β_u values. The fourth column gives logarithmic standard deviations of the final capacity from each of the basic parameters using equation 5. Note that the three terms in the parentheses of equation 5 represent influence coefficients for the effects of the basic parameters on the variability of the Starting Air Tank capacity. Both Tu and AF_H (i.e., the angle capacity and horizontal earthquake effect, respectively) have potentially significant influence, but F_V (i.e., the vertical earthquake effect) has little influence.

The fifth column gives the γ values, which are the number of standard deviations (a measure of conservatism) assumed in the CDFM approach for each of the basic parameters. They come from the standard relationship for the lognormal distribution which relates the number of standard deviations which a value is away from the median (in a multiplicative sense) knowing the median and β (see equation 2).

The sixth column gives the $\gamma\beta_{CAP}$ value corresponding to each basic parameter using the results from columns four and five. Finally, the last column gives the probability level (which is a measure of the level of conservatism) assumed for each basic parameter used in the CDFM approach.

Based on the results in column six, most of the contribution to the conservatism in the HCLPF capacity using the CDFM approach comes from the angle capacity, Tu, and the horizontal response factor, AF_H. Almost none comes from the vertical earthquake component. This last result is to be expected since the vertical component has little potential influence on the tank capacity.

From the probability level results it is seen that the horizontal and vertical earthquake component values used in the CDFM analysis are only slightly conservative (0.50 would be median) while a very large factor of conservatism was introduced in the angle capacity value. Finally, Table 1 shows that the $\sum_{\gamma\beta}$ value calculated (i.e., 0.83) is close to the target value of 0.81. This slight difference is expected since the second moment approach used is approximate.

Table 1 Reconciliation Between CDFM and FA Approaches

Basic Parameter	CDFM Value	FA Parameters	β_{CAP}	γ	$\gamma\beta_{CAP}$ (fraction of total)	Prob. Level
				$\gamma = \frac{1}{0.16} \ln(\frac{3.34}{1.79})$		
Angle Capacity, Tu	1.79	Tu = 3.34 $\beta_C = 0.16$	(0.89)(0.16) = 0.14	$\gamma = 3.90$	(0.14)(3.90) = 0.55 (.66)	$\Phi(3.90) = 0.99995$
				$\gamma = \frac{1}{0.16} \ln(\frac{3.33}{2.51})$		
Horizontal Response Factor, AF_H	3.33	$AF_H = 2.51$ $\beta_C = 0.31$	(0.96)(0.31) = 0.3	$\gamma = 0.91$	(0.30)(0.91) = 0.27 (.33)	$\Phi(0.91) = 0.82$
				$\gamma = \frac{1}{0.56} \ln(\frac{0.44}{0.32})$		
Combined Vertical Response Factor, F_V	(1.11)(0.40) = 0.44	$F_V = 0.32$ $\beta_C = 0.56$	(0.40)(0.56) = 0.02	$\gamma = 0.57$	(0.02)(0.57) = 0.01 (.01)	$\Phi(0.57) = 0.72$
				Total	0.83	(compare to 0.81)

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APPENDIX D

This appendix gives listings of the response spectra used in the analysis

Pages	
2-4	Horizontal ground response spectra (3, 5, 7, and 10% damping)
5-7	Horizontal floor response spectra (2, 3.5, 4, 5, and 7% damping)
8-11	Averaged Horizontal floor response spectra (2, 3.5, 4, 5, and 7% damping)
12-15	Vertical floor response spectra (2, 3.5, 4, 5 and 7% damping)

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MCAD FILES GSPECTRA.CKH

This program reads in the natural logarithms of the median rock ground response spectra for 3%, 5%, and 7% and 10% damping and the corresponding natural logarithms of the frequency values. Note that there are N+1 data values for each array ranging from 0 to N. Each spectrum is fit with a cubic spline.

HORIZONTAL DIRECTION

$N := 40$

$i := 0 ..N$

Read in frequency values:

$XLFR3_i := READ(XLFR3)$ FMEDRSPC.3% contains the lns of the frequencies corresponding to the 3% Sa values

$XLFR5_i := READ(XLFR5)$ FMEDRSPC.5% contains the lns of the frequencies corresponding to the 5% Sa values

$XLFR7_i := READ(XLFR7)$ FMEDRSPC.7% contains the lns of the frequencies corresponding to the 7% Sa values

$XLFR10_i := READ(XLFR10)$ FMEDRSPC.10% contains the lns of the frequencies corresponding to the 10% Sa values

Read in spectral values:

$XSA3_i := READ(XSA3)$ MEDRSPC.3% contains the lns of the 3% Sa values

$XSA5_i := READ(XSA5)$ MEDRSPC.5% contains the lns of the 5% Sa values

$XSA7_i := READ(XSA7)$ MEDRSPC.7% contains the lns of the 7% Sa values

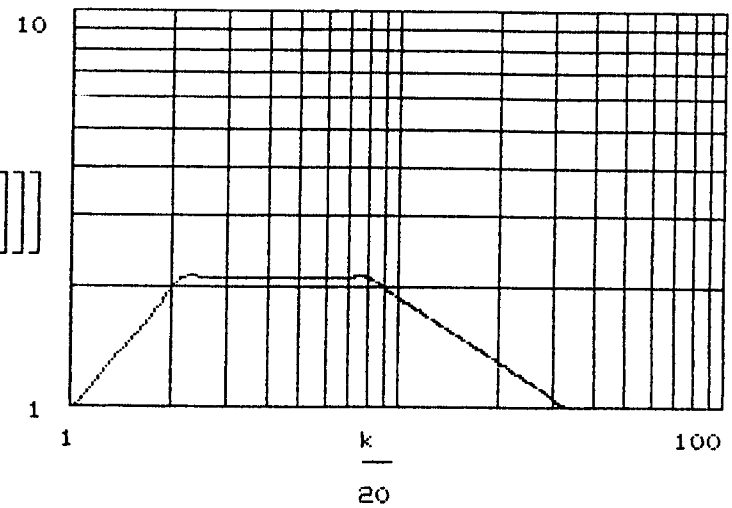
$XSA10_i := READ(XSA10)$ MEDRSPC.10% contains the lns of the 10% Sa values

s3 := cspline(XLFR3,XSA3)
s5 := cspline(XLFR5,XSA5)
s7 := cspline(XLFR7,XSA7)
s10 := cspline(XLFR10,XSA10)

$k := 1 ..1200$

Example 5% damped ground response spectrum scaled to 1.0g pga

$$\exp\left[\text{interp}\left[s5, XLFR5, XSA5, \ln\left[\frac{k}{20}\right]\right]\right]$$



$j := 0 ..40$

$FR3_j := \exp\left[XLFR3_j\right]$ $\quad SA3_j := \exp\left[XSA3_j\right]$

$FR5_j := \exp\left[XLFR5_j\right]$ $\quad SA5_j := \exp\left[XSA5_j\right]$

$FR7_j := \exp\left[XLFR7_j\right]$ $\quad SA7_j := \exp\left[XSA7_j\right]$

$FR10_j := \exp\left[XLFR10_j\right]$ $\quad SA10_j := \exp\left[XSA10_j\right]$

$FR3_j$	$SA3_j$	$FR5_j$	$SA5_j$	$FR7_j$	$SA7_j$	$FR10_j$	$SA10_j$
0.3	0.327	0.3	0.29	0.3	0.266	0.3	0.24
0.367	0.4	0.366	0.354	0.365	0.323	0.364	0.291
0.449	0.489	0.446	0.431	0.444	0.393	0.441	0.353
0.55	0.599	0.545	0.526	0.54	0.478	0.534	0.427
0.673	0.733	0.664	0.642	0.657	0.582	0.648	0.518
0.824	0.897	0.811	0.783	0.8	0.708	0.785	0.628
1.008	1.098	0.989	0.955	0.973	0.861	0.952	0.761
1.234	1.343	1.206	1.165	1.184	1.048	1.154	0.923
1.51	1.644	1.472	1.422	1.44	1.275	1.399	1.119
1.849	2.012	1.795	1.734	1.752	1.551	1.696	1.356
2.263	2.463	2.19	2.116	2.131	1.887	2.056	1.644
2.567	2.463	2.493	2.116	2.433	1.887	2.356	1.644
2.913	2.463	2.838	2.116	2.777	1.887	2.698	1.644
3.305	2.463	3.23	2.116	3.17	1.887	3.091	1.644
3.75	2.463	3.677	2.116	3.618	1.887	3.541	1.644
4.254	2.463	4.186	2.116	4.129	1.887	4.056	1.644
4.827	2.463	4.765	2.116	4.713	1.887	4.646	1.644
5.477	2.463	5.424	2.116	5.38	1.887	5.322	1.644
6.214	2.463	6.174	2.116	6.141	1.887	6.097	1.644
7.051	2.463	7.028	2.116	7.009	1.887	6.984	1.644
8	2.463	8	2.116	8	1.887	8	1.644
9.218	2.251	9.218	1.963	9.218	1.771	9.218	1.564
10.621	2.057	10.621	1.821	10.621	1.662	10.621	1.489
12.238	1.879	12.238	1.69	12.238	1.56	12.238	1.416
14.101	1.717	14.101	1.568	14.101	1.464	14.101	1.348
16.248	1.569	16.248	1.455	16.248	1.374	16.248	1.282
18.722	1.434	18.722	1.349	18.722	1.289	18.722	1.22
21.572	1.31	21.572	1.252	21.572	1.21	21.572	1.161
24.856	1.198	24.856	1.162	24.856	1.135	24.856	1.105
28.64	1.094	28.64	1.078	28.64	1.066	28.64	1.051
33	1	33	1	33	1	33	1
35.033	1	35.033	1	35.033	1	35.033	1
37.191	1	37.191	1	37.191	1	37.191	1
39.483	1	39.483	1	39.483	1	39.483	1
41.915	1	41.915	1	41.915	1	41.915	1
44.497	1	44.497	1	44.497	1	44.497	1
47.239	1	47.239	1	47.239	1	47.239	1
50.149	1	50.149	1	50.149	1	50.149	1
53.238	1	53.238	1	53.238	1	53.238	1
56.518	1	56.518	1	56.518	1	56.518	1
60	1	60	1	60	1	60	1

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MCAD FILES FSPECTRA.CKH

This program reads in the HORIZONTAL floor response spectra for 2%, 3.5%, 4%, 5% and 7% damping and the corresponding natural logarithms of the frequency values. Note that there are N+1 data values for each array ranging from 0 to N. Each spectrum is fit with a cubic spline.

HORIZONTAL DIRECTION

$N := 45$

$i := 0 \,..N$

$XLFR_i := READ(XLFR)$ HLFR.DAT contains the lns of the frequencies

$XSA2_i := READ(XSA2)$ HSA2.DAT contains the 2% Sa values

$XSA35_i := READ(XSA35)$ HSA35.DAT contains the 3.5% Sa values

$XSA4_i := READ(XSA4)$ HSA4.DAT contains the 4% Sa values

$XSA5_i := READ(XSA5)$ HSA5.DAT contains the 5% Sa values

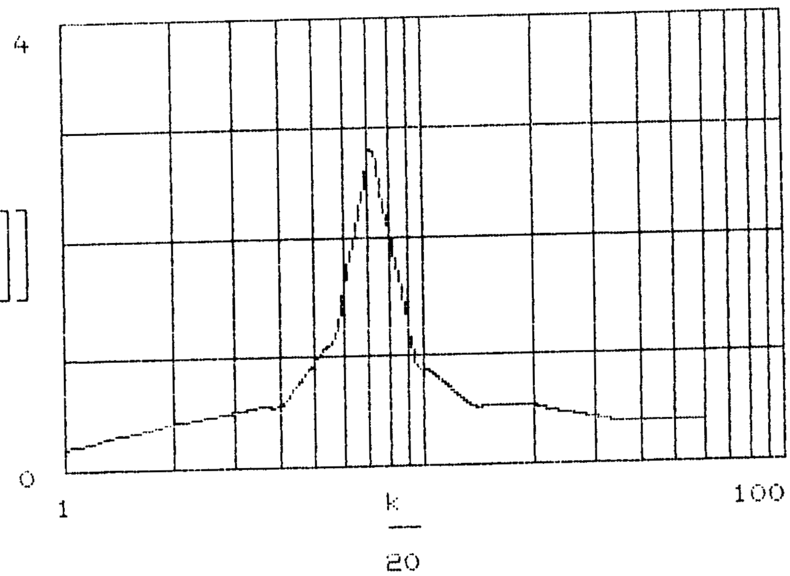
$XSA7_i := READ(XSA7)$ HSA7.DAT contains the 7% Sa values

```
s2 := cspline(XLFR,XSA2)
s35 := cspline(XLFR,XSA35)
s4 := cspline(XLFR,XSA4)
s5 := cspline(XLFR,XSA5)
s7 := cspline(XLFR,XSA7)
```

$k := 1 .. 1200$

Example plot of 3.5% floor response spectrum

$$\mathrm{interp}\left[s35, XLFR, XSA35, \ln\left[\frac{k}{20}\right]\right]$$



This plot took 12 seconds to compute

$j := 0 .. 45$

$$FR_j := \exp\left[XLFR_j\right]$$

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FR_j	$XSA2_j$	$XSA35_j$	$XSA4_j$	$XSA5_j$	$XSA7_j$
1	0.21	0.186	0.18	0.17	0.16
1.171	0.266	0.235	0.226	0.212	0.196
1.371	0.322	0.283	0.272	0.254	0.232
1.605	0.378	0.331	0.318	0.296	0.268
1.879	0.434	0.379	0.364	0.338	0.304
2.2	0.49	0.427	0.41	0.38	0.34
2.492	0.536	0.458	0.436	0.399	0.352
2.822	0.582	0.488	0.462	0.418	0.364
3.196	0.628	0.518	0.488	0.438	0.376
3.62	0.674	0.548	0.514	0.457	0.388
4.1	0.72	0.579	0.54	0.476	0.4
4.379	0.868	0.701	0.656	0.583	0.504
4.678	1.016	0.823	0.772	0.69	0.608
4.996	1.164	0.945	0.888	0.798	0.712
5.337	1.312	1.067	1.004	0.905	0.816
5.7	1.46	1.19	1.12	1.012	0.92
5.973	1.888	1.515	1.416	1.258	1.096
6.258	2.316	1.84	1.712	1.503	1.272
6.558	2.744	2.165	2.008	1.749	1.448
6.871	3.172	2.49	2.304	1.994	1.624
7.2	3.6	2.815	2.6	2.24	1.8
7.61	3.076	2.439	2.264	1.971	1.61
8.044	2.552	2.063	1.928	1.702	1.42
8.503	2.028	1.686	1.592	1.432	1.23
8.988	1.504	1.31	1.256	1.163	1.04
9.5	0.98	0.934	0.92	0.894	0.85
10.237	0.892	0.852	0.84	0.818	0.78
11.03	0.804	0.77	0.76	0.741	0.71
11.885	0.716	0.688	0.68	0.665	0.64
12.807	0.628	0.606	0.6	0.588	0.57
13.8	0.54	0.525	0.52	0.512	0.5
14.863	0.54	0.525	0.52	0.512	0.5
16.008	0.54	0.525	0.52	0.512	0.5
17.241	0.54	0.525	0.52	0.512	0.5
18.569	0.54	0.525	0.52	0.512	0.5
20	0.54	0.525	0.52	0.512	0.5
22.107	0.508	0.496	0.492	0.486	0.476
24.436	0.476	0.467	0.464	0.459	0.452
27.01	0.444	0.438	0.436	0.433	0.428
29.855	0.412	0.409	0.408	0.406	0.404
33	0.38	0.38	0.38	0.38	0.38
37.191	0.38	0.38	0.38	0.38	0.38
41.915	0.38	0.38	0.38	0.38	0.38
47.239	0.38	0.38	0.38	0.38	0.38
53.238	0.38	0.38	0.38	0.38	0.38
60	0.38	0.38	0.38	0.38	0.38

MCAD FILES AFHSPECT.AVE

This program reads in the averaged HORIZONTAL floor response spectra for 2%, 3.5%, 4%, 5%, and 7% damping (and the corresponding floor spectra) and the natural logarithms of the frequency values. Note that there are N + 1 data values for each array ranging from 0 to N. Each spectrum is fit with a cubic spline.

HORIZONTAL DIRECTION

$N := 45$

$i := 0 \,..N$

$XLFR_i := READ(XLFR)$	HLFR.DAT	contains the lns of the frequencies
$XSA2_i := READ(XSA2)$	HSA2.DAT	contains the 2% Sa values
$XSA35_i := READ(XSA35)$	HSA35.DAT	contains the 3.5% Sa values
$XSA4_i := READ(XSA4)$	HSA4.DAT	contains the 4% Sa values
$XSA5_i := READ(XSA5)$	HSA5.DAT	contains the 5% Sa values
$XSA7_i := READ(XSA7)$	HSA7.DAT	contains the 7% Sa values

$s35 := cspline(XLFR, XSA35)$

$ASA2_i := READ(ASA2)$	ASA2.DAT	contains the AVERAGED 2% Sa values
$ASA35_i := READ(ASA35)$	ASA35.DAT	contains the AVERAGED 3.5% Sa values
$ASA4_i := READ(ASA4)$	ASA4.DAT	contains the AVERAGED 4% Sa values
$ASA5_i := READ(ASA5)$	ASA5.DAT	contains the AVERAGED 5% Sa values
$ASA7_i := READ(ASA7)$	ASA7.DAT	contains the AVERAGED 7% Sa values

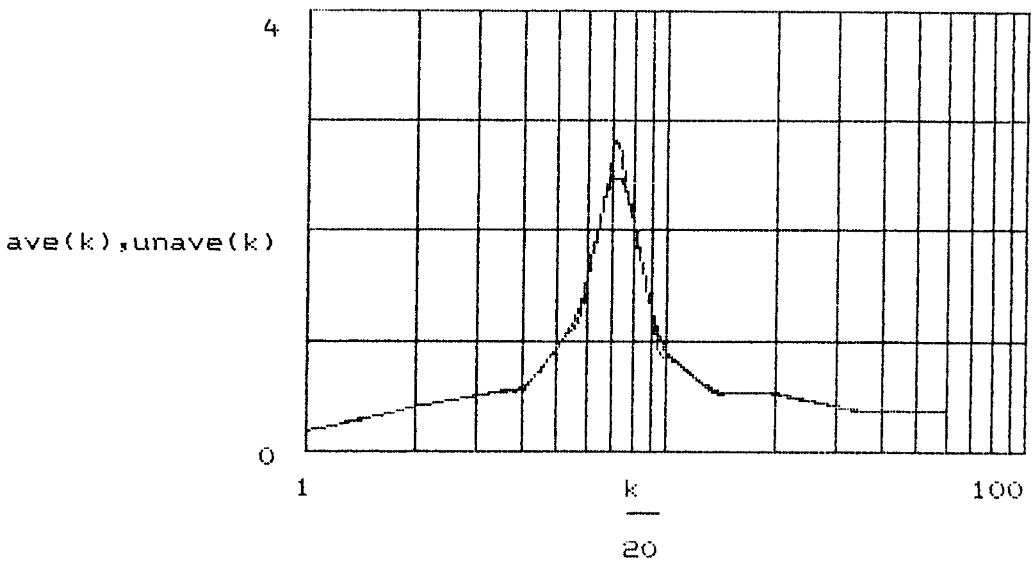
$a35 := cspline(XLFR, ASA35)$

$$unave(k) := interp\left[s35, XLFR, XSA35, \ln\left[\frac{k}{20}\right]\right]$$

$$ave(k) := interp\left[a35, XLFR, ASA35, \ln\left[\frac{k}{20}\right]\right]$$

$k := 1 \,..1200$

Example plot of 3.5% floor response spectra



$j := 0 \,..45$

$$FR_j := \exp\left[XLFR_j\right]$$

FR_j	$XSA2_j$	$ASA2_j$	$XSA35_j$	$ASA35_j$	$XSA4_j$	$ASA4_j$
1	0.21	0.209	0.186	0.186	0.18	0.179
1.171	0.266	0.265	0.235	0.234	0.226	0.225
1.371	0.322	0.321	0.283	0.282	0.272	0.272
1.605	0.378	0.377	0.331	0.33	0.318	0.317
1.879	0.434	0.433	0.379	0.379	0.364	0.364
2.2	0.49	0.49	0.427	0.426	0.41	0.408
2.492	0.536	0.535	0.458	0.458	0.436	0.436
2.822	0.582	0.581	0.488	0.487	0.462	0.461
3.196	0.628	0.629	0.518	0.519	0.488	0.489
3.62	0.674	0.667	0.548	0.543	0.514	0.509
4.1	0.72	0.754	0.579	0.608	0.54	0.569
4.379	0.868	0.867	0.701	0.7	0.656	0.656
4.678	1.016	1.012	0.823	0.82	0.772	0.769
4.996	1.164	1.159	0.945	0.941	0.888	0.884
5.337	1.312	1.314	1.067	1.069	1.004	1.005
5.7	1.46	1.614	1.19	1.303	1.12	1.221
5.973	1.888	1.917	1.515	1.536	1.416	1.435
6.258	2.316	2.298	1.84	1.826	1.712	1.699
6.558	2.744	2.737	2.165	2.159	2.008	2.003
6.871	3.172	3.063	2.49	2.409	2.304	2.231
7.2	3.6	3.154	2.815	2.485	2.6	2.302
7.61	3.076	3.007	2.439	2.387	2.264	2.217
8.044	2.552	2.579	2.063	2.082	1.928	1.946
8.503	2.028	2.038	1.686	1.693	1.592	1.598
8.988	1.504	1.542	1.31	1.337	1.256	1.28
9.5	0.98	1.178	0.934	1.071	0.92	1.04
10.237	0.892	0.898	0.852	0.857	0.84	0.845
11.03	0.804	0.805	0.77	0.771	0.76	0.761
11.885	0.716	0.718	0.688	0.69	0.68	0.682
12.807	0.628	0.629	0.606	0.608	0.6	0.601
13.8	0.54	0.567	0.525	0.55	0.52	0.545
14.863	0.54	0.541	0.525	0.525	0.52	0.521
16.008	0.54	0.54	0.525	0.524	0.52	0.52
17.241	0.54	0.54	0.525	0.525	0.52	0.52
18.569	0.54	0.54	0.525	0.525	0.52	0.52
20	0.54	0.534	0.525	0.519	0.52	0.514
22.107	0.508	0.509	0.496	0.497	0.492	0.493
24.436	0.476	0.476	0.467	0.467	0.464	0.464
27.01	0.444	0.445	0.438	0.438	0.436	0.437
29.855	0.412	0.412	0.409	0.409	0.408	0.408
33	0.38	0.386	0.38	0.386	0.38	0.386
37.191	0.38	0.379	0.38	0.379	0.38	0.379
41.915	0.38	0.38	0.38	0.38	0.38	0.38
47.239	0.38	0.38	0.38	0.38	0.38	0.38
53.238	0.38	0.38	0.38	0.38	0.38	0.38
60	0.38	0.38	0.38	0.38	0.38	0.38

FR_j	$XSA5_j$	$ASA5_j$	$XSA7_j$	$ASA7_j$
1	0.17	0.169	0.16	0.16
1.171	0.212	0.212	0.196	0.196
1.371	0.254	0.254	0.232	0.232
1.605	0.296	0.295	0.268	0.268
1.879	0.338	0.338	0.304	0.304
2.2	0.38	0.378	0.34	0.338
2.492	0.399	0.399	0.352	0.352
2.822	0.418	0.418	0.364	0.363
3.196	0.438	0.439	0.376	0.377
3.62	0.457	0.452	0.388	0.383
4.1	0.476	0.503	0.4	0.428
4.379	0.583	0.583	0.504	0.504
4.678	0.69	0.687	0.608	0.605
4.996	0.798	0.794	0.712	0.709
5.337	0.905	0.905	0.816	0.815
5.7	1.012	1.092	0.92	0.967
5.973	1.258	1.272	1.096	1.104
6.258	1.503	1.493	1.272	1.265
6.558	1.749	1.744	1.448	1.445
6.871	1.994	1.935	1.624	1.581
7.2	2.24	1.997	1.8	1.627
7.61	1.971	1.933	1.61	1.583
8.044	1.702	1.716	1.42	1.43
8.503	1.432	1.438	1.23	1.234
8.988	1.163	1.182	1.04	1.053
9.5	0.894	0.987	0.85	0.911
10.237	0.818	0.821	0.78	0.783
11.03	0.741	0.742	0.71	0.711
11.885	0.665	0.667	0.64	0.642
12.807	0.588	0.59	0.57	0.571
13.8	0.512	0.536	0.5	0.522
14.863	0.512	0.513	0.5	0.501
16.008	0.512	0.512	0.5	0.5
17.241	0.512	0.512	0.5	0.5
18.569	0.512	0.512	0.5	0.5
20	0.512	0.507	0.5	0.495
22.107	0.486	0.487	0.476	0.477
24.436	0.459	0.459	0.452	0.452
27.01	0.433	0.433	0.428	0.429
29.855	0.406	0.406	0.404	0.404
33	0.38	0.385	0.38	0.385
37.191	0.38	0.379	0.38	0.379
41.915	0.38	0.38	0.38	0.38
47.239	0.38	0.38	0.38	0.38
53.238	0.38	0.38	0.38	0.38
60	0.38	0.38	0.38	0.38

MCAD FILES FSPECTRA.CKV

This program reads in the VERTICAL floor response spectra for 2%, 3.5%, 4%, 5% and 7% damping and the corresponding natural logarithms of the frequency values. Note that there are N+1 data values for each array ranging from 0 to N. Each spectrum is fit with a cubic spline.

VERTICAL DIRECTION

$N := 70$

$i := 0 \,..N$

$XLFR_i := READ(XLFR)$ VLFR.DAT contains the lns of the frequencies

$XSA2_i := READ(XSA2)$ VSA2.DAT contains the 2% Sa values

$XSA35_i := READ(XSA35)$ VSA35.DAT contains the 3.5% Sa values

$XSA4_i := READ(XSA4)$ VSA4.DAT contains the 4% Sa values

$XSA5_i := READ(XSA5)$ VSA5.DAT contains the 5% Sa values

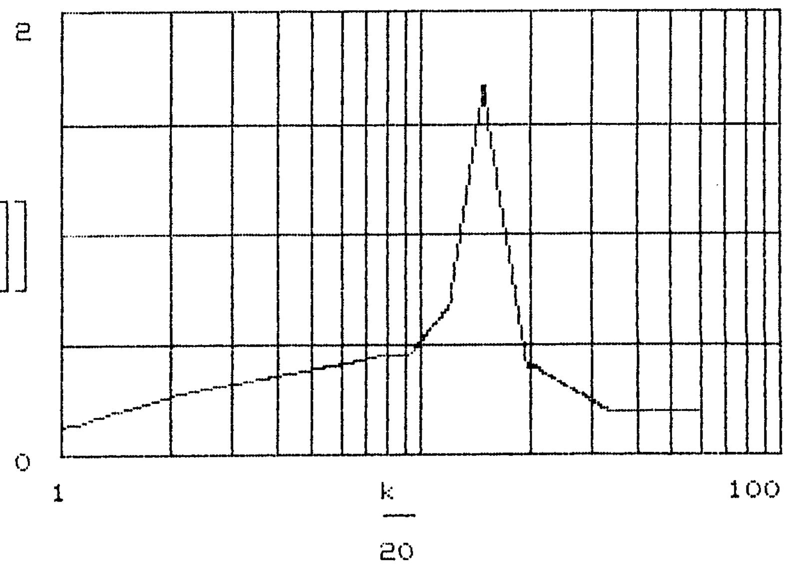
$XSA7_i := READ(XSA7)$ VSA7.DAT contains the 7% Sa values

```
s2 := cspline(XLFR,XSA2)
s35 := cspline(XLFR,XSA35)
s4 := cspline(XLFR,XSA4)
s5 := cspline(XLFR,XSA5)
s7 := cspline(XLFR,XSA7)
```

$k := 1 .. 1200$

Example plot of 3.5% floor response spectrum

$\mathrm{interp}\left[s35, XLFR, XSA35, \ln\left[\frac{k}{20}\right]\right]$



$j := 0 .. 50$

$FR_j := \exp\left[XLFR_j\right]$

FR j	XSA2 j	XSA35 j	XSA4 j	XSA5 j	XSA7 j
1	0.14	0.124	0.12	0.114	0.11
1.082	0.159	0.14	0.135	0.128	0.122
1.171	0.178	0.156	0.15	0.141	0.134
1.267	0.197	0.172	0.165	0.155	0.146
1.371	0.216	0.187	0.18	0.168	0.158
1.483	0.235	0.203	0.195	0.182	0.17
1.605	0.254	0.219	0.21	0.196	0.182
1.737	0.273	0.235	0.225	0.209	0.194
1.879	0.292	0.251	0.24	0.223	0.206
2.033	0.311	0.267	0.255	0.236	0.218
2.2	0.33	0.283	0.27	0.25	0.23
2.549	0.357	0.302	0.287	0.263	0.239
2.954	0.384	0.321	0.304	0.277	0.248
3.423	0.411	0.34	0.321	0.29	0.257
3.966	0.438	0.359	0.338	0.304	0.266
4.596	0.465	0.378	0.355	0.317	0.275
5.325	0.492	0.397	0.372	0.33	0.284
6.17	0.519	0.417	0.389	0.344	0.293
7.15	0.546	0.436	0.406	0.357	0.302
8.285	0.573	0.455	0.423	0.371	0.311
9.6	0.6	0.474	0.44	0.384	0.32
9.817	0.62	0.496	0.462	0.406	0.34
10.038	0.64	0.518	0.484	0.428	0.36
10.265	0.66	0.539	0.506	0.45	0.38
10.496	0.68	0.561	0.528	0.472	0.4
10.733	0.7	0.583	0.55	0.494	0.42
10.975	0.72	0.604	0.572	0.516	0.44
11.223	0.74	0.626	0.594	0.538	0.46
11.476	0.76	0.648	0.616	0.56	0.48
11.735	0.78	0.67	0.638	0.582	0.5
12	0.8	0.692	0.66	0.604	0.52
12.263	0.95	0.789	0.744	0.667	0.566
12.531	1.1	0.886	0.828	0.73	0.612
12.805	1.25	0.984	0.912	0.794	0.658
13.085	1.4	1.081	0.996	0.857	0.704
13.372	1.55	1.179	1.08	0.92	0.75
13.664	1.7	1.276	1.164	0.983	0.796
13.963	1.85	1.374	1.248	1.046	0.842
14.269	2	1.471	1.332	1.11	0.888
14.581	2.15	1.569	1.416	1.173	0.934
14.9	2.3	1.666	1.5	1.236	0.98
15.314	2.114	1.542	1.392	1.154	0.922
15.74	1.928	1.418	1.284	1.071	0.864
16.177	1.742	1.294	1.176	0.989	0.806
16.627	1.556	1.169	1.068	0.906	0.748
17.089	1.37	1.045	0.96	0.824	0.69
17.564	1.184	0.921	0.852	0.742	0.632
18.052	0.998	0.797	0.744	0.659	0.574
18.554	0.812	0.673	0.636	0.577	0.516
19.07	0.626	0.549	0.528	0.494	0.458
19.6	0.44	0.425	0.42	0.412	0.4

j := 51 ..70

FR_j	$XSA2_j$	$XSA35_j$	$XSA4_j$	$XSA5_j$	$XSA7_j$
20.648	0.416	0.402	0.398	0.391	0.38
21.752	0.392	0.38	0.376	0.37	0.36
22.916	0.368	0.357	0.354	0.348	0.34
24.141	0.344	0.335	0.332	0.327	0.32
25.432	0.32	0.312	0.31	0.306	0.3
26.792	0.296	0.29	0.288	0.285	0.28
28.225	0.272	0.267	0.266	0.264	0.26
29.735	0.248	0.245	0.244	0.242	0.24
31.325	0.224	0.222	0.222	0.221	0.22
33	0.2	0.2	0.2	0.2	0.2
35.033	0.2	0.2	0.2	0.2	0.2
37.191	0.2	0.2	0.2	0.2	0.2
39.483	0.2	0.2	0.2	0.2	0.2
41.915	0.2	0.2	0.2	0.2	0.2
44.497	0.2	0.2	0.2	0.2	0.2
47.239	0.2	0.2	0.2	0.2	0.2
50.149	0.2	0.2	0.2	0.2	0.2
53.238	0.2	0.2	0.2	0.2	0.2
56.518	0.2	0.2	0.2	0.2	0.2
60	0.2	0.2	0.2	0.2	0.2

APPENDIX D

JOHN W. STEVENSON

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0046G
Rev. 1 1/15/88
Rev. 2 2/15/88

Report on Development of High Confidence Low Probability of Failure, HCLPF, Based on the Conservative Deterministic Failure Margin, CDFM Method for Selected Mechanical and Electrical Equipment

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1.0 INTRODUCTION

In this report High Confidence Low Probability of Failure, HCLPF's estimates based on the Conservative Deterministic Failure Margin, CDFM method are developed for the following mechanical and electrical components:

(1) Refueling Water Storage Tank (Vertical Mounted Flat Bottom Tank) Located on the ground assuming no soil structure interaction.

(2) Diesel Starting Air Tank (Vertical Clip Angle Mounted Tank) Located on an Auxiliary Building Upper Floor

(3) Generic Motor Control Centers (Low Voltage) As Contained in the EPRI RP. No.-5223 pages D-34 to D-39
    (a) Located on the ground assuming no soil structure interaction
    (b) Located at an Auxiliary Building Upper Floor

(4) Component Cooling Heat Exchanger (Horizontal HX on Two Saddles) Located on an Auxiliary Building Upper Floor

(5) Sample Block Wall
Located on an Auxiliary Building Upper Floor

for which descriptions were transmitted by LLNL letter dated 4 August 1987.

For each of these five components, the limiting failure mode(s) are identified and initially the HCLPF seismic load capacity computed using the procedures contained in References 1 and 2 which are attached hereto as Appendix A. Subsequently HCLPF's were evaluated using the HCLPF estimating procedures recommended in Table 2.5 of the draft EPRI Margin Report (3) shown in Appendix B to this report. The use of the Appendix B procedure, only significantly effects the HCLPF estimation for component 3b.

The seismic load capacity for each component is initially expressed in terms of the limiting resultant seismic inertia load applied to the center of gravity of the components assuming the simultaneous application of two horizontal and one vertical components of earthquake. It is further assumed that resultant forces and moments developed from each direction of earthquake are combined on a SRSS basis and this resultant is combined absolutely with normal operating stresses to determine the limiting capacities.

The seismic spectral capacities are converted to HCLPF's expressed in peak floor and peak ground acceleration values using the spectral[1] and information contained in Appendix C.

In addition to the HCLPF estimates presented herein best estimates (50 percent confidence and 50 percent probability of failure) of failure are presented in Section 8.0 of this report.

2.0 EVALUATION OF THE VERTICAL MOUNTED TANK

2.1 Tank Geometry and Loading Data

2.1.1 Tank Geometry

The tank overall geometry is shown in Figure 1a. Other geometric data are summarized as follows:

Anchor Bolts - 8-2"ø spaced at 45° (typ.), 4'-0 Long

Anchor bolt Chairs - Details of the modified anchor bolt chairs are shown in Figure 1b

2.1.2 Materials

Tank Shell, head and lugs are A240-304 Stainless Steel

Anchor bolts - ASTM A307 Ferretic Steel

2.1.3 Design Data

Operating pressure-0.25 psig
Operating temperature - Ambient
Design pressure-0.5 psig, 2" H_2O vacuum
Design temperature-200°F
Weight-76,500 lb empty, 2,385,000 lbs full[2]
Capacity-375,600 gal. full, 336,000 gal. usable.
Design Depth of Water, 37.0 ft.

2.2 Tank Analysis

The first activity in determining the CDFM for the ground supported tank is to identify the lower bound failure mode(s) which are to be evaluated. Past experience in the design and evaluation of similar vertical tanks suggest the following two lower bound failure modes:

---

[1] It is noted that the Spectra provided are median spectra from NUREG/CR-0098 while the CDFM in accordance with Refs. 1 and 2 uses 84 percent NEP Spectra. For the purposes of this study the spectra provide in Appendix C will be considered as 84 percentile NEP spectra.

[2] The weight full appears low, however this weight was assumed in this calculation since it is the same as the weight used by the panel throughout this study.

(1) Tension failure of anchor bolt or anchor bolt chairs

(2) Compression (buckling) failure of tank wall

Following either of these two analytically determined failure modes, a separation between the bottom and walls of the tank would be expected which would result in gross leakage of the tank.

The overturning moment for a nominal spectral acceleration of 1.0g using the weight and geometry data shown in Figure 1a is determined as follows:

Overall Seismic Loads:

V_1 = Base Shear
= $(W_1 + W_{ss} + W_{sr})$ Sa

M_1 = Base overturning moment
= $(W_1X_1 + W_{ss}$ X ss $+ W_{sr}X_{sr})$ Sa

W_1 = Effective impulsive fluid weight
= 2217^K

X_1 = Distance from tank Base to centroid of effective impulsive fluid
= 14.74 ft.

Wss = Weight of tank shell
46.6^k

Xss = Distance from tank base to centroid of tank shell
= 19 ft.

Wsr = Weight of tank roof
= 17.1^K

Xsr = Distance from tank base to centroid of tank roof
= 38 ft.

Base Shear for a Nominal 1.0g Lateral Acceleration

V_1 = (2217 + 46.6 + 17.1) X 1.0
= 2280 Kips

Overturning moment for a Nominal 1.0g Lateral Acceleration

M_1 = [2217 (14.74) + 46.6(19) + 17.1(38)] X 1.0 x 12 in/ft

= 410,582 K - in

2.2.1 Evaluation of Anchor Bolt and Chair Capacities

Bolt Capacity:

Actual yield strength capacity, F_y of 2" ø A307 anchor bolts is determined

$$F_y = S_y \times AYf \times A_{bt}$$

where:

S_y = Specified yield of material = 36 Ksi
AYf = Actual yield coefficient = 1.05 (assumed)
A_{bt} = Area of threaded bolt

$$F_y = 36 \text{ Ksi} \times 1.05 \times 2.5 \text{ in}^2 = 94.5 \text{ Kips}$$

Chair Capacity:

In defining this capacity the bolt chair capacity calculations performed in Reference 4 have been considered. The Reference 4 Calculation show the capacity of the anchor bolt chairs to be 67.7 Kips each. However, I believe the limiting value of 67.7 K/bolt chair can be increased by 10 percent to account for anticipated actual material properties of austenitic steel compared to the specified minimum yield properties.

$$67.7 \times 1.10 = 74.5 \text{ Kip/bolt}$$

Anchor capacities are therefore limited by chairs behavior before modification.

Subsequent to the Reference 4 analysis, the bolt chairs on the tank were modified to increase their capacities to 120 Kips each as determine by Kennedy[5]. This increase in bolt chair capacity now means the bolt capacities of 94.5 Kips each now control anchorage design of the tank.

Overturning Moment Capacity of Bolts Based on Assumed Distribution Shown in Figure 2.

$$M_u = F_y \text{ bolt (lever arm to N.A.)}$$

$$M_u = 94.5 \text{ k} \times (2 \times 46.3 + 2 \times 216 + 2 \times 385.7 + 1 \times 456)$$

$$M_u = 94.5 \times (92.6 + 432 + 771.4 + 456)$$

$$M_u = 94.5 \times 1752 \text{ in}$$

$$= 165{,}564 \text{ K - in}$$

2.2.2 Buckling Capacity of Shell Due to Overturning Earthquake Moment

Using the simple equation from Rourk[6] Table 35 (16), the critical buckling moment for the tank is determined.

$$M_{CR} = \frac{0.72 \text{ E } rt^2}{(1-v^2)} \quad (1)$$

$$M_{CR} = \frac{.72 \times 27{,}000 \times 240 \times (3/8)^2}{0.91}$$

$$M_{CR} = 720{,}989 \text{ K - in}$$

Obviously the overturning capacity is significantly larger than the bolt overturning capacity.

2.2.3 Overturning Moment Mobilized by Weight of Fluid

The moment mobilized to overcome dead weight of tank and liquid contents acts to counter the seismic overturning moment. In the limit, this dead weight moment is limited by the plastic moment capacity and membrane action of the base plate carrying the weight of water which lifts off the base as shown in Figure 3.

The elastic section modulus, Z of the 1/4" base plate:

$Z = bh^2/6 = 12" \times (1/4)^2/6 = 0.125\ in^3/ft$ of plate
S_y = 30 Ksi; S_u = 75 Ksi; for SA 240 Type 304SS. from Tables I-2.2 and I-3.2. ASME Section III, Div. 1

Elastic moment capacity of base plate:

$M_e = ZS_y = .125 \times 30 = 3.75$ k-in/ft of plate

Plastic moment capacity of Plate

$M_p = 1.5 \times 3.75 = 5.63$ k-in/ft of plate

Plastic Moment Capacity of Uniformly load fixed end beams:

$M_p = 1/24\ wl^2$

where:

w = pressure weight of water

$w = 62.4\ lb/ft^3\ 37.0\ ft = 2308.8\ lbs/ft^2$

$w = 2308.8/12 = 192.4$ lbs/in/ft. of wall

$M = wl \times l$

$5.63\ k\text{-}in = .192 \times l^2$

$l^2 = 29.32\ in^2$

$l = 5.42\ in$

T_e = Holdown per inch of wall:

$T_e = 192/12 \times 5.42 = 86.7$ Lbs/in of wall

Assume the membrane action of the plate increases the tensile capacity of the tank wall by 100 percent.

$T_e = 173.4$ lb/in of wall

Total Tension in Tank Wall

173.4 x 480 in x π = 261.48 Kips

Assume Centroid of semi-circle is 2/5 from centerline

240 x 2/5 = 96 + 216 = 312 in

261.4 x 312" = 81557 K-in

(0.85 x 261.4) x (216 - 96) = 26663 K-in

Total restoring dead weight moment

81557 + 26663 = 108220 K-in

Total O.T. moment capacity of the tank is the moment capacity of the bolts plus the stablizing effect of the weight of the fluid in the lift off region of the shell.

165,564 + 108220 = 273784 K-in

(273784/410582) x 1.0g = 0.667g lateral load capacity

2.2.4 Determine Fundamental Impulse Frequency

Fundamental Impulse Frequency:(4)

The horizontal impulsive response fundamental frequency is estimated using the Haroun & Housner (H & H) method given in Ref. 7. The H & H coefficients are used since they are available for a variety of h/R valves.

The H & H frequency coefficients are developed for steel tanks of constant thickness filled with water. For a tank of varying thickness, the frequency coefficient can be selected using an average thickness. A reasonable estimate of the effective tank thickness is 3/16" since this thickness is used over the top 60% of the tank shell where deformations and hydrodynamic pressures are the greatest.

h = Shell thickness
= 3/16"

H = Fluid Height
= 37' - 0
= 444"

R = Tank radius
= 20'-0"
= 240"

d = Shell density

$$= \frac{490 \text{ lbs/ft}^3}{1728(386.4)}$$

$= 0.000734 \text{ lb-sec}^2/\text{in}^4$

E = Shell modulus of elasticity
= 28×10^6 psi

$\frac{H}{R} = \frac{444}{240}$
= 1.85

$\frac{h}{R} = \frac{0.1875}{240} = 0.00078$

$\omega_f H \quad 1/(d/E) \simeq 0.08$ (Fig. 5 of Ref. 7)

Circular natural frequency:

$$\omega_f = \frac{0.08}{444\sqrt{(0.000734/28 \times 10^6)}}$$

$$2\pi f = 35.2 = \omega_f$$

$$f = 5.6 \text{ Hz}$$

2.2.5 HCLPF Capacity of Tank

Given a fundamental impulse frequence of 5.6 Hz for the tank and an assumed damping of 7 percent and neglecting sloshing gives an amplication factor to the ground from the spectrum given in Appendix C.

0.88/.38 = 2.31

0.667/2.31 = .29g pga based on elastic response

and

Given that yielding in the bolt chairs would provide a small amount of additional global ductility in the resonant region of the tank, assume elastic response demand would be reduced by 10 percent.

This results in

0.29g + 0.03g = .32g pga HCLPF
for the Appendix C spectra.

If the NUREG/CR 0098 Median Spectra for 7 percent damping is used in place of the Appendix B spectra and the tank is assumed ground mounted

0.667/1.85 = 0.36 pga HCLPF

3.0 EVALUATION OF VERTICAL ANGLE CLIP MOUNTED TANK

Given a vertical tank arrangement as shown in Figure 4. By inspection it appears that the limiting seismic capacity of the tank will be in the clip angle anchorage of the tank's mounting ring.

3.1 Determine Maximum Uplift Force on An Anchor

Given the total weight of tank and mounting ring of tank shown in Figure 4 is 920 lbs. The distance to the c.g. of the tank from the base is 41.5 in. Total overturning seismic moment for a nominal 1.0g lateral load is

$$920 \times 41.5 = 38180 \text{ in-lbs.}$$

less the restraining force of dead weight of the tank. Given that the vertical upward component of the earthquake is 2/3 of horizontal only 1/3 x 920 = 307 lbs. acting down at the center of the tank would be available to offset the lateral overturning moment.

Taking moments about the assumed neutral axis of the tank which is taken as 2.4 inches from the edge of the mounting rings and neglecting the two inner clip angle supports as shown in Figure 4.

Maximum Tension in Clip Angle

$$[38180 - (307 \times 10.1)] / 22.6$$
$$(38180 - 3101) / 22.6 = 1552 \text{ lbs}$$

for a nominal 1.0g lateral load.

3.2 Determine the Clip Angle Anchor Capacity

Given a A-36 angle and 3/4" diameter bolt hole as shown in Figure 5, since the angle leg is welded on three sides, it can be assumed that vertical leg is fixed.

Maximum Moment in the Angle at the Bolt Hole

$$M = \frac{TL}{2}$$

$$= \frac{1552(1.625)}{2}$$

$$= 1261 \text{ lbs}$$

Angle Section Modulus

$$Z = \frac{2.25\ (0.25)^2}{6} = 0.0234 \text{ in}^3$$

$$S = \frac{1261}{0.0234} = 53889 \text{ psi}$$

The equivalent elastic moment stress capacity would be computed as follows:

$$S_e = SF \times F_y \times AYC$$

where:

SF = shape factor = 1.5

F_y = Specified Minimum Yield = 36 Ksi

AYC = Actual Yield Coefficient = 1.05

S_e = 1.5 x 36000 x 1.05 = 56.7 Ksi

Given that the maximum computed stress in the angle is 53889 psi from a net uplift of 1552 lbs for an earthquake lateral acceleration of 1.0g. Therefore the seismic capacity of the clip angle is 56.7/53.9 = 1.05g.

3.3 Determine Fundamental Frequency of Air Tank

$$I = \frac{\pi}{4}(12^4 - 11.625^4) = 1942.3 \text{ in}^4$$

E = 29,000,000 psi

Total weight of tank and support collar = 920 lbs.
Unit wieght w = 921/91 = 10.11 lbs/in

Assume Distributed Mass

From Ref. 6 - Table 36 (3)

$$f = \frac{1}{2\pi} \sqrt{\frac{12.4 \text{ E Ig}}{wL^4}}$$

$$f = 1/6.28 \sqrt{\frac{12.4 \times 29000000 \times 1942.3 \times 386}{10.11 \times 91^4}} \cong 99.3 \text{ Hz}$$

Estimate Stiffness Considering Only Clip Angle as Support Restraint

$$I = Ad^2 = 0.5 \times 12^2 \times 3 = 216 \text{ in}^4$$

This results in a factor of approximately 9 decrease in stiffness and therefore approximately 3 in frequency. However, frequency still should be above 30 Hz.

Therefore there should be no amplification of floor acceleration as a function of frequency.

3.4 Estimate of HCLPF Capacity of the Air Tank

The tank as evaluated considers only one horizontal component of earthquake. For rotationally symmetric components the second horizontal component would not change the component's earthquake capacity. For rectangular components in the limit the lateral capacity would be reduced in the ratio of 1/1.41. For this component which is only slightly rotationally unsymmetric considering the location of the clip angles in Figure 4 use a ratio of 1/1.2. The lateral load capacity of the tank is 1.05/1.2 = 0.88g pfa HCLPF using the amplification in going from ground to floor acceleration for the Appendix C spectrum yields.

0.88 x .18/.38 = 0.42 pga HCLPF

Yielding in the small clip angles would not add significantly to inelastic response tank.

4.0 EVALUATION OF GENERIC MOTOR CONTROL CENTER

To make this evaluation the GERS curve for MCC functioning during the E.Q. shown in Figure 1 page D-38 of the EPRI NP-5223 Report as shown in Appendix D was compared to the Appendix C spectra for 5 percent damping. The GERS shows a spectra acceleration value of 1.5g. The RRS from Appendix C shows a peak spectra acceleration of 2.2g for 5 percent damping. Using the 0.8 demand factor from Table 2.5 from Appendix B; 2.2 x 0.8 = 1.76g. The resultant HCLPF for the Appendix C spectrum:

$$1.5/1.76 \times .38 = 0.32\text{g pfa HCLPF}$$

$$0.32 \times .18/.38 = .15\text{g pga HCLPF}$$

for the NUREG/0098 median spectrum at 5 percent damping

$$1.5/2.1 = .71\text{g pga}$$

5.0 EVALUATION OF HORIZONTAL HX

Given the horizontal heat exchanger shown in Figure 6, it is assumed the shear load on the bolts at the fixed saddle will control the seismic load capacity of the HX. Given a total weight of 23.5 Kips and a nominal lateral load coefficient of 1.0g applied to the cg. of the HX the bolt shear reaction in the longitudinal direction would be 23.5/2 Kips or 11.75 Kips/bolt and in the transverse direction 23.5/4 or 5.88K/bolt Kips load in each bolt.

The component shear stress in each bolt is determined:

Longitudinal = 11.75/0.601 = 19.55 Ksi; Transverse = 5.88/0.601 = 9.78 Ksi

Allowable bolt stress from ASME Section III-NF-1986 for Service Level D

$$F_v \leq .42\, S_u < .6\, S_y$$

where:

S_u = Specified Minimum Ultimate Strength = 60 Ksi
S_y = Specified Minimum Yield Strength = (not defined) assume 36 Ksi
F_v = 0.42 x 60 = 25.2 Ksi allowable
F_v = 0.6 x 36 = 21.6 Ksi allowable (controls)

Resultant Shear Stress in Bolt for nominal 1.0g lateral load

$$f_v = \sqrt{(21.21)^2 + (9.78)^2} = \sqrt{545.51}$$

Resultant Shear Stress:

$$f_v = 23.36\text{ Ksi}$$

It should be noted that the current LRFD AISC Specification limits the allowable shear stress in the A307 bolt based on the nominal area 0.601 in^2 to 16.1 Ksi for factored loads and the AISC N690 AISC Specification limits shear on nominal A307 bolt areas to 14.0 Ksi for a large earthquake loading. Since the allowable values from the AISC specification are significantly lower than the ASME, the ASME code value is used without any adjustment for actual versus specified minimum properties.

Resultant Applied Seismic Load:

$$21.6/23.36 \times 1.0g = 0.925g$$

5.1 <u>Compute Limiting Longitudinal Frequency of the Tank</u>

$f = 1/2\pi \sqrt{K/M}$ WT = 23.50 Kips

Mass = $23.5k/386\text{-}in/sec^2 = 0.0608\ K\text{-}sec^2/in$

weight = 23.50 K

Horizontal Stiffness, - one saddle is slotted and free to slide, the other saddle is similar to a pin connection; therefore the stiffness is the same as a load at the end of a cantilever beam

Saddle Prop. $A_S = 6.625\ in^2$

$I_y = 66\ in^4$

$l = 13.12\ in$

$K = 3EI/l^3$

$K_{Horz} = 3(30{,}000\ ksi)(66 in^4)/(13.12in)^3$

$= 2630\ K/in$

$$f = 1/2\pi \sqrt{2630\ k/in/0.0608\ k.sec^2/in} = 33.3\ Hz$$

5.2 <u>Computation of HCLPF for the Horizontal HX</u>

Given that the HX has frequency response above 33 Hz for the 7 percent damping and the spectrum contained in Appendix C is applicable:

0.93g pfa HCLPF

0.93 x .18/.38 = .44g pga HCLPF

Since shear failures tend to be non ductile, it is assumed that the component responds elastically.

6.0 EVALUATION OF BLOCK WALL

Assume a 12" thick reinforced concrete block wall 10 feet high can be analyzed as a one way vertical cantilever slab. Wall is reinforced by one layer of #5 bar @ 16" center to center located in the center of the block with $f_y = 60$ ksi Assume compressive strength $f'_c = 3000$ psi for Type S mortar.

From Table 4.3 of ACI 531-79 for 12" x 16" block

w = 111 psf assuming grouted core
f_m = 1700 psi masonry compressive strength
E_m = 1000 f_m = 1700,000 psi, As = 0.31 x 12/16 = 0.233 in^2/ft
Kd≃ 1.5in; d = 11.625/2 = 5.8 in; jd = 5.05 in
l = 10 ft.

6.1 Compute Bending Moment for a Nominal 1.0g Lateral Load and Resultant Concrete and Steel Stresses

$$M = 1/2\ wl^2 = 1/2 \times 111 \times 10^2 \times 12 \text{ in/ft} = 66600 \text{ in-lbs/ft}$$

$$M = A_s f_s jd = 0.233 \times 5.05\ f_s = 66600 \text{ in-lbs}$$

Tensile Stress in Reinforcement due to bending

f_s = 56.60 Ksi
f_s all = 60 Ksi
f_s/f_sall = .94

Compressive Stress in concrete due to bending

C = M/jd = 66600/5.05 = 13188 lbs
f_c = 13188/(12 x 1.5)= 733 psi
$f_c$all. = 0.72 x 1700 = 1224 psi
f_c/f_call = .60

Tensile Stress in reinforcement controls design

Note: ACI 531-79 Coupled with SRP 3.8.4 Appendix A would give a slightly larger value for f_c all.

6.2 Compute Frequency of Wall

Uncracked Moment of Inertia

$$I = 1/12 \times 16 \times (11.625)^3 - 1/12 \times 11.8 \times (8.63)^3$$

$$I = 1.33 \times 1571 - 632$$

$$= 2089 - 632 = 1457$$

$$I = 1457/1.33 = 1095 \text{ in}^3\text{/ft of wall}$$

$$E_m = 1,700,000$$

From Ref. 6.

$$f = 3.52/2\pi \sqrt{EIg/wl^4}$$

$$= 3.52/6.28 \sqrt{[1700000 \times 1095 \times 386/9.25 \text{ lbs/in} \times (120)^4 \text{ in}^4]}$$

$= 3.52/6.28 \sqrt{374.6} = 10.8$ Hz[1]

6.3 Computation of HCLPF for Block Wall

Acceleration at c.g. of wall

1.0/.94 = 1.06

Spectral Amplification for 10.8 Hz from Appendix C spectrum

.75/.38 = 1.97

Use 0.8 x spectral demand to account for inelastic response 0.8 x 1.97 = 1.58

1.06/1.58 = 0.67g pfa HCLPF

0.67 x .18/.38 = 0.32g pga HCLPF

Note: In reinforced concrete shear walls changes in stiffness by a factor of 15 or more have been observed from first cracking to failure. If this same shift in stiffness before failure were to occur in the block wall, the frequency would be reduced to ≈ 3.0 Hz. At this frequency for the Appendix C Spectrum, the spectral amplification would be reduced to 1.0 and the effective pga HCLPF would be increased 0.32 x 1.97 = 0.63 g

7.0 ESTIMATION OF BEST ESTIMATE RATHER THAN HCLPF Pga's

In performing this evaluation, a computer program FRAGIL was used. This program is designed to take estimates of seismic capacities (HCLPF or otherwise) and apply estimated variability on both failure probability as well as confidence as shown in Table 1 to determine the probability of failure at the 95, 50 and 5 percent probability and at the 95, 50, and 5 percent confidence levels. In this evaluation variability as to probability of failure and confidence were estimated as coefficients of variation on both failure and confidence. The results of these estimates are shown in Figures 7-13 of this report. However, in no case were the median estimates permitted to exceed 1.25 x the calculated lateral load capacity at the center of gravity of the equipment.

8.0 RESULTS AND CONCLUSION

8.1 HCLPF Estimation

The following is a summary of the HCLPF determined in this evaluation.

[1] Note this is based on uncracked section actual frequency at or near failure could be based on at least a partially cracked section. This calculation results in a conservative HCLPF estimate.

8.1.1 Vertical Flat Bottom Tank

Spectra Given	NUREG/CR 0098
in Appendix C	Median Ground Spectra
0.32g pga	0.36g pga

8.1.2 Vertical Clip Angle Mounted Tank

Floor Spectra Given
in Appendix C
0.88g pfa
0.42g pga

8.1.3 Generic Motor Control Center (Operating During EQ.)

a) Mounted on Floor
Floor Spectra Given
in Appendix C
0.32g pfa
0.15g pga

b) Mounted on Ground

NUREG/CR 0098 Median Ground Spectra
.71g pga

8.1.4 Horizontal HX on Two Saddles

Floor Spectra
Given in Appendix C
0.93g pfa
0.44g pga

8.1.5 Block Wall

Floor Spectra
Given in Appendix C
0.67g pfa
0.32g pga[2]

8.2 Best Estimate (50 Percent Probability) Peak Ground Acceleration Evaluation of Failure at the 50 Percent Confidence Levels

The following is a summary of the best estimate seismic peak ground acceleraion required to cause component failure.

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[2] See discussion in Section 6.3 concerning the change in wall stiffness and its effect on HCLPF estimation.

8.2.1 Vertical Flat Bottom Tank

Floor Spectra Given in Appendix C (Figure 7)	NUREG/CR 0098 Median Ground Spectra (Figure 8)
1.25 x .667 = 0.83g	1.25 x 0.667 = 0.83g
50% = 1.10g	50% = 1.20g
Use 0.83g for median	Use 0.83g for median

8.2.2 Vertical Clip Angle Mounted Tank

Floor Spectra Given
in Appendix C (Figure 9)

1.25 x .88 = 1.10g
50% = 2.6g
Use 1.10g for median

8.2.3 Generic Motor Control Center (Operating During E.Q.)

Floor Spectra Given in Appendix C (Figure 10)	NUREG/CR 0098 Median Ground Spectra (Figure 11)
1.25 x 1.5g = 1.88g	1.25 x 1.5 = 1.88g
50% = 4.0g	50% = 2.7g
Use 1.88g for median	Use 1.88g for median

It should be noted that in developing the median estimate of failure, I do not consider the GERS (Function during) curve shown in Figure 2 to Appendix D of this report as a best estimate of functional failure. Rather the curve shown is a lower bound estimate based on the test data shown. For this reason, I do not believe it is appropriate to base a median estimate of failure on this curve. Hence the median estimate contained in this report is higher than what would be expected using the GERS (Function during) curve only.

8.2.4 Horizontal HX on Two Saddles

Floor Spectra Given
in Appendix C (Figure 12)

1.25 x 0.925 = 1.15g
50% = 1.95g
Use 1.15g for median

8.2.5 Block Wall

Floor Spectra Given
in Appendix C (Figure 13)

1.25 x 1.04 = 1.30g
50% = 2.10g
Use 1.30g for median

9.0 REFERENCES

(1) Kennedy, R.P., "Various Types of Reported Safety Margins and Their Uses," Section 2, Proceedings of EPRI/NRC Workshop on Nuclear Power Plant Reevaluation for Earthquakes Larger than SSE," Palo Alto, October 1984.

(2) Prassinos, P.G. et. al. "Recommendations to the Nuclear Regulatory Commission on Trial Guidelines for Seismic Margin Reviews of Nuclear Power Plants," NUREG/CR-4482, March 1986.

(3) NTS Engineering and RPK Consulting; "Evaluation of Nuclear Power Plant Seismic Margin," Tech. Paper No. 1551.05 Prepared for EPRI, Nov. 1986

(4) EQE, "Refueling Cavity Water Storage Tank - TK4" Seismic Margin Study, April 1987

(5) Kennedy, R.P. "Memorandum to HCLPF Panel - Modified Bolt Chairs for Tank TK-4"," 8 July 1987

(6) Roark, R.J. and Young, W.C. Formulas for Stress and Strain 5th Ed. McGraw-Hill, 1975.

(7) Haroun, M.A. and G.W. Housner, "Seismic Design of Liquid Storage Tanks," Journal of the Technical Councils of ASCE, Vol. 107, No. TC1, 1981, pp 191-207

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Table 1 Estimates of COV's of Equipment Seismic Margins as a Function of Location, Frequency and Damping

EQUIPMENT MOUNTING	COV (Percent)								
	Failure								Confidence
	Resonant 2 - 15 Hz Damping				Out of Resonance > 15 Hz < 2 Hz Damping				
	3	5	7	10	3	5	7	10	
Floor > 40'	200	175	150	125	100	87	75	62	35
Floor ≤ 40'	150	125	110	90	85	60	50	45	25
Ground W/SSI	100	85	75	65	60	55	45	40	15
Ground w/O/SSI	90	80	70	60	55	50	40	40	10

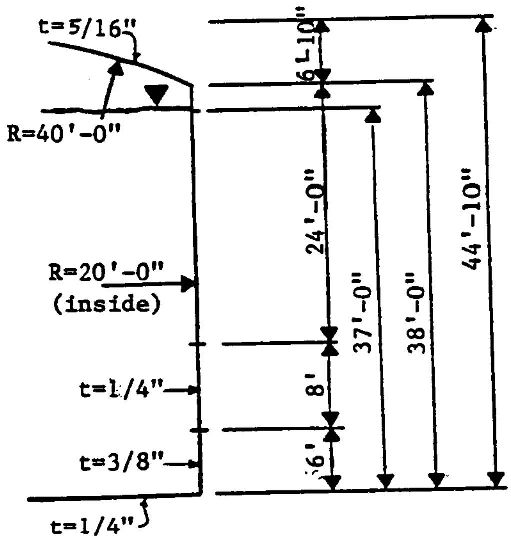


Figure 1a--Vertical Flat Bottom Tank Geometry

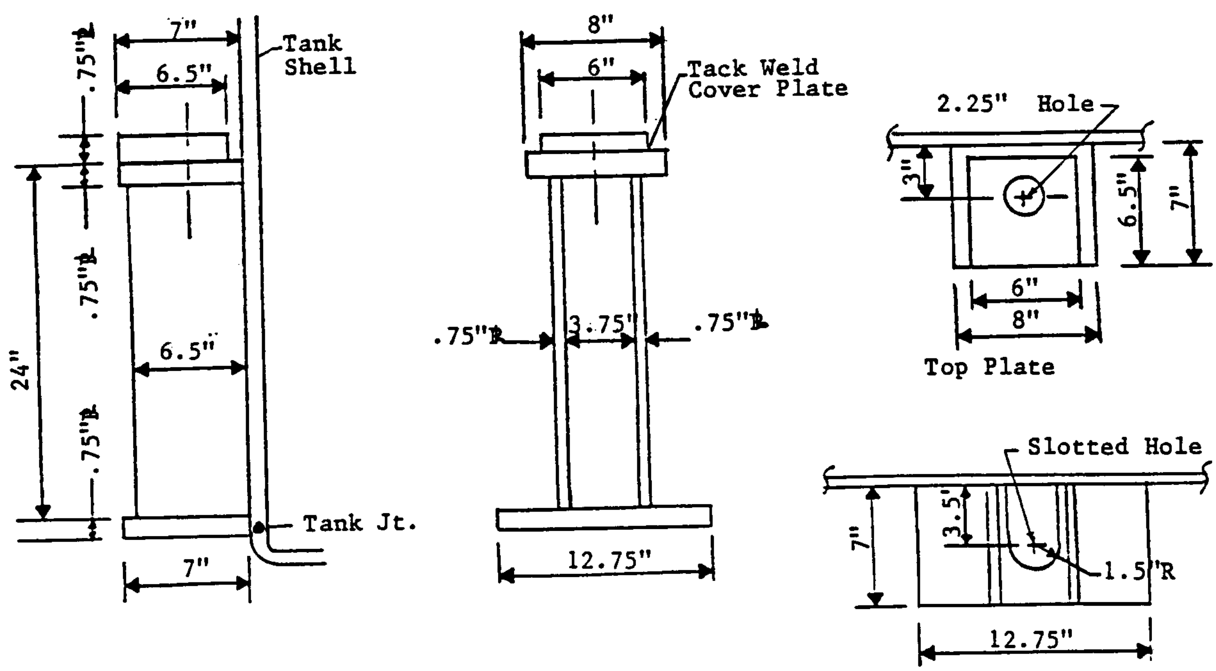


Figure 1b--Modified Bolt Chair for Flat Bottom Tanks

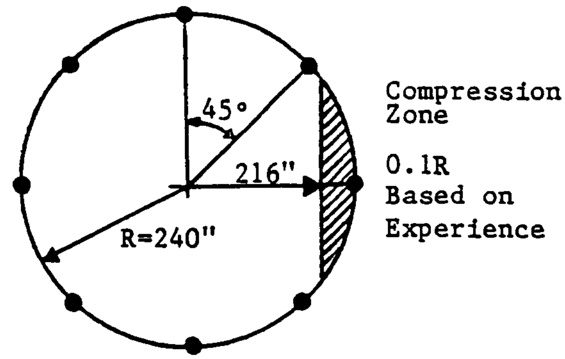


Figure 2--Overturning Diagram

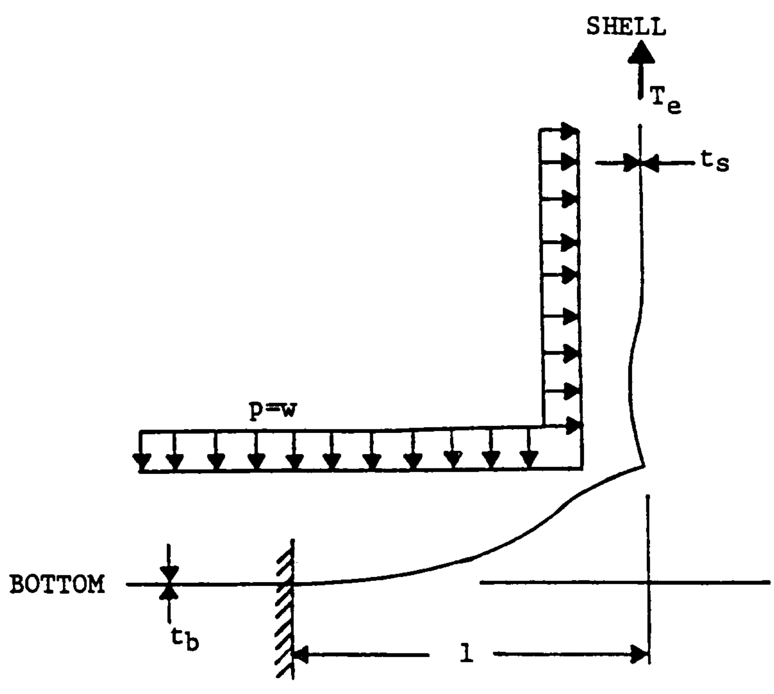


Figure 3--Schematic Illustration of Tank Bottom Behavior Near Tensile Lift-off Region of Tank Shell

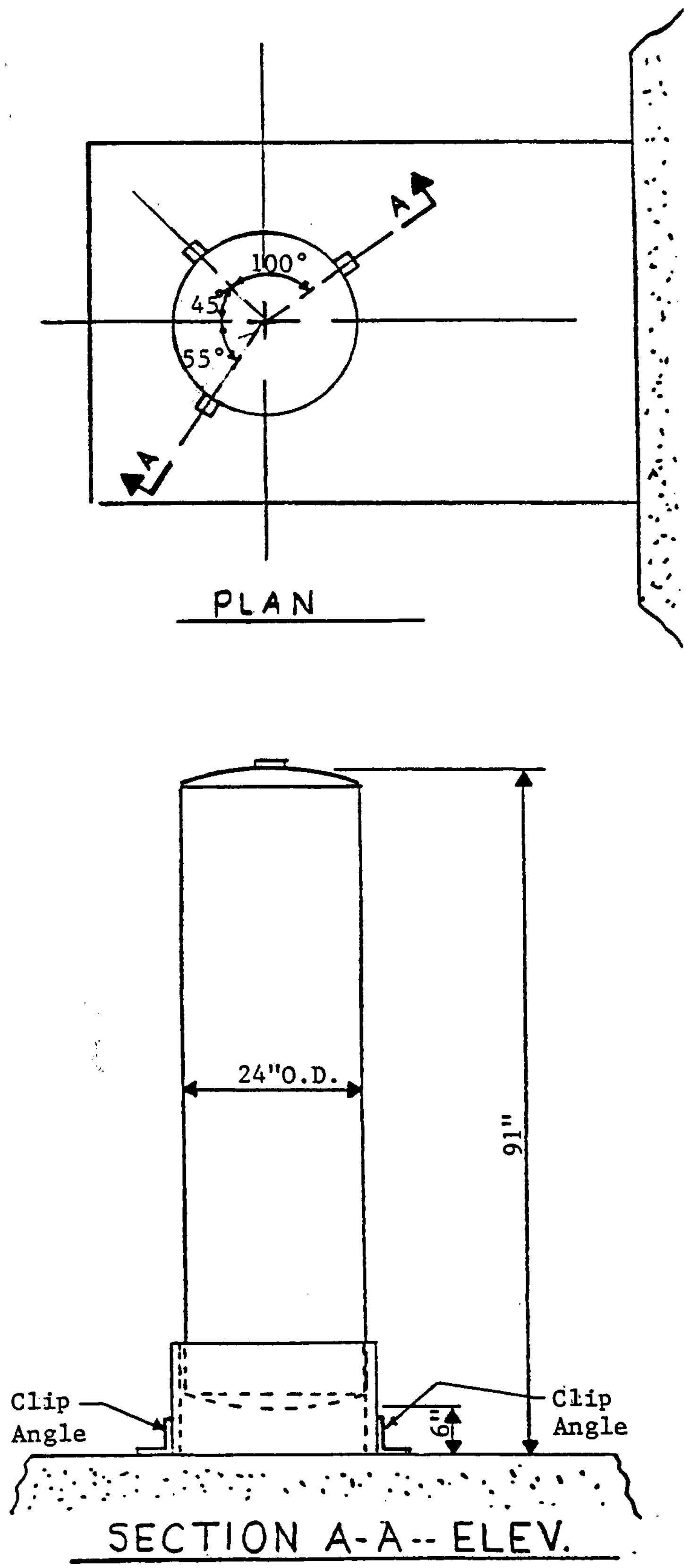


Figure 4--General Arrangement of Air Tank

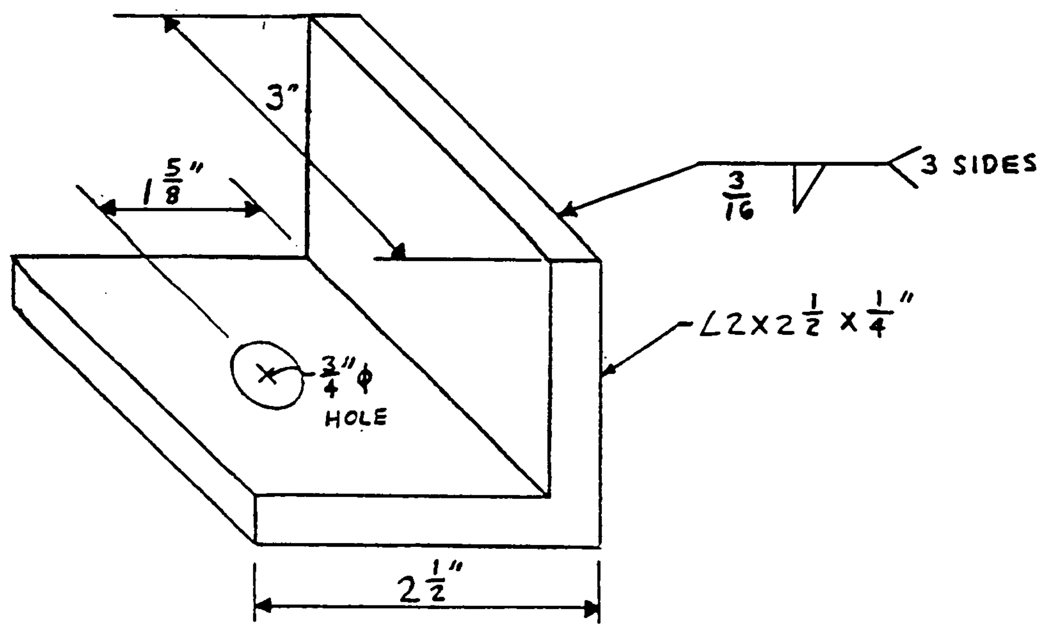


Figure 5--Clip Angle Detail

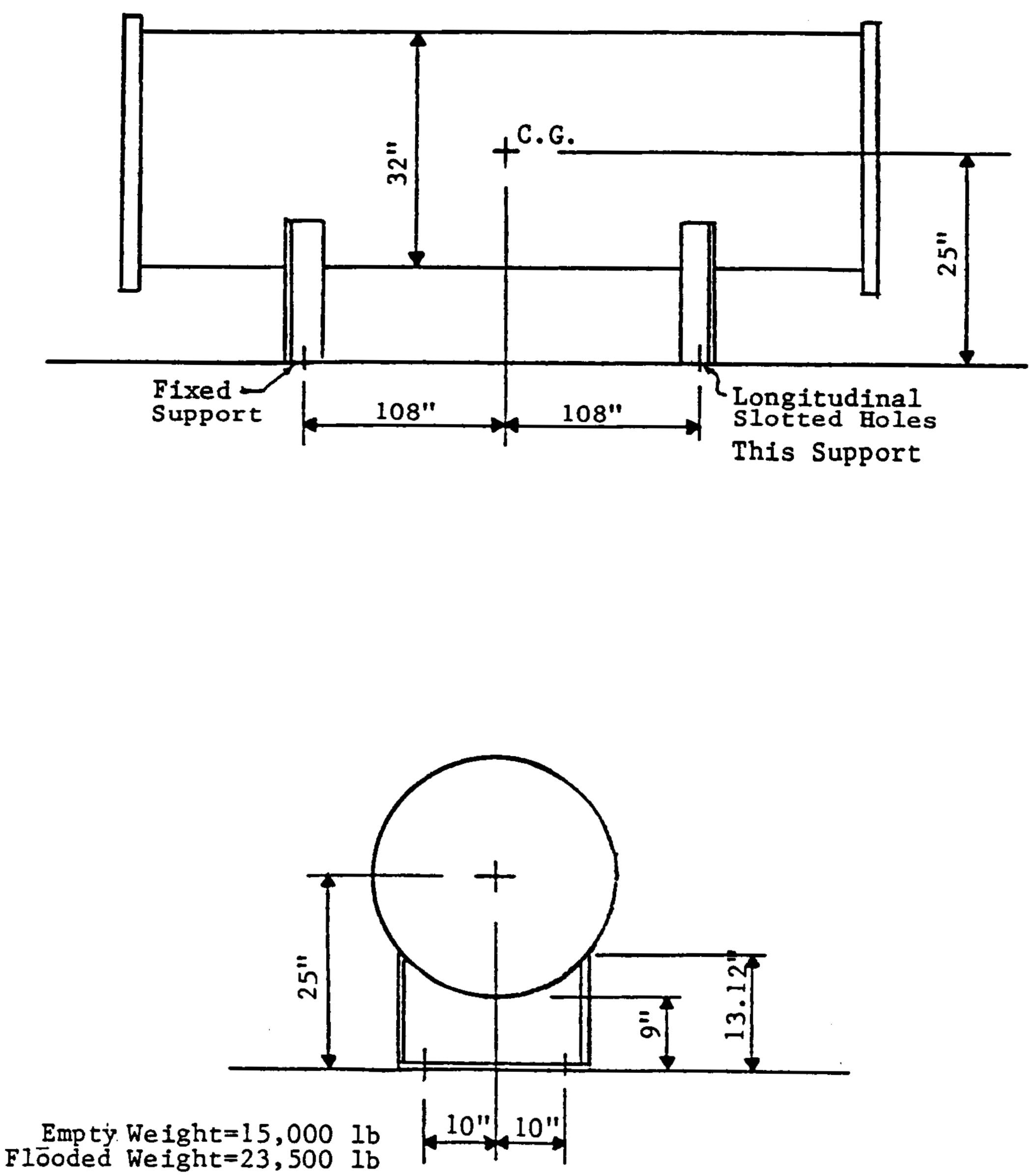


Figure 6--General Arrangement of Horizontal Heat Exchanger

Figure 7--HCLPF FRAGILITY CURVES FOR 95%, 50% AND 5% CONFIDENCE LEVELS
Component: Vertical Flat Bottom Tank
HCLPF= .32 g COV: Failure=0.70 Confidence=0.10

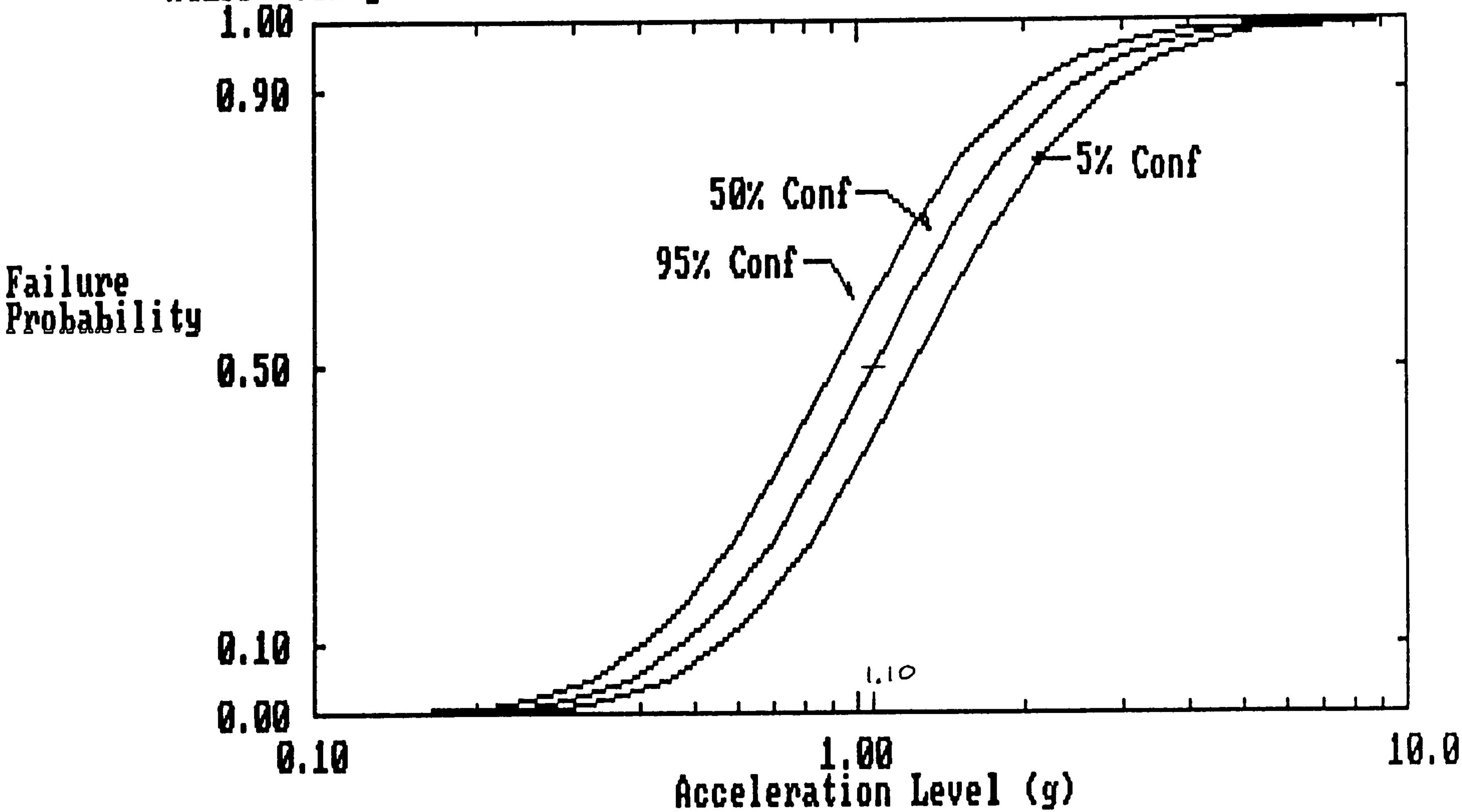


Figure 8--HCLPF FRAGILITY CURVES FOR 95%, 50% AND 5% CONFIDENCE LEVELS
Component: Vertical Flat Bottom Tank
HCLPF= .36 g   COV: Failure=0.70   Confidence=0.10

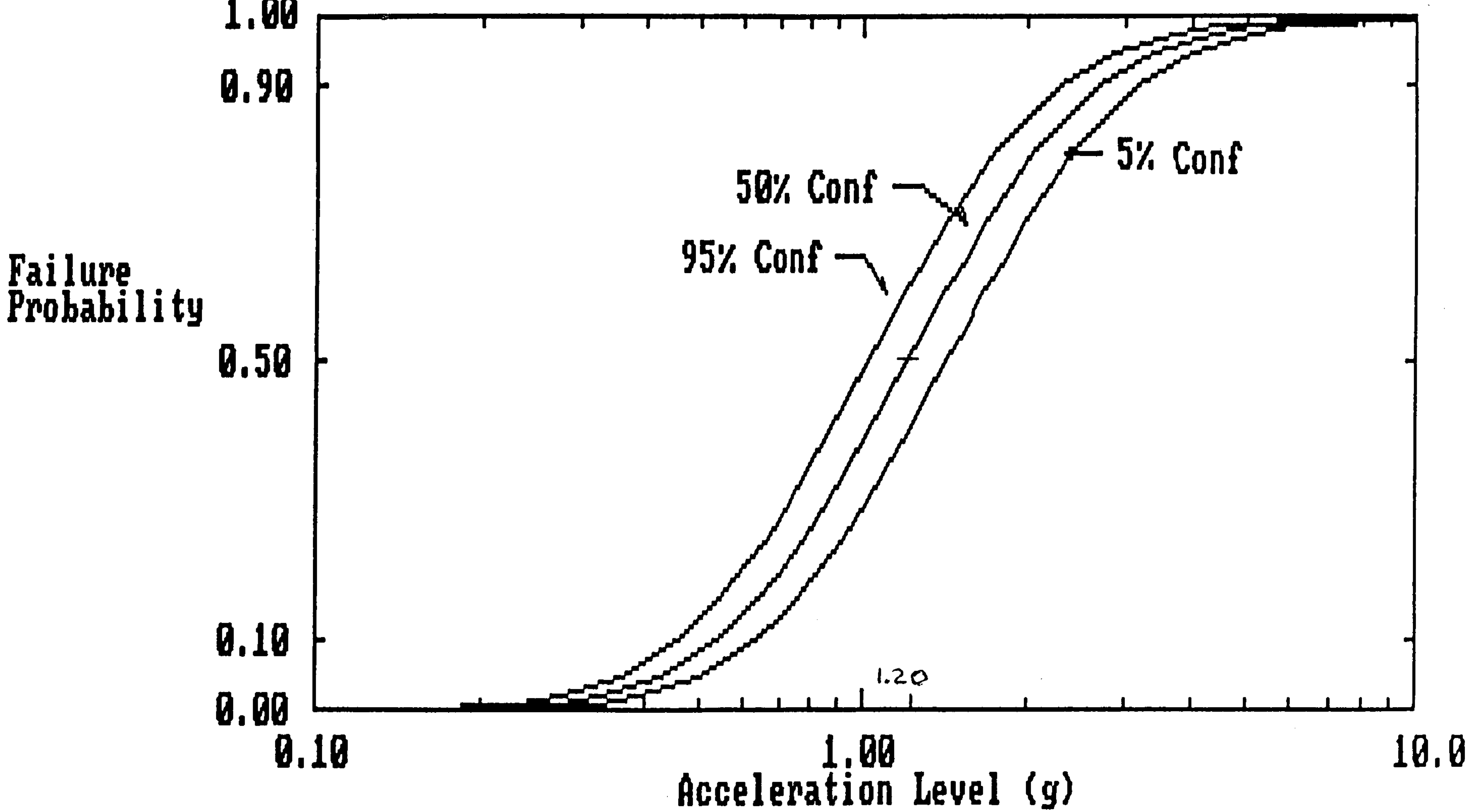


Figure 9--HCLPF FRAGILITY CURVES FOR 95%, 50% AND 5% CONFIDENCE LEVELS
Component: Vertical Clip Angle Mounted Tank
HCLPF= .42 g COV: Failure=0.87 Confidence=0.35

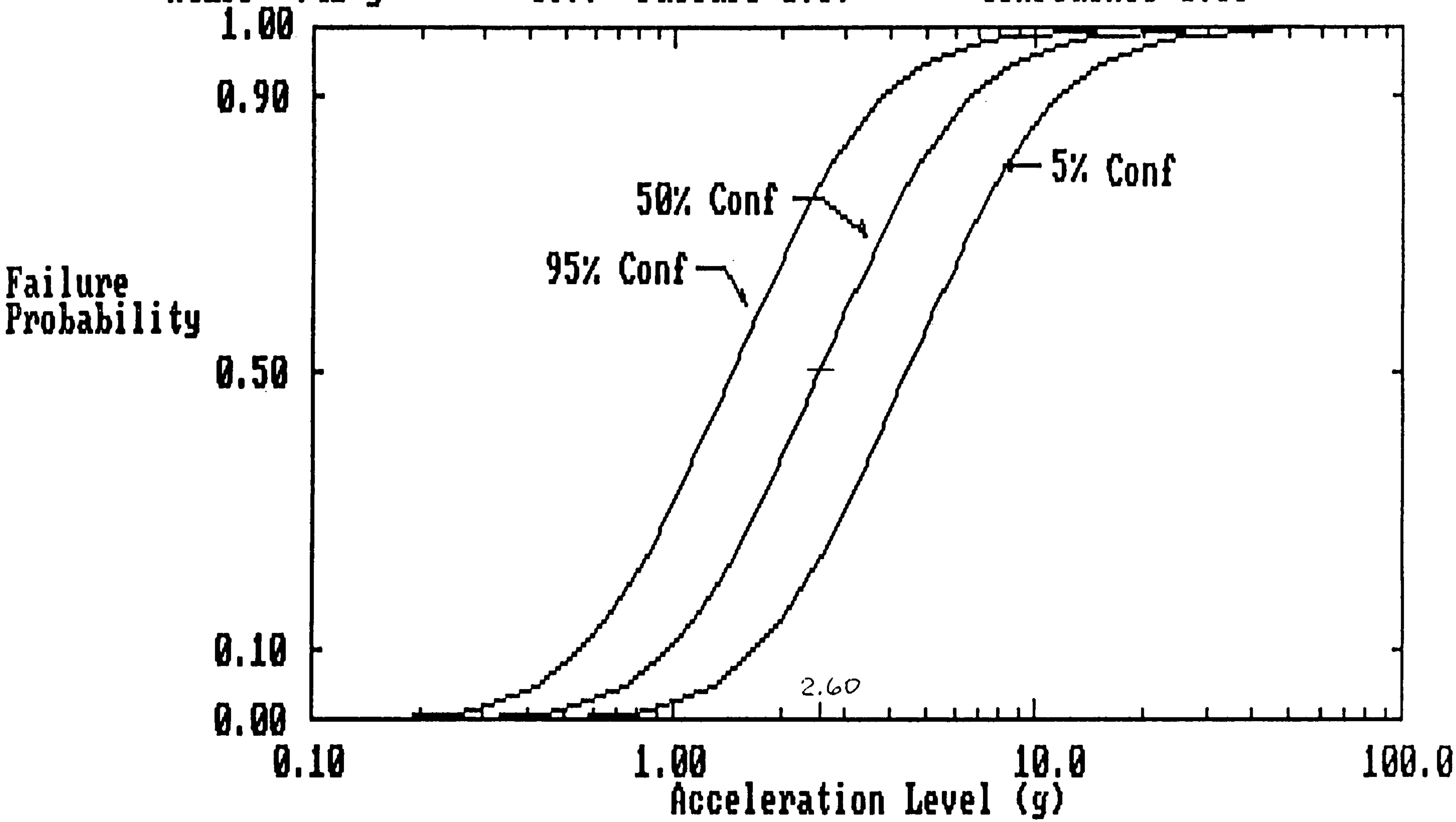


Figure 10-HCLPF FRAGILITY CURVES FOR 95%, 50% AND 5% CONFIDENCE LEVELS
Component: Generic Motor Control Center--Mounted on Floor
HCLPF= .32 g   COV: Failure=1.75   Confidence=0.35

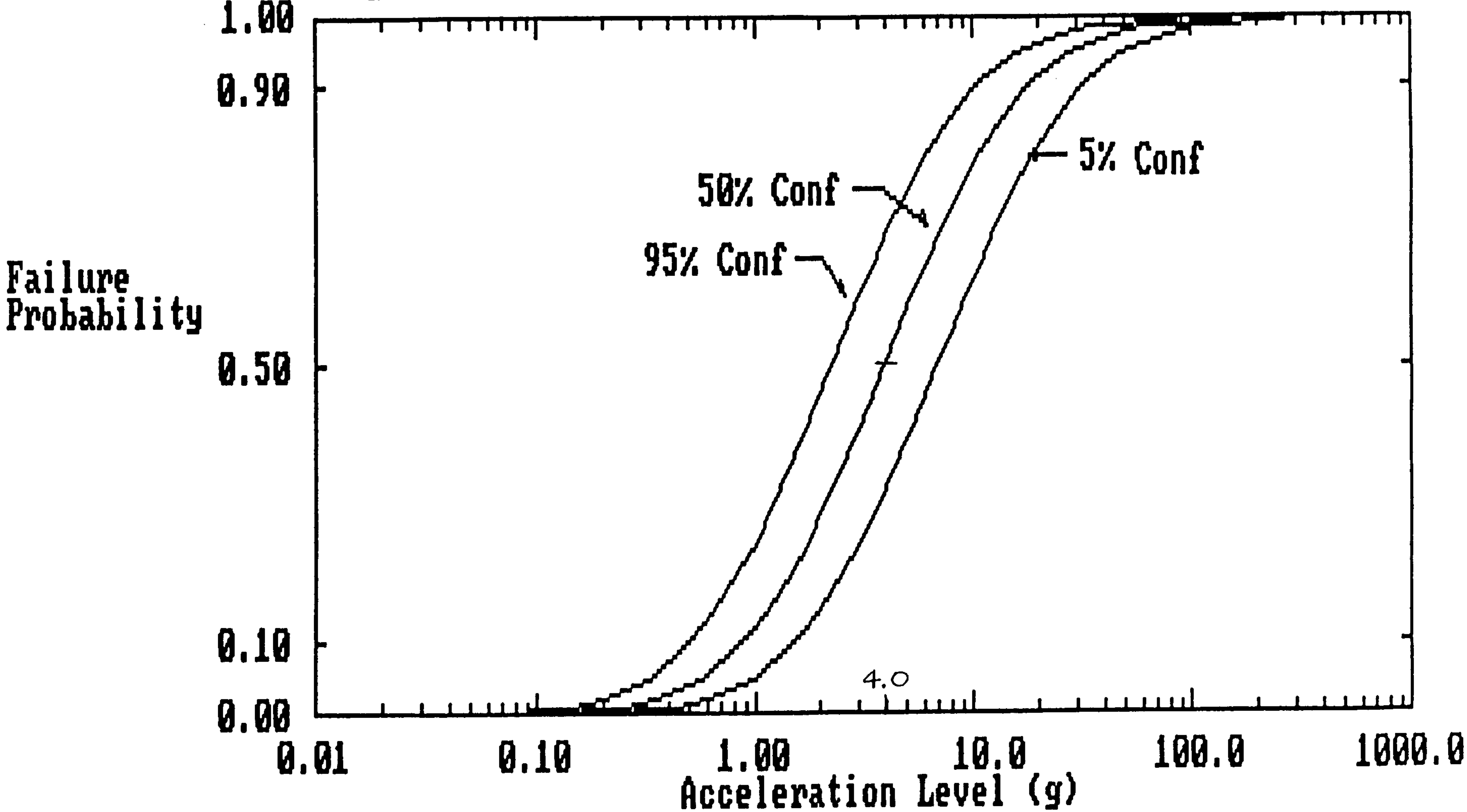


Figure 11-HCLPF FRAGILITY CURVES FOR 95%, 50% AND 5% CONFIDENCE LEVELS
Component: Generic Motor Control Center--Mounted on Ground
HCLPF= .71 g COV: Failure=0.80 Confidence=0.10

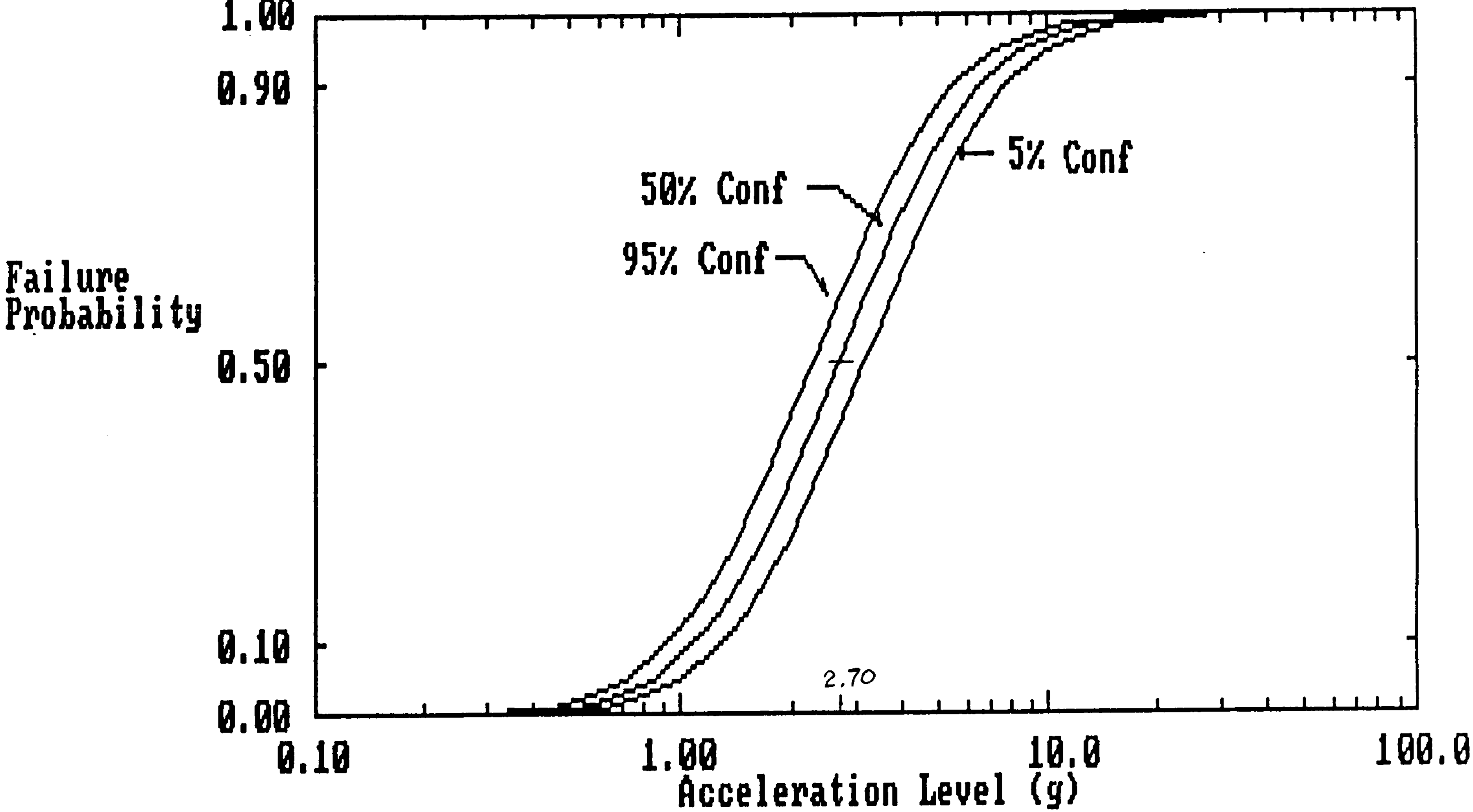


Figure 12-HCLPF FRAGILITY CURVES FOR 95%, 50% AND 5% CONFIDENCE LEVELS
Component: Horizontal HX on Two Saddles
HCLPF= .44 g COV: Failure=0.60 Confidence=0.35

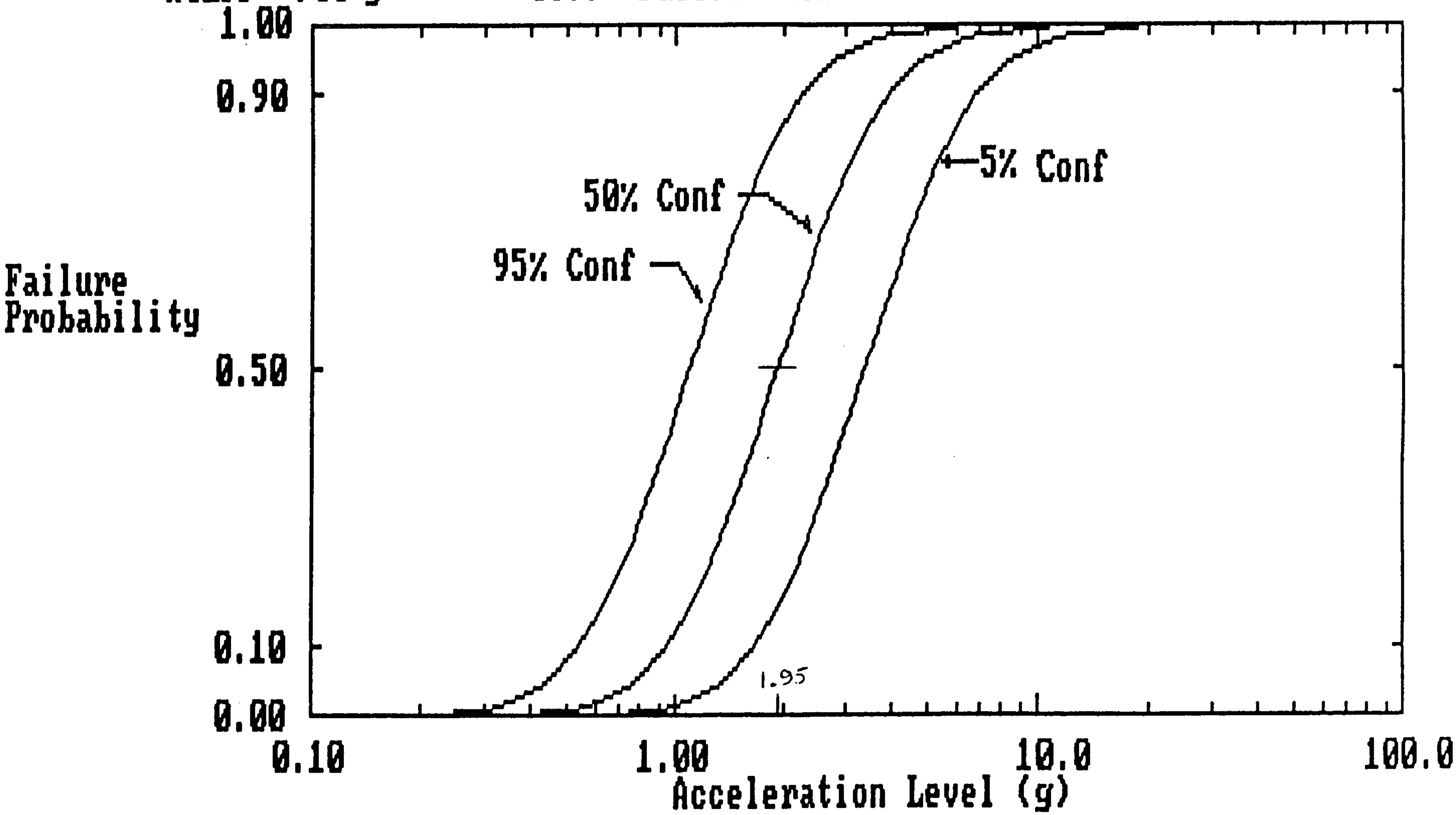
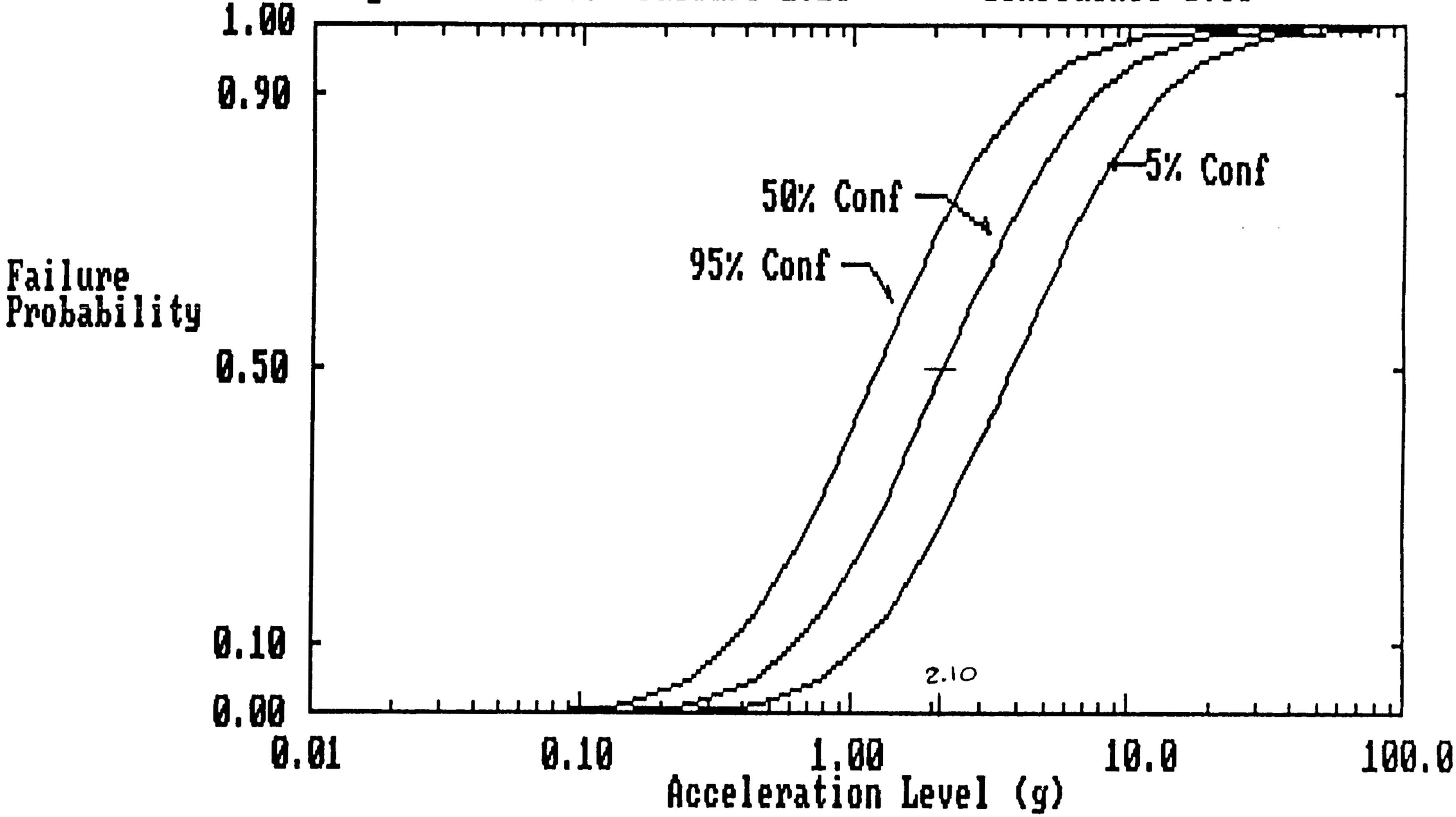


Figure 13-HCLPF FRAGILITY CURVES FOR 95%, 50% AND 5% CONFIDENCE LEVELS
Component: Block Wall
HCLPF= .25 g COV: Failure=1.25 Confidence=0.35



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Appendix A

Background on Development of CDFM Method for Determination of HCLPF's

NUREG/CR-4482
UCID-20579

Recommendations to the Nuclear Regulatory Commission on Trial Guidelines for Seismic Margin Reviews of Nuclear Power Plants

Draft Report for Comment

Manuscript Completed: December 1985
Date Published: March 1986

Prepared by
P. G. Prassinos, M. K. Ravindra, and J. B. Savy, under the guidance and with the concurrence of the Expert Panel on the Quantification of Seismic Margins: R. J. Budnitz (Chairman), P. J. Amico, C. A. Cornell, W. J. Hall, R. P. Kennedy, J. W. Reed, and M. Shinozuka.

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U.S. Nuclear Regulatory Commission
Washington, D.C. 20555
NRC FIN No. A-0398

The systems analyses performed to develop the Boolean expression should be fully documented. This documentation should include the analysis techniques and tools used, the method used to "prune" the fault trees and the justification, and a description of the basic events (their failure modes) that make up the Boolean expression

4.8 Step 8 - Margin Evaluation of Components and Plant

Steps 7 and 8 are performed concurrently and with close interaction between the system and fragility analysts.

The components that require margin evaluation, called the "screened in" components, have been identified during the plant review and the two plant walkdowns. Design details and actual existing conditions have been recorded (as far as practical).
The objectives of the analysis in Step 8 are:

- To estimate the HCLPF of these components
- To estimate the HCLPF of the plant.

For each HCLPF evaluation, two alternative approaches are presented.

4.8.1 Capacity of Components

The concept of HCLPF is similar to the traditional notion of using code-minimum strengths and code-maximum loads in structural design codes. The specification of these minimums and maximums was done by code committees using past performance data, results of analysis and research, and collective expert judgments. They implicitly or explicitly recognize the uncertainties in loads and strengths. The capacity of a component calculated using these specifications was considered to be conservatively low. The HCLPF value calculated using the procedures described in this report has similar attributes: it is conservative, and it recognizes the uncertainties based on the Panel's judgment.

There are two candidate approaches for calculating the HCLPF value of components: the Conservative Deterministic Failure Margin (CDFM) method proposed by Kennedy (Ref.7) and the fragility-analysis method. In the CDFM method, a set of deterministic rules (e.g., ground response spectra, damping, material strength, and ductility) is prescribed; the capacity of the component determined using these rules gives a HCLPF value that may be more conservative than necessary. In the fragility-analysis method, the median ground acceleration capacity A_m and the logarithmic standard deviations β_R and β_U for which there is less than a five percent probability of failure with 95 percent confidence. The randomness and uncertainty in the median capacity are assumed to be lognormally distributed. In the trial plant reviews, both these methods may require that seismic response analyses separate from the design analyses be performed. The fragility analyst must review the structural models used in the plant design to confirm the adequacy of these models and the appropriateness of scaling the responses. If scaling is not appropriate, the response analysis becomes a major effort in seismic margin reviews. In the CDFM method, values for a number of parameters (e.g., system ductility, damping, and response spectra) need to be selected. In the fragility-analysis method median values β_R and β_U need to ber estimated by the fragility analyst. There have not been enough studies done to compare the HCLPF estimated using these two candidate methods for different components. Additional comparison studies should be conducted to identify situations where both methods would yield comparable results and those where the results would widely differ. A review of such results would also lead to a "calibration" of the parameter values of either or both methods so that the two methods give essentially identical capacity estimates. The final goal of such studies would be to provide a set of deterministic rules in the CDFM method for calculating the HCLPF of screened in components. Until such research is done, it is recommended that both the candidate methods be used to calculate the HCLPF of components in trial plant reviews. The trial plant reviews should be viewed as providing further basis and guidance on research towards calibration of the two candidate methods.

4.8.1.1 Conservative Deterministic Failure Margin (CDFM) Method

In this method a failure margin is computed using conservative material and response parameters but taking credit for conservatively defined failure capacity and inelastic energy absorption capability of structures and components. The following parameter values have been proposed (Ref. 7) and might be more conservative than necessary:

Load Combination:	Normal + Earthquake Review Level
Ground Response Spectrum:	84% Non-Exceedence Probability Site-Specific Spectrum
Damping:	Depending on the earthquake review level, the following are the conservative estimates of the median values: Structure: 7% Piping: 5% Cable trays: 15%
Structural Model:	Best-estimate - median
Soil-Structure Interaction:	Envelope expected parameter variation
Material Strength:	95% exceedance actual strength
Static Capacity Equations:	84% exceedance by test data or code equation
System Ductility: (Inelastic Energy Absorption)	Conservatively selected to be between 1.0 and 1.5. For shear wall structures, should not be less than 1.3.
Floor Spectra Generation:	Median damping value for equipment Frequency shifting of floor spectra rather than peak broadening.

For structure/equipment qualified by analysis, the response of the equipment is calculated using the above structural and equipment response parameters. Potential failure modes of the equipment are identified. The static inelastic capacities of the structure/equipment are estimated. If the capacity of the structure/equipment exceeds the calculated response for the load combination (Normal + Earthquake Review Level), it is assumed that the component has a HCLPF value exceeding the earthquake review level peak ground acceleration. For equipment qualified by test, the floor spectrum for median equipment damping is generated using the above conservative structural and/or equipment response parameters. If the floor spectral values throughout the equipment frequency range of interest are less than generic equipment ruggedness spectrum (GERS) for the equipment (Ref. 18), it is assumed that the equipment has a HCLPF exceeding the earthquake review level PGA. So far, GERS has been developed for seven classes of equipment (i.e., motor-operated valves, motor control centers, switchgear, batteries and battery racks, inverters, battery chargers, and relays). For other equipment, one should use the highest spectral value for which similar equipment has been qualified as the capacity.

The GERS will be lower than the lowest observed failure level for the equipment (i.e., the GERS is the highest level for which the equipment did not fail). For equipment mounted on floors at higher elevations in the structure, the conservatisms in floor spectra generation and the conservatisms in structural parameters (i.e., damping and system ductility) yield HCLPF values that are considerably less than the median capacities. However, for equipment on grade that do not include significant response conservatism, use of GERS or experience data may not guarantee that there is no "cliff" in the capacity beyond the value of HCLPF (i.e., the component may fail suddenly when the peak ground acceleration exceeds the HCLPF value, instead of a gradual increase in the probability of failure increases). To avoid this problem, it is recommended that the capacity determined by experience data for grade level equipment be reduced by a factor. This factor may be determined during the trial plant reviews.

By a judicious selection of the values of different parameters, the CDFM method aims to produce a conservative estimate of the component's HCLPF. However, the CDFM method is less conservative than the procedures given in the Standard Review Plan (Ref. 9). The load combination specified is more liberal compared to the SRP requirements, i.e., no OBE load combination and no LOCA + review earthquake load combination in the CDFM method. The ground response spectrum is a 84% nonexceedence probability site-specific spectrum and is expected to be less conservative than the R.G. 1.60 spectrum. Similarly, the damping values proposed for the seismic margin review are more liberal than those specified in the Standard Review Plan.

The basis for the selection of values of different parameters in the CDFM methods and how they contribute to the high confidence in the capacity that assures a low probability of failure should be studied. For example, the use of 84% nonexceedence-probability site-specific spectrum and conservative estimates of the median damping are expected to result in a computed capacity indicating a low probability of failure. The use of material strength at 95% exceedance value and 84% exceedance value for static capacity prediction equations is expected to contribute to the high confidence statement about the capacity. However, this approach cannot be used to determine the contributions of different parameters because the seismic capacity of a component is a nonlinear function of these parameters; the impact on capacity of any value of a single parameter depends not only on the significance of the parameter on the median capacity but also on the relative variabilities (i.e., randomness and uncertainties) of all the parameters. The CDFM method discussed here may be even more conservative than necessary. Until further research on calibration is performed (discussed earlier), the degree conservatism cannot be quantified.

4.8.1.2 Fragility-Analysis Method

One method of describing the fragility of a component is to express it in terms of three parameters (Ref. 19): median capacity A_m, logarithmic standard deviations β_R, and β_U representing, respectively, randomness in the capacity and uncertainty in the median value. (Fragility Handbook, Ref. 19) Rather than estimating the median capacity as a product of an overall median safety factor times the SSE pga for the plant (where the overall safety factor is a product of a number of factors representing the conservatisms at different stages of analysis and design), the median capacity is evaluated using median structural and equipment response parameters, median material properties, and ductility factors, median static capacity predictions, and realistic structural modeling and method of analysis. If the fragility analyst is convinced that the scaling of response is appropriate, the median seismic capacity may be estimated as the product of the overall median safety factor and the SSE pga.

The median response of the structure/equipment for the earthquake review level (REL) is calculated. The median capacity of the structure/equipment is estimated as the median static capacity multiplied by the median inelastic energy absorption capacity factor. The median ground acceleration capacity of the structure/element is approximately estimated as:

$$A_m \approx (\text{REL}) \frac{\text{Median Capacity} - \text{Normal Design Load Response}}{\text{Median Response caused by REL}}$$

This is valid because the normal loads have low variability and the normal design loads are conservatively selected.

In lieu of explicitly determined β_R and β_U, the HCLPF value for the structure/equipment may be conservatively estimated by assuming $\beta_R + \beta_U = 0.08$ and the lognormal model:(Ref. 10 and 12)

$$\text{HCLPF} \approx 0.25\ A_m$$

If the HCLPF value calculated as above does not exceed the earthquake review level, the analyst may revise the capacity by estimating β_R and β_U using plant-specific data and PRA methods (i.e., seismic fragilities). Another option, if this proves to be too conservative, is to revise the median-capacity estimate by performing further studies such as nonlinear, inelastic static, or time history dynamic analyses.

This section is an excerpt from
EPRI Report NP-4101-SR

Section 2

VARIOUS TYPES OF REPORTED SEISMIC MARGINS AND THEIR USES

R. P. Kennedy*

INTRODUCTION

Nuclear power plant structures and safety-related systems have been generally designed conservatively for a safe shutdown earthquake (SSE) and more conservativel for a smaller operating basis earthquake (OBE). Depending upon the relative conservatism of the design criteria, either the SSE or the OBE will control the design. For plants with SSE levels less than 0.2g, often non-seismic loadings control the design.

In recent years, increasing knowledge in the geoscience field has led to a better understanding that, although highly unlikely, it is possible for the nuclear power plant to be subjected to earthquake ground motion greater than the ground motion for which the plant was designed. For this reason, interest has developed in demonstrating that nuclear plant structures and safety-related systems can safely withstand earthquake ground motion larger than their design earthquake ground motions (SSE and OBE). Within this paper, this larger-than-design earthquake ground motion will be called the seismic margin earthquake (SME) to distinguish it from the design earthquakes. The plant has already been designed. Therefore, for the SME the goal is not to design the plant. The goal is to determine the performance of already-designed structures, components, and systems when subjected to the SME. Different and generally more liberal criteria should be used when evaluating the performance of structures, components and systems for the SME than were used i design. Retrofit, and redesign, should only be contemplated if one cannot show a seismic margin greater than unity for the SME using these more liberal criteria. In other words, the SME is not a design earthquake. It is not a replacement for the SSE and generally has nothing to do with design. The SME is a performance-check earthquake.

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VARIOUS TYPES OF SEISMIC MARGIN

Existing literature contains a wide variety of highly dissimilar criteria for determining seismic margin. Therefore, one must be careful to distinguish what type of seismic margin is being reported. Seismic margins reported in the literature can generally be divided into the following four categories:

1. Design Seismic Margin (DM) - The seismic margin computed using US NRC Reg. Guides (R.G.), Standard Review Plans (SRP), Design Code Capacities and load combinations.

2. Code (Elastic-Computed) Seismic Margin (CM) - Margin computed using possibly less stringent structural response parameters (such as damping) and less stringent load combination (normal plus seismic) but assuming essentially elastic behavior and capacities defined by code.

3. Conservative Deterministic Seismic Margin Against Failure (CDFM)- A failure margin computed using conservative material and response parameters but taking credit for conservatively defined failure capacity and inelastic energy absorption capability of structures and components.

4. Probabilistic Seismic Margin Against Failure (PFM) - Median-centered estimate of seismic margin which also displays uncertainties in the estimate.

Both the Design Margin (DM) and the Code Margin (CM) represent traditionally computed margins. As such, they are prescriptive and essentially non-controversial. However, in most cases, such margins are very conservative and are not a good measure of the failure capacity. For some SME problems, it might be adequate to determine the DM or CM. Some examples of when the DM or CM might be adequate are:

a. Where the SSE design response spectrum was a Housner spectrum, one might be required to demonstrate margin for a R.G. 1.60 spectrum.

b. Where the SSE was 0.12g, one might be required to demonstrate margin for an SME of 0.14g.

Both of these examples do not require one to push oneself to demonstrate substantial margin. In my experience, the DM or CM approaches are generally adequate to demonstrate margin for a SME less than about 0.15g or a SME less than about 1.2 times the SSE. For more severe SME problems, it is generally necessary to consider failure margins.

The Conservative Deterministic Failure Margin (CDFM) is more controversial than either the DM or CM. Actual failure capacities are highly uncertain. A CDFM does not display the uncertainty. However, it does represent a reasonably conservative, but realistic, measure of the failure capacity of the structure or component. The Probabilistic Failure Margin (PFM) fully displays the uncertainty in failure capacity and represents the most complete and best descriptor of the failure margin. However, there will always be great uncertainty about uncertainty. Secondly, the PFM attempts to remove all or nearly all conservatism. As such, the PFM will tend to be more judgmental and controversial than the CDFM.

Irrespective of which of the above four types of seismic margin (SM) is being computed, this margin is generally obtained by one of the following equations:

$$SM_1 = \frac{C}{D_{SME} + D_{NS}} \tag{1}$$

$$SM_2 = \frac{C-D_{NS}}{D_{SME}} \tag{2}$$

where C represents the capacity, D_{SME} represents the demand (loading) from the SME, and D_{NS} represents the non-seismic demand (laoding) from all non-seismic loads in the load combination. In my opinion, Equation (2) provides a better description of the seismic margin than does Equation (1). As an example, assume: C = 100; D_{SME} = 20; D_{NS} = 60. For this example, SM_1 = 1.25 and SM_2 = 2.00. The seismic margin SM_2 represents the multiplier by which the SME can be factored before reaching capacity C while the margin SM_1 does not truly provide a seismic margin but provides a margin for the entire load combination. The seismic margin SM_2 will often be much larger than the margin SM_1 for structures or components in which seismic is not the dominant loading. In these cases, SM_1 is misleadingly low.

CANDIDATE CRITERIA FOR VARIOUS SEISMIC MARGINS

Table 2-1 presents some recommended criteria for use in estimating each of the different types of seismic margin for structures. Essentially each of the deterministic seismic margins (DM, CM, and CDFM) uses conservatively biased criteria. Generally, it is suggested that parameters be set at about either the 84% or 95% exceedance probability or non-exceedance probability (NEP) levels depending upon the degree of conservatism desired. For normally distributed parameters, the 84% NEP and the 84% exceedance probability values correspond to plus and minus one standard deviation from the mean. This level of conservatism is considered to represent a reasonable degree-of-conservatism for individual structural response parameters. Some capacity parameters should probably be more conservatively selected. The 95% exceedance probability corresponds to minus 1.65 standard deviations from the mean for normally distributed parameters. Such a value envelopes essentially all capacity data except for extreme outliers which are likely to be suspect data.

The probabilistic failure margin (PFM) approach uses median-centered estimates with uncertainty bands for each parameter. It is suggested that these uncertainty bands should encompass about the central 90% of all possible parameter values. Thus, the uncertainty bands should encompass from about the 5% to the 95% NEP range with extreme outliers again being ignored.

TABLE 2-1

CANDIDATE SEISMIC MARGIN CRITERIA FOR STRUCTURES

(Example for Reinforced Concrete Structure)

PARAMETER	DESIGN MARGIN DM	CODE MARGIN CM	CONSERVATIVE DETERMINISTIC FAILURE MARGIN CDFM	PROBABILISTIC FAILURE MARGIN PFM
EARTHQUAKE	SSE OBE	SME	SME	SME
LOAD COMBINATION	SRP	NORMAL + SME	NORMAL + SME	NORMAL + SME
SPECTRA	R.G. 1.60	84% NEP	84% NEP	5.-95% NEP
DAMPING	R.G. 1.61 (4% - OBE) (7% - SSE)	84% EXCEEDANCE (7%)	84% EXCEEDANCE (7%)	5%-95% NEP (5%-20%)
STRUCTURAL MODEL	MEDIAN	MEDIAN	MEDIAN	MEDIAN & UNCERTAINTY
SOIL-STRUCTURE-INTERACTION-	ENVELOPE EXPECTED PARAMETER VARIATION	ENVELOPE EXPECTED PARAMETER VARIATION	ENVELOPE EXPECTED PARAMETER VARIATION	MEDIAN & UNCERTAINTY
MATERIAL STRENGTH	DESIGN STRENGTH (3000 PSI)	95% EXCEEDANCE ACTUAL STRENGTH (3400 PSI)	95% EXCEEDANCE ACTUAL STRENGTH (3400 PSI)	5%-95% NEP ACTUAL STRENGTH (3400 PSI -5000 PSI)
STATIC CAPACITY EQUATIONS	CODE CAPACITY (ACI 318)	CODE CAPACITY (ACI 318)	84% EXCEEDANCE BY ACTUAL TEST DATA (1.6*CODE)	5%-95% NEP BY ACTUAL TEST DATA (1.4-2.4*CODE)
SYSTEM DUCTILITY (INELASTIC ENERGY ABSORPTION)	IGNORE (1.0)	IGNORE (1.0)	95% EXCEEDANCE (1.3)	5%-95% NEP (1.3-3.0)

CONSERVATISM OF COMPUTED RESPONSE

Deterministic computed responses for a SME obtained using the parameters suggested in Table 2-1 and discussed in the previous sections will clearly be conservative when compared to median response results of a probabilistic margin review. In fact, one would have high confidence that there is greater than a 84% probability that actual responses to the SME would not exceed this deterministic computed response. Comparison of deterministic computed responses with probabilistic responses reported in several seismic PRAs indicate that the median response factor of safety for these deterministic computed responses range from a low of about 1.4 for a stiff concrete structure on rock to a high of about 3.0 for structures with significant SSI effects.

MATERIAL STRENGTH

For design, one generally uses conservatively biased design material strengths. For instance, concrete might have a design compressive strength of 3000 psi and one would have high confidence that this strength would be achieved or exceeded within 28 days after placement of the concrete. For seismic margin reviews to a SME, one should use conservatively biased actual material strengths which in the case of concrete take into account that concrete strength continues to increase with time beyond 28 days after placement. Material strengths used in deterministic seismic margin reviews for the SME should be sufficiently conservative that there is very little likelihood that actual strengths are less than those used in the margin review (approximately 95% exceedance probability strengths achieve this goal). For a PFM review, median material strengths plus the probable range of strengths should be used.

For a typical 3000 psi concrete design strength, one would likely expect that the 90% bounds on actual strengths after two years would range from about 3400 psi to 5000 psi with a median of about 4200 psi. This full range should be considered in a PFM review while a deterministic margin review could use a conservative strength of 3400 psi in lieu of the design strength of 3000 psi.

STATIC STRENGTH OR CAPACITY EQUATIONS

Code equations for static strength or capacity are generally very conservatively biased. In cases where one is attempting to predict a failure margin (either CDFM or PFM) and when failure test data exists to demonstrate excessive conservatism in code equations for static strength or capacity, one should use actual failure test data in lieu of code equations to predict seismic margins for the SME.

INELASTIC ENERGY ABSORPTION CAPACITY

Nearly all structures and components exhibit at least some ductility (i.e., ability to strain beyond the elastic limit) before failure. Because of the limited energy content and oscillatory nature of earthquake ground motion, this ductility is highly beneficial in increasing the seismic margin against failure for structures and components. The inelastic energy absorption, F_μ, represents the ratio of the SME at which a certain system ductility μ is reached to the earthquake level for which failure would be predicted by linear elastic analysis. The additional seismic margin due to this inelastic energy absorption factor F_μ should be considered in any failure margin review. Ignoring this effect will lead to unrealistically low estimates of the failure margin. It is impossible to correlate performance of structures and equipment in past earthquake experience with capacities predicted by elastic analyses without considering the F_μ factor.

In a probabilistic failure margin (PFM) review, one should estimate the probable range on F_μ. For instance, for a shear wall structure with highly non-uniform Demand/Capacity ratios throughout the structure (i.e., inelastic response is concentrated in localized regions), one might estimate the probable range for F_μ to be 1.3 to 3.0. This entire range should be used in a PFM review. For a conservative deterministic failure margin (CDFM) review, a conservative lower bound estimate on F_μ should be used. For this shear wall structure, such a conservative lower bound on F_μ might be 1.3. Actually, all but the most brittle structures and components will exhibit F_μ values of at least 1.3 so that values less than 1.3 should seldom be used for F_μ in a failure margin review.

CONSERVATISM OF COMPUTED CAPACITY

In a CDFM review, the capacity should be computed sufficiently conservatively so that if the computed response actually occurs, one has high confidence that the probability of failure is negligible. In other words, failure will occur if a severe unknown construction error exists or if the actual seismic response significantly exceeds the computed seismic response. Capacities computed following the guidelines in Table 2-1 and described in the previous sections for CDFM reviews are expected to achieve this goal.

Based upon comparison of capacities computed by the guidelines of Table 2-1 for CDFM reviews with median probabilistic computed capacities reported in several seismic PRAs, it is estimated that the median probable capacities (50% probability of failure) typically lie between 1.4 and 3.0 times the CDFM computed lower bound capacities. Responses would have to be increased by factors of about 1.4 to 3.0 before failures would be expected.

COMPARISON OF VARIOUS SEISMIC MARGINS

Probabilistic Failure margins (PFM) are typically displayed by fragility curves such as that shown in Figure 2-1 for a typical 0.15g SSE designed stiff shear wall structure founded on rock. These fragility curves illustrate that one has high confidence of low probability of failure for an SME less than 0.3g (i.e., 2 times the SSE) and has high confidence that the SME associated with a 50% probability of failure lies between 0.5g and 1.6g with a median value of 0.9g. Discussion on the development of such fragility curves is contained in References 1 and 2.

For this same structure, the SME associated with the Code Margin (CM) was only 0.16g and the SME associated with a Conservative Deterministic Failure Margin (CDFM) was 0.32g. Note that the CDFM value for the SME agrees closely with the high confidence of low probability of failure value of 0.3g obtained from a PFM review. This close agreement between the CDFM value and the high confidence, low probability of failure value from a PFM review has been observed in a number of cases for which both margin reviews have been conducted. In other words, the CDFM criteria in Table 2-1 can probably be used to deterministically establish the lower bound on PFM fragility curves. In fact, one might prescriptively define

$$\text{CDFM} \approx 95\% \text{ confidence of less than } 5\% \text{ probability of failure} \quad (3)$$

Use of Equation (3) would provide a simpler and probably more consistent method of obtaining the high confidence (≈ 95%), low probability (< 5%) point on fragility curves than the separation of variables method described in References 1 and 2. This point on the fragility curve would be deterministically determined using the CDFM criteria in Table 2-1. However, further validation of Equation (3) is needed.

A review of fragility curves presented in several seismic PRAs (Zion, Indian Point 2, Indian Point 3, Limerick, Millstone, Midland, and Seabrook) generally indicate a factor of 2.5 to 6.0 between the median (50% failure) fragility value and the high confidence (≈ 95%), low probability (< 5%) point on the fragility curve. Thus, margins defined by the CDFM criteria would still contain substantial conservatism below median fragilities.

level, the input to floor-mounted equipment should also be defined at the 84% NEP level. Use of 84% NEP floor spectra as input to equipment would provide the same level of response conservatism for equipment as exists for the structure. Unfortunately, the generation of 84% NEP floor spectra would require probabilistic structural response analyses which are more costly and have seldom been performed.

If 84% NEP floor spectra were generated and used in a margin study, one would have to multiply such spectra by a scale factor for equipment qualified by testing. The CDFM procedure defined in Table 2-1 introduces considerable conservatism in estimating the capacity of equipment qualified by analysis for a given floor spectrum. This factor of conservatism from the median failure capacity is estimated to range from about 1.5 to more than 3 for equipment qualified by analysis. Thus, to introduce a similar conservatism for equipment qualified by testing would require a scale factor of about 2. In other words, for a CDFM review the floor response spectrum to be used for comparison with equipment qualification test response spectrum (TRS) should be:

$$TRS \approx 2 * (84\% \text{ NEP Floor Response Spectrum}) \quad (4)$$

Again, it should be noted that within the current state-of-art one would not generally generate 84% NEP floor spectra for a seismic margin review. Instead, one would likely use conservative broadened floor spectra generated using conservative deterministic structural response parameters similar to those described in Table 2-1. These conservative broadened floor spectra can be used directly to determine a CDFM for equipment qualified by testing. However, for equipment qualified by analysis some conservatism should be removed in a CDFM review. This reduction in conservatism can be accomplished by:

1. Using median or slightly greater than median damping values for computing equipment response.
2. Perform frequency shifting of floor spectra rather than frequency broadening to account for frequency uncertainty.

For a seismic margin review of equipment mounted on structures, one should generally use 5% to 15% damped floor spectra as input to the equipment to partially compensate for the conservatism introduced in the generation of these spectra.

USE OF THE CONCEPT OF SEISMIC-INDUCED SCENARIOS IN SEISMIC MARGIN STUDIES

It is unnecessary to demonstrate seismic margin for all structures, components, and systems subjected to an SME. Instead, one should concentrate on the more likely

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This appendix is an excerpt from
EPRI Report NP-6041

Appendix B

Summary of CDFM Method Description From EPRI Report

Table 2-5.

SUMMARY OF CONSERVATIVE DETERMINISTIC FAILURE MARGIN APPROACH

Load Combination:	Normal + SME
Ground Response Spectrum:	Conservatively specified (preferably 84% Non-Exceedance Probability Site-Specific Spectrum, if Available)
Damping:	Conservative estimate of median damping
Structural Model:	Best Estimate (Median) + Uncertainty Variation in Frequency
Soil-Structure-Interaction:	Best Estimate (Median) + Parameter Variation
Material Strength:	Code Specified minimum strength or 95% exceedance actual strength if test data are available.
Static Capacity Equations:	Code ultimate strength (ACI), maximum strength (AISC), Service Level D (ASME), or functional limits. If test data are available to demonstrate excessive conservatism of code equations, then use 84% exceedance of test data for capacity equation.
Inelastic Energy Absorption: (ductility)	For non-brittle failure modes and linear analysis, use 80% of computed seismic stress in capacity evaluation to account for ductility benefits, or perform nonlinear analysis and go to 95% exceedance ductility levels.
In-Structure (Floor) Spectra Generation:	Use frequency shifting rather than peak broadening to account for uncertainty plus use median damping.

Source: (4)

Appendix C

Floor Spectra

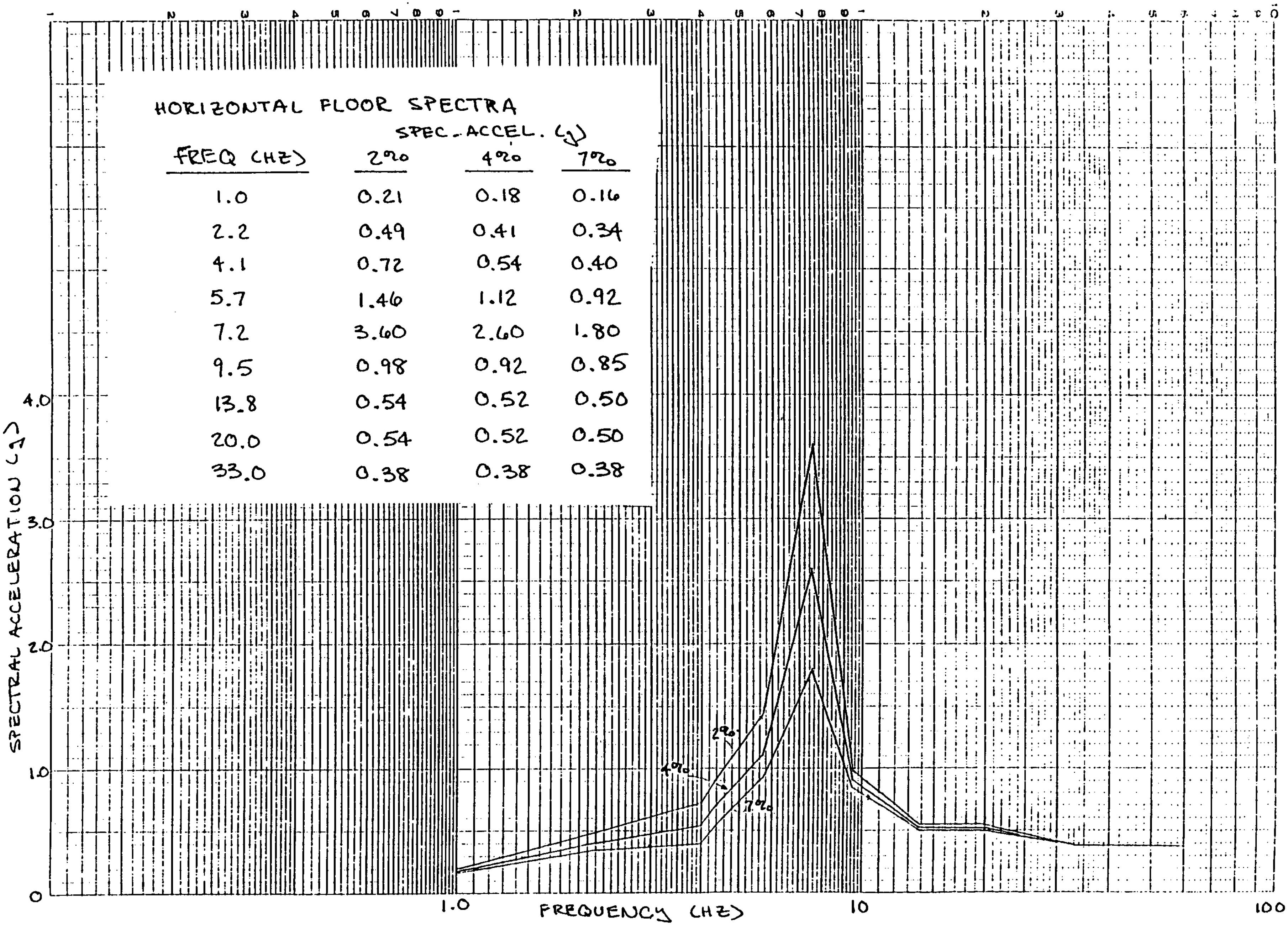
FLOOR SPECTRA

Attached are selected horizontal and vertical floor spectra at 2%, 4%, and 7% damping. They have been "debroadened", with some associated simplifications, from the broadened Maine Yankee turbine/service building spectra at EL 61'-0", E-W and vertical directions. A description of the dynamic model is contained on pp. 111 to 134 of Cygna's report (Appendix C of the prior transmittal). 7% structure damping was used in the dynamic analysis. Ground motion input consisted of time-histories matching the median NUREG/CR-0098 spectra scaled to 0.18g horizontal PGA and 0.12g vertical PGA.

Unbroadened floor spectra at the desired equipment frequencies are not contained in the Dresden SEP report (only 0.5% damping). Broadened horizontal spectra at 2%, 3%, and 7% dampings are available. Also, vertical spectra are not contained. The Maine Yankee spectra were selected as the basis for the "debroadened" spectra for the following reasons:

- Vertical spectra are available.
- The fundamental horizontal frequency is comparable (7.5 Hz vs. 5 Hz)
- In-structure spectral accelerations at the peaks and high frequencies are about the same as for the Dresden spectra at upper elevations, for nearly equivalent ground input.

While some information is lost towards the higher frequencies in the debroadening, the same problem would exist with the broadened Dresden spectra.

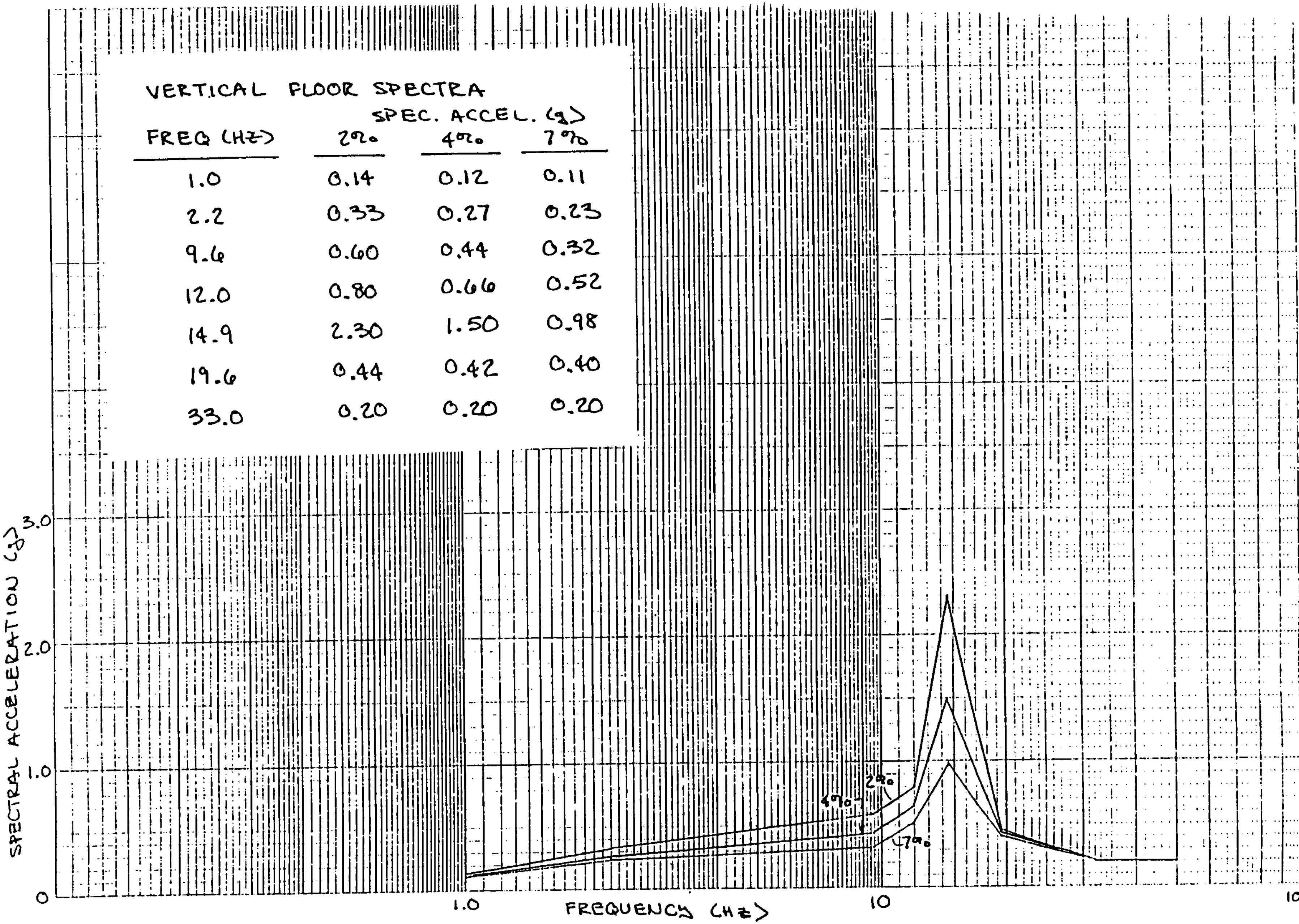


HORIZONTAL FLOOR SPECTRA

FREQ (HZ)	SPEC. ACCEL. (g) 2%	4%	7%
1.0	0.21	0.18	0.16
2.2	0.49	0.41	0.34
4.1	0.72	0.54	0.40
5.7	1.46	1.12	0.92
7.2	3.60	2.60	1.80
9.5	0.98	0.92	0.85
13.8	0.54	0.52	0.50
20.0	0.54	0.52	0.50
33.0	0.38	0.38	0.38

VERTICAL FLOOR SPECTRA

FREQ (HZ)	SPEC. ACCEL. (g) 2%	4%	7%
1.0	0.14	0.12	0.11
2.2	0.33	0.27	0.23
9.6	0.60	0.44	0.32
12.0	0.80	0.66	0.52
14.9	2.30	1.50	0.98
19.6	0.44	0.42	0.40
33.0	0.20	0.20	0.20



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Appendix D

Generic Seismic Ruggedness of Power Plant Equipment

Generic Seismic Ruggedness of Power Plant Equipment

NP-5223
Research Project 1707-15

Final Report, May 1987

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GERS-MCC.3

GENERIC EQUIPMENT RUGGEDNESS SPECTRUM

FOR

MOTOR CONTROL CENTERS
(LOW VOLTAGE)

Prepared for

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December 1986

1.0 INTRODUCTION

A Generic Equipment Ruggedness Spectrum (GERS) for Motor Control Centers (low voltage) is presented and discussed in the following sections.

2.0 EQUIPMENT DESCRIPTION

The equipment class covered by the GERS presented here is Low Voltage Motor Control Centers (MCC) which are steel enclosures containing various sizes of motor starters (contactors and control relays), circuit breakers, auxiliary relays, disconnect switches, control or distribution transformers, and panelboards. They may also have indicator lamps and meters. Cable or conduit entry can be from the bottom, top, or side. Units are low voltage rated at 600 VAC or 250 VDC. Typical low-voltage NEMA nominal enclosure section sizes are 20 inches wide, 14 to 20 inches deep, and 90 inches high. They are fabricated of 14 gage or heavier steel sheets, framed with angles, and supported on channels at the bottom. The base channels are either integral with the MCC frame or are external members connected by internal bolts to the MCC frame. Multiple MCC sections may be grouped together to make widths to 120 inches or greater. The units must be anchored at the base to a supporting structure. The validation of anchorage adequacy requires an independent evaluation. This equipment class covers virtually all low voltage MCCs used in power plants for critical motor control. The checklist given in Section 5 can be used to screen for outliers.

3.0 TEST DATA BASE

The data base includes basic equipment descriptive information, test methods/description, and test data covering a wide range of MCC for fifteen separate tests. The earliest test in the data base was conducted in 1974. Forty vertical MCC sections with weights ranging from 200 to 800 pounds (per section) from ten manufacturers which represent the range of units found in actual power plants are included in the data base. The units tested involved both single- and multi-section MCC units (up to six sections). Two of the tests included valid data; however, they do not meet the class inclusion rules. One of these tests had a top brace attachment (i.e., not entirely base-mounted), and the other MCC unit was housed in a non-typical (larger) enclosure. All units were mounted within NEMA-type metal enclosures with either welded or bolted anchorage.

Twelve tests were performed with random, independent, biaxial input motions. One test used random, independent, triaxial input motion, while two additional tests utilized single-axis sine-beat inputs. The test results span the entire range of possible success and failure. Failure modes are relay chatter and minor structural base damage. In one test series involving four MCC sections, the equipment fragility limits were sought. In some cases, the tests were performed on MCCs in which artificially aged components had been installed, while in others, the components were new. Typical parameters which are monitored during testing include contact chatter and coil dropout. Tests are typically performed in a deenergized state and then repeated with the circuits energized. Hi-pot tests, under- and over-voltage relay functionality, and circuit breaker functionality are checked before and after the tests.

In general, the functionality of MCCs is limited by auxiliary relay and motor starter auxiliary contact chatter in the deenergized state. Thus, the issue of MCC function during a dynamic event is governed by the ruggedness of the relays present. It should be noted that all MCC units, dynamically tested, functioned in post-test operation, including those units that sustained minor base damage.

4.0 GENERIC EQUIPMENT RUGGEDNESS SPECTRUM

Figure 1 compares the GERS to the horizontal Test Response Spectra (TRS) (standardized to 5% damping) for all of the thirteen TRS used to construct the GERS. Both the energized and the deenergized data from successful are compared in Figure 1. For this equipment class, dual GERS are proposed. The "function after" GERS accommodates the test data of several manufacturers over the frequency range of 1 to 33 Hz. Also the "function after" GERS accommodates the tests where minor structural damage (not affecting function) occurred. The "function during" GERS conservatively accommodates the low bound of data base TRS for which relay chatter was noted in a wide spectrum of cabinet and relay or starter types as shown in Figure 2. The vertical TRS in all tests on which the GERS is based was approximately equal to the horizontal input motion. Thus, the GERS presented is valid for concurrent vertical and horizontal motion.

5.0 CHECKLIST

To apply this GERS to Motor Control Centers, the following criteria must be verified.

- The MCC must be a low voltage unit with a floor-mounted NEMA-type enclosure with an average weight per vertical section that does not exceed 800 pounds (review of manufacturer's submittals is sufficient).

- The MCC must be base anchored and the installed anchorage must be evaluated (units which utilize a top brace attachment as part of the unit anchorage require a separate evaluation in order to justify the "function after" GERS limit; however, the "function during" GERS may be used directly with such units).

- The base anchorage must utilize the MCC base channels for attachment. Base anchorage details that induce significant bending of sheet are not acceptable.

- Cutouts in cabinet sheathing are less than 6-in. wide and 12-in. high in the lower half of the cabinet height.

- All door latches or screwdriver operated door fasteners must be secured.

- In order to utilize the "function during" GERS certain relays with low ruggedness must be excluded. All relays must have a GERS greater than 4.5 g within the amplified spectral region.

- Auxiliary contacts of contactors require a separate evaluation if they are used for interlocks or control signals. The "function during" GERS spectral levels must be factored by ~~0.67~~ 0.67 to be applied to the auxiliary contacts of contactors.

- If the "function during" GERS limits for MCCs are exceeded by certain plant floor response spectra, a separate relay evaluation is required which accounts for specific relay ruggedness, relay location within an enclosure, and enclosure amplification.

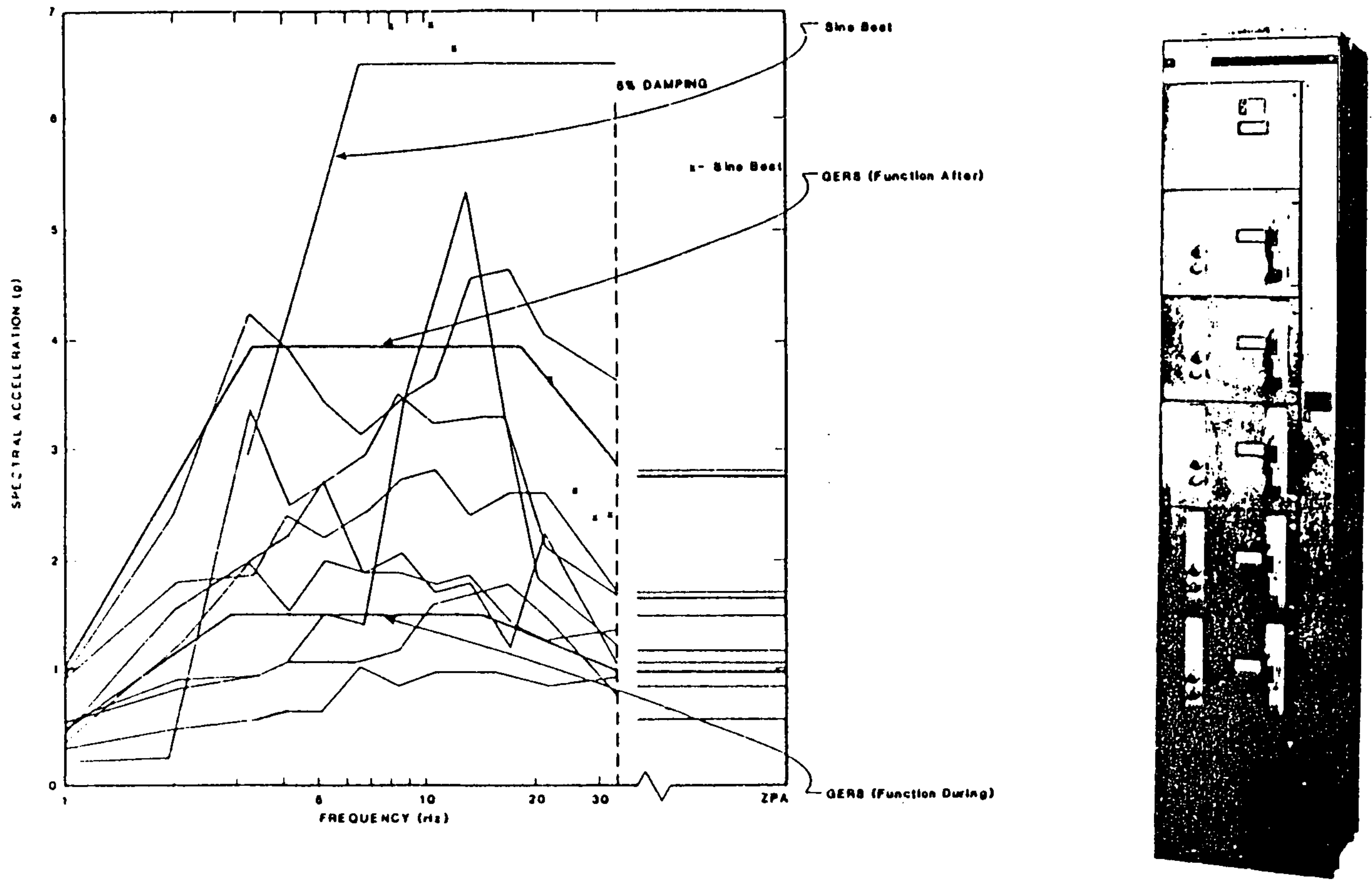

Figure 1. Comparison of GERS With Ruggedness Data: Function During and After for MCC

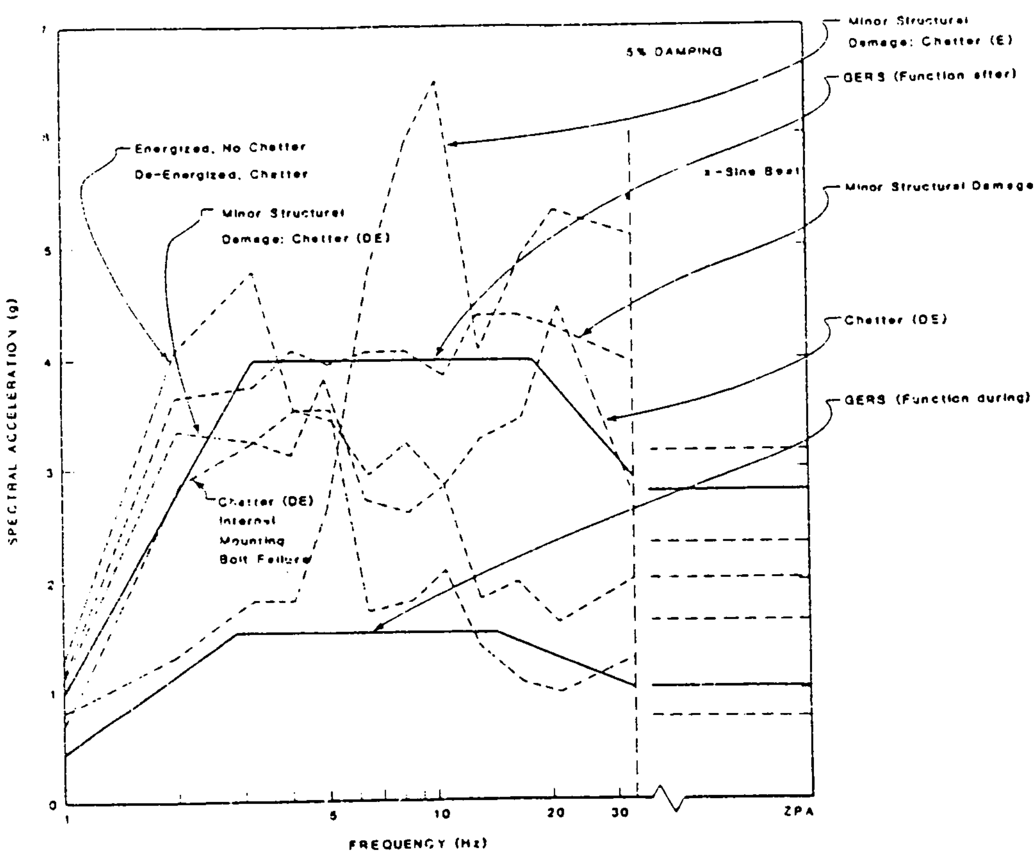

Figure 2. Comparison of GERS with failure data: function during and after for MCC.

Table 1

Comparison of HCLPF Capacity Computations for Representative Components (First Round Calculations)

Component	HCLPF Capacity (g) CDFM	HCLPF Capacity (g) FA	Median (g)	Capacity Failure Mode	Comments/Remarks/Assumptions
Flat Bottom Storage Tank (At Grade)					
RPK	0.29	0.31	0.67	Combination of shell buckling	
MKR/PSH	0.29	0.26	0.54	and anchor bolt yields	
JWR	-.---	0.27	0.53		
JDS	~~0.48~~ 0.32	-.---	1.13	Yield of anchor bolts	
Auxiliary Contactor Chatter (Function during GERS lock-in circuit potential)					
a) Cabinet at Grade					
RPK	0.54	0.59	1.26	Contactor Chatter	
MKR/RDC	0.47	0.39	1.58	Contactor Chatter	(used .87 knock-down factor)
JWR	-.---	0.48	1.20	Contactor Chatter	
JDS	0.71	-.---	1.88	Contactor Chatter	
b) Cabinet High-up (Function during GERS lock-in circuit potential)					
RPK	0.10	0.11	0.30	Contactor Chatter	
MKR/RDC	0.09	0.07	0.36	Contactor Chatter	(used .87 knock-down factor)
JWR	-.---	0.11	0.43	Contactor Chatter	
JDS	0.12	-.---	0.43	Contactor Chatter	

RPK — Did calculations by CDFM (by EPRI methodology) first, then FA.
MKR/RDC/PSH — Did calculations by FA first, then CDFM (by EPRI methodology).
JWR — Tabulated values are from HCLPF Capacity calculations using input spectra as 84% NEP maximum horizontal direction.
JDS — Did calculations by CDFM (by deterministic approach).

Table 2

Comparison of HCLPF Capacity Computations for Representative Components (Second Round Calculations)

Component	HCLPF (g)	Median Capacity (g)	Capacity Failure Mode	Comments/Remarks/Assumptions
Flat Bottom Storage Tank (At Grade)				
RPK	0.29	0.67	Combination of shell buckling	
MKR/PSH	0.29	0.54	and anchor bolt yields	
JWR	0.28	0.55		
JDS	0.32	0.83	Yield of anchor bolts and separation of base plate from wall plate	
Auxiliary Contactor Chatter (Function during GERS lock-in circuit potential)				
a) Cabinet at Grade				
RPK	0.54	1.26	Contactor Chatter	
MKR/RDC	0.47	1.58	Contactor Chatter	(used .87 knock-down factor)
JWR	0.48	1.20	Contactor Chatter	
JDS	0.71	1.88	Contactor Chatter	
b) Cabinet High-up (Function during GERS lock-in circuit potential)				
RPK	0.11	0.30	Contactor Chatter	
MKR/RDC	0.09	0.36	Contactor Chatter	(used .87 knock-down factor)
JWR	0.11	0.43	Contactor Chatter	
JDS	0.15	1.88*	Contactor Chatter	

* See calculations (Appendix A) for further explanation.

Table 2 (Continued)

Comparison of HCLPF Capacity Computations for Representative Components (Second Round Calculations)

Component	HCLPF (g)	Median Capacity (g)	Capacity Failure Mode	Comments/Remarks/Assumptions
Starting Air Tank (High-up)				
RPK	0.48	1.07	Plastic Bending of Mounting Angles	
MKR/RDC	0.53	1.55	Plastic Bending of Mounting Angles	
JWR	0.43	1.40	Plastic Bending of Mounting Angles	
JDS	0.42	1.10	Plastic Bending of Mounting Angles	
Heat Exchanger (High-up) (Bolted to Rigid Support Frame)				
RPK	0.40	1.18	Anchor Bolt Shear Failure; failure through the threads	
MKR/RDC	0.44	1.08	Anchor Bolt Shear & Tension Failure	
JWR	0.39	1.00	Anchor Bolt Shear Failure; failure through the threads	
JDS	0.44	1.15	Anchor Bolt Shear Failure	
Block Wall (High-up)				
RPK	0.62	1.94	Out-of-Plane Bending	
MKR/PSH	0.63	2.10	Out-of-Plane Bending	
JWR	0.52	1.96	Out-of-Plane Bending	
JDS	0.31	1.30	Out-of-Plane Bending	HCLPF could be as much as 0.63g depending on the change in wall stiffness (hence frequency) prior to failure

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Seismic margin review of nuclear power plants requires that the High Confidence of Low Probability of Failure (HCLPF) capacity be calculated for certain components. The candidate methods for calculating the HCLPF capacity as recommended by the Expert Panel on Quantification of Seismic Margins are the Conservative Deterministic Failure Margin (CDFM) method and the Fragility Analysis (FA) method. The present study evaluated these two methods using some representative components in order to provide further guidance in conducting seismic margin reviews. It is concluded that either of the two methods could be used for calculating HCLPF capacities.

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