5A.3 DESIGN BASES

5A.3.1 CATEGORY I STRUCTURE DESIGN

5A.3.1.1 Time History

The time-history analysis was utilized in the seismic analysis of Category I structures to construct the floor response-spectrum curves. These curves were used as input in the evaluation of equipment and piping systems. The response-spectrum curves were generated utilizing El Centro, California, 1940 earthquake. Horizontal (East-West) components were scaled down to the 8% gravity maximum acceleration specified for the OBE. The vertical spectrum response component was scaled down to the 5.3% gravity maximum acceleration specified for the OBE.

The time-history response at the mass points consists, in general, of a superposition of the time-varying responses admitted through narrow frequency bands whose central frequencies are the natural frequencies of the system. The ground spectra curves of El Centro E-W component do envelope the design response spectra with a considerable margin at or near the natural frequencies of all Category I structures, thus assuring conservatism of the earthquake time history.

A parametric study was performed to investigate the effects of the variations in the basic time-history input. The El Centro E-W time-history accelerograph was modified by passing it successively through arbitrary filters, so that the new accelerograph produced a smooth ground spectra which completely enveloped the 2% design spectra reducing the valleys and peaks. This modified time history was then applied to the structures and new floor response curves obtained. The resulting curves were compared to those produced by the actual earthquake and found to be very similar in shape, but lower in magnitude at all frequencies. It was concluded that the use of the original earthquake time history was more conservative.

The fact that the original earthquake time-history responses are higher than those of design response spectra attests to the above conclusion.

The Containment Structure seismic analysis results obtained by both the time-history and the response-spectrum methods are provided in Tables 5A-2, 5A-3, and 5A-4. Table 5A-1 identifies the location of the structural model points for which responses are tabulated. The results of the response-spectrum method for the 8% gravity OBE, and for the 15% gravity SSE, are presented in Tables 5A-2 and 5A-3, respectively. Table 5A-2 presents the responses to El Centro E-W component that has been linearly scaled down to have an 8% gravity maximum acceleration, and used as the operating basis time history. The tabulated responses indicate that:

a. The relative response of structural points with respect to each other compare well for both methods.
b. The results of the 8% gravity time history are nearly equivalent to the results of the frequency response method for 15% gravity, thus reflecting the conservatism in use of the El Centro earthquake.

5A.3.1.2 Ground Spectra Curves

The ground spectra curves of the El Centro E-W component are shown in Figures 5A-1 through 5A-4. These curves are drawn over the design response spectra curves for comparison and are considerably above the spectra in the damping range of 2 to 5% of the critical, reflecting a high safety margin.

Computations of the spectrum curves from the digitalized time-history records were accomplished utilizing Bechtel Computer Program CE-791. This program is basically a fourth order Runge-Kutta stepwise numerical solution of the equation of motion and calculates, prints, and plots the absolute values of maximum displacements, velocities, and spectral accelerations for a total of 124 characteristic frequencies ranging from 0.1 to 25.0 CPS, for a given critical damping value.

5A.3.1.3 Confidence Limits

The confidence limits of the fourth order Runge-Kutta algorithm exceed the round-off error encountered in the single-precision accuracy of GE-635 (a digital electronic computer used by Bechtel) for the above frequency range and for the 0.01 second time interval on which the digital accelerogram is based. The frequency interval, in CPS, is 0.05 between frequencies 0.1 and 1.0, 0.10 between frequencies 1.00 and 10.0, and 1.00 between frequencies 10.0 and 25.0, thus describing the spectrum with 124 coordinates. These frequency intervals produce spectra which compare well with spectra produced by analog computers for all damping values, except 0.0% of critical for which no interval can be defined with certainty.

5A.3.1.4 Amplified Response Loading for Structure and Floors

The response of the structure and floors to the SSE and OBE was computed using the response-spectrum technique. This method, described in Section 5.1.3.2.b, computes the horizontal response in the direction that will give the maximum stresses.

For the response in vertical direction, the structure was reduced into a one-degree-of-freedom system. The responses (shears, moments, and inertia forces) in both vertical and horizontal directions were combined to produce maximum loading.

Since the frequencies of a structure cannot be precisely determined because of material property variations, lumping of masses, idealization of stiffness (both structure and foundation), and nonlinear response characteristics of actual structures, the spectrum response curves obtained for the above-ground elevations in structures were modified by a shifting of the peak response by an approximate amount of 10% on both sides of the original curve. This was done to ensure against any uncertainties of the important variables in the structural model.

A multi-mass and multi-degree of freedom analysis was the original intent for determining the response of Category I structures in the vertical direction. However, the thick concrete members of the structures produced a large extensional stiffness in the vertical direction with respect to the relatively lower soil stiffness and, hence, resulted in a singular flexibility matrix. An investigation showed that the vertical degrees of freedom were constrained, that is, they moved approximately the same amount at the same time. Therefore, the constrained vertical degrees of freedom were reduced into one independent vertical degree of freedom for the response spectra generation.

The natural frequencies of the structure and the location of the amplified spectral values were subjected to some error due to assumptions made in the development of the analytical model, the possible variation in the building mass, and the stiffness calculated for the analytical model. The spectrum response curves were modified by shifting a peak response by an approximate amount of 10% on both sides of the original curve. This was done to ensure against any uncertainties of the important variables.

5A.3.1.5 Amplified Response Loading for Equipment and Components

The spectrum response curves, for Category I equipment and components, were generated using the time-history technique of seismic analysis. These curves were generated for two horizontal directions (North-South and East-West) and for the vertical direction, using various damping values at designated floor elevations (see Table 5A-8).

The seismic analyses are based on elastic and linear behavior of all components involved, and as such, does not include any gradual or accidental deterioration of the structure. The blowdown forces associated with a concurrent LOCA are computed separately and combined with the seismic loads.

The following was included in the analysis of all Category I equipment design:

a. Applicable response curve information from the specification concerning equipment location, and the appropriate damping value (see Table 5A-8).
b. Evaluation of the natural frequency(s) in both horizontal and vertical directions. The horizontal frequency(s) shall be computed in the direction that will give the highest stresses.
c. Utilizing the natural frequency(s), enter the applicable spectrum response curve to ascertain the comparable response acceleration.
d. For equipment and systems modeled as multi-degree systems, the acceleration per mode shall be combined by the normal mode method. The effect of adding closely spaced modes linearly has been investigated. It has been found that combining stress components by this method does not exceed the allowable stress levels.
e. The horizontal and vertical seismic forces are equal to the mass of the equipment times the respective spectrum acceleration.
f. Horizontal and vertical forces are applied simultaneously at the center of gravity of the equipment, or at lumped mass point.
g. Seismic stresses shall be computed and combined with all other stresses that might exist in critical components.
h. Stresses from the OBE, when combined with normal operating stresses, shall be kept at or below the applicable code allowable stresses.
i. Stresses from the SSE, when combined with normal operating stress, shall be kept at or below the applicable code allowable stresses, and shall be kept at or below yield strength of the material provided that no loss of function can occur.
j. For equipment, where analysis is not reliable, vibration tests shall be employed to verify the seismic adequacy of the equipment. The test procedure shall be submitted for approval.

5A.3.1.6 Amplified Response Loading for Piping and Instrumentation

A multi-mass response-spectrum, modal analysis method was employed in the seismic analysis of Category I piping, support systems and instrumentation. American Society of Mechanical Engineers (ASME) Code Case N-411 may be used to take advantage of the flexibility in piping systems (Section 5A.3.2.2). The natural frequencies, mode shapes, and the maximum response accelerations were determined using the appropriate response-spectrum curves in the horizontal and vertical direction. The response-spectrum curves are generated using the time-history of the floor, which includes the seismic response of the building. The horizontal and vertical seismic forces were applied simultaneously. Shear stresses, moments, and deflections were determined for the piping system and restraints. The load and stresses due to seismic loadings were assumed to be acting simultaneously with operating weights and longitudinal pressure loads.

5A.3.1.7 Normal Operation

For loads to be encountered during normal plant operation (including OBE loads), Category I structures are designed in accordance with design methods of accepted standards and codes insofar as they are applicable.

5A.3.1.8 Loss-of-Coolant Accident, Seismic and Tornado Loads

The Category I structures are in general proportioned to maintain elastic behavior when subjected to various combinations of dead loads, thermal loads, LOCA loads, seismic and tornado loads. The upper limit of elastic behavior, considered to be the yield (Y) for reinforced concrete structures, is considered to be the ultimate resisting capacity as calculated from the "Ultimate Strength Design" portion of the ACI-318-63 code when ϕ is taken as unity. Reinforced concrete structures are designed for ductile behavior whenever possible, i.e., with steel stresses controlling the design. For structural steel, the allowable stresses are per the American Institute of Steel Construction Manual, 6th Edition.

The load factors and load combinations represent the consensus of a group of individual engineers and consultants who are experienced in both structural and nuclear power plant design. Additionally, their judgments have been influenced by current and past practice, by the degree of conservativeness inherent in the basic loads, and particularly by the probabilities of coincident occurrences in the case of incident, wind, and seismic loads.

Factored load equations used for the design of Category I structures, except the Containment Structure, are in accordance with the equations presented in the ACI-318-63 code. The following discussion will explain the justification of individual load factors:

LOAD FACTORS

a. Dead Load -- Dead load in a large structure such as this is easily identified and its effect can be accurately determined at each point in the vessel. For combination with accident and seismic or wind loads, a load factor representing a tolerance of 5% was chosen to account for inaccuracies. The ACI code allows a tolerance of +25% and -10%, but was written to cover a variety of conditions where weights and configurations of materials in and on the structure may not be clearly defined and are subject to change during the life of the structure.

b. Live Load -- The live load in combination with accident, seismic, and wind loads produces a very small portion of the stress at any point. It is extremely unlikely that the full live load would be present over a large area at the time of an unusual occurrence. Therefore, a low load factor is felt to be justified, and live load will be considered together with dead load at a load factor of 1.05.

c. Seismic -- The design earthquake is considered to be the strongest probable earthquake which could occur during the life of the plant. In addition, a maximum earthquake which could occur at the site is also considered in design. Category I structures are designed so that no loss of function would result. Consequently, the probability of a LOCA is very small. For this reason the two events, seismic and LOCA, are considered together, but at much lower load factors than those applied separately. The earthquake load factors of 1.25 and 1.0 are conservative for the design and maximum earthquakes combination with the factored LOCA.

d. Winds -- Loads are determined from the design tornado wind speed. The Containment Structure is designed for this extreme wind and it is inconceivable that it would cause a LOCA. Therefore, wind loads will not be considered with incident loads, but a factor of 1.25 will be applied to the tornado load to provide assurance of the structure performing satisfactorily.

e. Loss-of-Coolant Accident -- The design pressure and temperature are based on the operation of partial safeguards equipment using emergency diesel power.

   European practice has been to use a load factor of 1.5 on the design pressure (Reference 1). This factor is reasonable and has been adopted for this design. The probabilities of a LOCA occurring simultaneously with a maximum wind or seismic disturbance are very small; therefore, a reduced load factor of 1.25 is used.

   In all cases the design temperature is defined as that corresponding to the factored pressure of 1.5P, and will be somewhat higher than that temperature at P. A temperature factor of 1.5 is unrealistic since this could only occur with a pressure much greater than that of 1.5P.

The ϕ factors are provided to allow for variations in materials and workmanship. In ACI Code 318-63, ϕ varies with the type of stress or member considered; that is, with flexure, bond or shear stress, or compression.

The ϕ factor is multiplied into the basic strength equation or, for shear, into the basic permissible unit shear to obtain the dependable strength. The basic strength equation gives the "ideal" strength assuming materials are as strong as specified, sizes are as shown on the drawings, the workmanship is excellent, and the strength equation itself is theoretically correct. The practical, dependable strength may be something less since all these factors vary.

The ACI Code provides for these variables as indicated above by specifying appropriate ϕ values. These values are larger for concrete flexure because the variability of steel is less than that of concrete and the concrete in compression has a fail-safe mode of behavior; that is, material understrength without failure. The values for columns are lower (favoring the toughness of spiral columns over tied columns) because columns fail in compression where concrete strength is critical. It is possible that the analysis might not combine the worst combination of axial load and moment. Since the column member is critical in the gross collapse of the structure, a lower value is used.

The additional ϕ values used represent the best judgment of how much understrength should be assigned to each material and condition not covered directly by the ACI code, and have not been selected based on material quality in relation to the existing ϕ factors.

Conventional concrete design of beams requires that the design be controlled by yielding of the tensile reinforcing steel. This steel is generally spliced by lapping in an area of reduced tension. For members in flexure, ACI used ϕ = 0.90 to reinforcing steel which now includes axial tension. The code recognizes the possibility of reduced bond of bars at the laps by specifying a ϕ of 0.85.

Mechanical and welded splices will develop at least 125% of the yield strength of the reinforcing steel. Therefore, ϕ = 0.85 is recommended for this type of splice.

The only significantly new value introduced is ϕ = 0.95 for prestressed tendons in direct tension. The higher ϕ value has been allowed because: (1) during installation, the tendons are each jacked to about 94% of their yield strength so, in effect, each tendon has been proof tested; and (2) the method of manufacturing prestressing steel (cold drawing and stress relieving) ensures a higher quality product than conventional reinforcing steel.

The final design of Category I structures (except the Containment Structure) satisfied the most severe of the following load combination equations.

$S \geq D + L + T + E$
$Y \geq 1/\phi\ (1.25D + 1.0R + 1.25E)$
$Y \geq 1/\phi\ (1.25D + 1.25T + 1.25E)$
$Y \geq 1/\phi\ (1.0D + 1.0R + 1.0E')$
$Y \geq 1/\phi\ (1.0D + 1.0T + 1.0E')$

The final design of the Containment Structure satisfies the following load combinations and factors (factored load cases):

$Y \geq 1/\phi\ (1.05D + 1.5P + 1.0T_A + 1.0F)$
$Y \geq 1/\phi\ (1.05D + 1.25P + 1.0T_A + 1.25H + 1.25E + 1.0F)$
$Y \geq 1/\phi\ (1.05D + 1.25H + 1.0R + 1.0F + 1.25E + 1.0T_o)$
$Y \geq 1/\phi\ (1.05D + 1.25H + 1.0F + 1.25W + 1.0T_o)$
$Y \geq 1/\phi\ (1.0D + 1.0P + 1.0T_A + 1.0H + 1.0E' + 1.0F)$
$Y \geq 1/\phi\ (1.0D + 1.0H + 1.0R + 1.0E' + 1.0F + 1.0T_o)$

(Wind, W, is to replace earthquake, E, in the above formula where wind stresses control)

(0.90 D is used where dead load reduces the critical stress in the first two equations).

Limiting yield allowables and allowable erosion of barriers under all load conditions, including the SSE (E') and jet or missile forces, respectively; is acceptable; provided deflections are also checked to ensure the affected Category I systems and equipment do not suffer loss of function. The Containment Structure must also retain its required leak-tight integrity under LOCA loadings.

S = Strength of structures with stresses ≤ their applicable code allowable value.

Y = Strength of structure with stresses not exceeding their yield value. For structural steel, it is the minimum specified yield strength. For reinforced concrete, it is the reinforcement yield strength. All accident loads, R, are considered purely dynamic and an increase in the static yield strength of the material have been given consideration as recommended in the American Society of Civil Engineers (ASCE) manual #42.

D = Dead load of structure and equipment plus any other permanent loads contributing stress, such as soil or hydrostatic loads. In addition, a portion of "live load" is added when such load is expected to be present when the plant is operating. An allowance is also made for future permanent loads.

L	=	Live Loads
R	=	Forces on structure due to a rupture of any one primary coolant pipe including the following: Reaction forces transmitted to the major component support structures. Jet impingement force on the structural component that can become a target to the ensuing jet. Differential pressures that can develop across structural compartments.
H and T	=	force on structure due to thermal expansion of restrained pipes under operating conditions.
E	=	OBE load from horizontal ground acceleration of 0.08 g.
E'	=	SSE load from horizontal ground acceleration of 0.15 g.
W	=	Tornado wind load.
P	=	LOCA pressure load.
F	=	Final prestress load.
T_A	=	Thermal load due to the incident temperature gradient through the wall and expansion of the liner. It is based on a temperature corresponding to the factored LOCA pressure.
T_o	=	Thermal load due to the normal operating temperature gradient through the walls.
ϕ	=	Yield capacity reduction factor as follows: 0.90 for reinforced concrete in flexure. 0.85 for tension, shear, bond, and anchorage in reinforced concrete. 0.75 for spirally reinforced concrete compression members. 0.70 for tied compression members. 0.90 for fabricated structural steel. 0.90 for unprestressed reinforcing steel in direct tension. 0.95 for prestressed tendons in direct tension.

The Containment Structure and engineered safety system components are protected by barriers from all credible missiles that might be generated from the primary system during a LOCA.

The final design of the missile barrier and equipment support structures inside the Containment were reviewed to assure that they can withstand applicable pressure loads, jet forces, pipe reactions and earthquake loads without loss of function. The deflections or deformations of structures and supports were checked to assure that the functions of the Containment and Engineered Safety Features are not impaired.

<u>SAFETY FACTORS</u>

The safety factor that would result from the factored load equations of ACI 318-63 code are based on the ratio of live load to dead load. This factor varies from 1.67 to 1.93 for a ratio of live load to dead load factor of 4. The safety factor that is actually provided from the use of the factored load equations used for the design of Containment Structure depends on the ratio of various loading combinations to the dead load. The safety factors are well within the range specified in ACI 318-63 code, to make failures very unlikely.

5A.3.1.9 Tornado Forces

All Category I structures and critical components of Category I structures are designed to resist a lateral force caused by a tornado having a velocity of 300 mph and a forward progression of 60 mph. There are no removable shielding blocks located on the external boundary of any Category I structure.

Category I structures are designed to resist the effects of a tornado. These structures are analyzed for tornado loading (not coincident with LOCA or earthquake) on the following basis:

a. Differential bursting pressure between the inside and outside of the Containment Structure is assumed to be 3 psi positive pressure.

   For the safety related Diesel Generator Building, the tornado-induced pressure differential is applied as a 3 psi positive pressure within the Diesel Generator Buildings occurring in 1.5 seconds (2 psi per second) followed by a calm for two seconds and then a repressurization to atmospheric pressure at a rate of 2 psi per second.

b. Lateral force is assumed as the force caused by a tornado funnel having a peripheral tangential velocity of 300 mph and a forward progression of 60 mph. The applicable portions of wind design methods described in ASCE Paper 3269 are used, particularly for shape factors. The provisions for gust factors and variation of wind velocity with height do not apply.

   For the safety-related Diesel Generator Building, the velocity components are applied as a funnel of wind traveling at 70 mph with a maximum tangential velocity of 290 mph (giving a total effective wind velocity of 360 mph).

c. Torsion of the Containment Structure is computed from the drag on a cylinder resulting from a 300 mph rotary wind centered over the structure.

d. A tornado-driven horizontal (i.e., no vertical velocity component) missile equivalent to a 4000 pound automobile flying horizontally through the air at 50 mph and at not more than 25' above the ground or a 4"x12"x12'-long piece of wood traveling end-on at 300 mph at any height.

   At the time of the original design and licensing of the plant, design-criteria for non-horizontal missiles (i.e., missiles with a vertical velocity component) did not exist and the site is not committed to any specific criteria for vertical missiles; with one exception being the safety-related Diesel Generator Building. Changes to the plant design will provide a similar level of protection as existed in the original licensed design. Since the safety-related Diesel Generator Building was constructed subsequent to the original design and licensing design of the plant, its design criteria is specifically mentioned below.

   The safety-related Diesel Generator Building is designed for missile impingement for the missiles given in Table 5-8. A vertical velocity of 70 percent of the postulated horizontal velocity is used for all missiles except for steel rods. A steel rod missile’s vertical velocity is assumed to be equal to the horizontal velocity.

e. All exposed Containment penetrations are designed for tornado forces.

f. The possible increase of the tornado loading on the Containment Structure due to the funneling effect of the tornado wind blowing between the two Containment Structures has also been considered.

Except for local crushing at the missile impact area, the allowable stresses to resist the effects of tornadoes are 90% of the yield of the reinforcing steel and 85% of the ultimate strength of the concrete.

Tornado-generated missile protection is not required for systems designed to meet the performance standards of draft General Design Criteria 2 if the resultant aggregated probability of exposures in excess of 10 CFR Part 100 guidelines is less than 10^{-6} per year per unit. The aggregate probability includes reasonable qualitative arguments, or conservative assumptions, such that the realistic probability can be shown to be lower than the calculated value.

Probabilistic Evaluation Techniques were used to determine the probability of exposures in excess of 10 CFR Part 100 guidelines due to tornado missiles. The evaluation was performed for the equipment listed in Table 5A-5. The evaluation determined that the aggregate probability of exposures in excess of 10 CFR Part 100 guidelines is less than 5E-07 per unit per year (Table 5A-5). The key parameters in the evaluation which could be affected by site activities are the total missile population, modifications that significantly change the size of the component target areas, and changes to plant equipment, procedures, or practices which would affect the Probabilistic Risk Assessment. This analysis conservatively assumed that:

1. The tornado causes a loss of offsite power which is not recoverable.
2. Tornado missile damage is not recoverable.
3. Guaranteed failure of the component is assumed to occur from a tornado missile strike to the exposed components as listed on Table 5A-5 with exceptions described in 4 and 5 below.
4. A tornado missile strike to any part of the array of 16 MSSV and 2 ADV vent stacks has a 1 in 100 chance crimping enough stacks to fail the steam generator decay heat removal function.
5. A tornado missile strike to the 1A EDG out exhaust ducts a 1 in 100 chance of failing the 1A EDG.
6. Tornado Point strike frequency is based on NUREG/CR-4661, Revision 2. (This frequency includes relatively recent tornado events.) This value is conservative because it includes tornadoes of all intensities, including the smaller and more frequent tornadoes which generally do not produce significant tornado missiles.
7. Core damage will result in exposures in excess of 10 CFR Part 100 guidelines. This assumption is conservative in that it assumes each core damage event results in a release, i.e., the containment function is guaranteed to fail. Use of this conservative assumption alleviates the need to calculate Large Early Release Frequency for tornado missiles. The threshold for acceptable risk is in the core damage frequency metric only.

5A.3.1.10 Seismic Forces (E and E')

Atomic Energy Commission publication TID 7024, "Nuclear Reactors and Earthquakes," is used as the basic design guide for seismic analysis. All Category I structures are designed for the loading combinations described in Chapter 5.

The "OBE" used for this plant is a maximum ground acceleration of 0.08 g horizontally and 0.053 g vertically, acting simultaneously. The "SSE" is a ground acceleration of 0.15 g horizontally and 0.10 g vertically, acting simultaneously. The maximum occurring vertical and horizontal accelerations are added directly with stresses developed from other load conditions.

Seismic loads on structures, systems, and equipment are determined by realistic evaluation of dynamic properties and the accelerations from the acceleration spectrum curves in Chapter 2.

Containment Structure Exterior:

The summary of stresses at critical locations in the Containment Structure exterior are listed in Table 5-1. Table 5-1 is the original analysis. See Appendix 5E for later tables associated with an evaluation that reduced the original containment minimum design prestress. The contributions of seismic stresses to total stresses in the Containment Structure exterior are shown in Table 5-1. The summary of stresses in the Containment Structure for different loading combinations such as dead loads, live loads, prestress pressure, temperature are listed in the table with and without seismic stresses. The technique used to combine seismic stresses with other stresses is described in Section 5.1.3.2. It can be seen from Table 5-1 that the contribution of seismic stresses to the total stresses in the Containment Structure varies widely; but, the total stresses including the seismic stresses are well within the allowable stresses. (Table 5-1 is the original analysis. See Appendix 5E for later tables associated with an evaluation that reduced the original containment minimum design prestress.)

Containment Structure Interior:

The maximum contribution of the seismic stresses to the total stresses for the Containment Structure interior is about 66% in the reactor cavity wall during an OBE, but the design of the reactor cavity wall is governed by an accident loading condition for which the contribution of seismic stresses during SSE is only 26% of the total stresses.

Auxiliary Building and Intake Structure:

In other Category I structures such as the Auxiliary Building and the Intake Structure, the contribution of seismic stresses to the total stresses varies widely, but under no conditions do the total stresses, including the seismic stresses, in these structures or their components, exceed the allowable stresses for concrete, reinforcement or structural steel, as defined in Chapter 5.

5A.3.1.11 Torsional Modes of Vibration

Torsional modes were not considered in the seismic modal analysis of Category I structures, but were calculated separately. In symmetrical Category I structures, torsional modes were not present. For non-symmetrical Category I structures, torsional moments were calculated using the following method:

a. The center of twist along with the center of gravity was obtained during the calculation for modal properties. The torsional moments were calculated as the product of resultant shear force and distance between the center of gravity and the center of twist.

b. Torsional shears were added to the shears due to lateral loading. However, when they acted in the opposite direction to the lateral shears, the greater shear values were used without reduction.

During an OBE and SSE the torsional modes of vibration of a structure may result from: (1) the unsymmetrical plan or elevation of the structure or the irregular arrangement of walls leading to a discord between the center of gravity and that of rigidity, and (2) the earthquake itself arising from rotational characteristics of the earthquake wave. The torsional loads in symmetrical structures are a result of the latter case, and Newmark's paper deals mainly with this type of torsion. Generally the flexural and torsional modes due to the first cause tend to be coupled, while those due to the second cause are not coupled. From the number of papers published to date (April 25, 1972), it can be stated that torsional loads are of significance to conventional multi-storied high-rise buildings with open moment-resisting frame due to their low torsional resistance.

Containment and Auxiliary Building

The structures considered to be Category I are the Containment Structure, Auxiliary Building and intake structure. The Containment Structure and the Auxiliary Building are respectively of closed circular and rectangular sections with concrete wall thicknesses of 3'9" for the former and 2' or more for the latter. The torsional resistance of these structures is considerably higher than that in any conventional structure, especially in the case of the Containment.

Intake Structure

The intake structure consists of concrete walls with thicknesses of 2' or more, but is not of closed sections. Based on theory of strength of materials, the torsional resistance of open sections is less than that of closed sections. In view of this, an analysis of the torsional effects on the intake structure based on the method as described has been made and the increase in shear stress is found to be in the range of 5 to 10%. The horizontal force to cause the torsional shear is that obtained from the uncoupled flexural vibration analysis. It has been found that rectangular buildings with either central cores or peripheral shear walls as resistive elements tend to have relatively uncoupled modes, while a smooth, even distribution of columns can result in strong modal coupling. In the light of such finding, it is believed that the horizontal force thus used in computing the torsional shear is reasonably accurate.

On the basis of the results of the torsional analysis of the intake structure, it was further concluded that the effects of the torsional loads on the Containment Structure and the Auxiliary Building should be of lesser significance and, therefore, no torsional analysis was made on these two structures.

Safety-Related Diesel Generator Building

Torsional effects for the safety-related Diesel Generator Building are accounted for directly in the seismic analysis through the incorporation of torsional degrees of freedom into the models which represent the Diesel Generator Building enclosure, the diesel generator pedestal and the fuel oil storage tank. In addition, torsional effects are represented in the building enclosure model by eccentrically located masses and by offsetting the beam elements representing the stiffness of the various levels in each Diesel Generator Building. Accidental torsion is accounted for by increasing the mass eccentricities by 5% of the maximum lateral building dimension.

5A.3.1.12 Foundation Isolation Joints

The foundations of the main structures are closely spaced and foundation isolation joints have been provided. Because the maximum separation of foundation is small in comparison with the length of the earthquake wave, it is unlikely that the foundations will be displaced, relative to one another, sufficiently to damage each other.

The differential movement of adjacent structures due to seismic motion was evaluated. The size of the isolation joint is based on the anticipated horizontal movement of the foundation during OBE and SSE. The isolation joint is filled with compressible material to reduce the influence of the foundations of the main structures on each other. Flexible joints are provided between the Containments and Auxiliary Building to serve as water stops and partial air leakage barriers.

5A.3.1.13 Soil/Foundation Interaction

Interaction between soil and the foundation is included in the seismic analysis in the form of soil spring constants. As shown in Figure 5-4 (Sheets 1 and 2), soil below the base slab of the Containment structure was included in the Finite Element Mesh and used in Bechtel's Finite Element Method Analysis computer program, CE 316-4, to compute the stresses generated in the soil and base slab. Figure 5-4 is based on the original analysis. See Appendix 5E for an evaluation that reduced the original containment minimum design prestress.

Stresses generated in the soil were included in the computation for the soil bearing pressure.

The total stresses in the soil, including those during the OBE and the SSE, were well below allowable bearing capacity of the soil. The total stress in the base slab of the Category I structure was also below allowable design values.

The effect of the soil on the sides of the walls below finish grade was negligible and was not included in the analysis. However, the walls below finish grade were designed for the dynamic earth pressure as well as static earth pressure (Section 2.7.6.4).

5A.3.1.14 Design Code References

The design and checking of the design have been made in accordance with the provisions indicated in the ACI Code and Commentary 318-63, Section 2603(a), 2603(b) and ACI Committee 334 (Concrete Shell Structures Practice and Commentary), Section 202(d), 202(e) and Commentary Part 4, except as modified in Updated Final Safety Analysis Report Sections 5.1.2.3 through 5.1.2.6 and Section 5.1.3.2.

5A.3.2 SEISMIC CATEGORY I SYSTEMS AND EQUIPMENT DESIGN

Seismic Category I systems and equipment, including pipe, are designed to meet the load combinations and stresses as stated in Table 4-8 for Nuclear Class 1, and Table 5A-6 for Nuclear Class 2 and 3, and non-class. Seismic Category I systems and equipment are bolted down rigidly to supports or braced (as in the case of cable tray supports) to resist seismic and tornado forces. The NSSS contractor is taking exception to this support approach and the individual supports were designed based on the criteria outlined in Sections 5.1.1 and 5.1.2.3. There are no significant gaps between the equipment and their supports or restraints. Any small gap will not cause significant impact forces on the equipment, restraints or the structures. Therefore, small gaps between the equipment and supports or restraints are not significant in the consideration of the seismic analysis.

Deformations in support structures will limit strains in piping systems to the criteria stated in Tables 4-8 and 5A-6 for those systems essential to safe shutdown of the plant following a LOCA.

The Containment penetration assemblies are designed to accommodate the forces and moments due to pipe rupture. Guides, pipe stops, increased pipe thicknesses or other means are provided to make the penetration the strongest part of the system.

The mathematical models employed in dynamic (seismic) analysis of the Reactor Coolant System components were formulated using lumped parameter modeling techniques. A single composite model was employed in the analysis of the couple components, which included the reactor vessel assembly, the two steam generators (SGs), the four reactor coolant pumps and the reactor coolant piping. The total mass and related stiffness of each of the coupled components was included in the model. Sufficient mass points were included in the model and, at each mass point, translational dynamic degrees of freedom retained, so dynamic analysis includes the combined vertical, torsional and horizontal response of the system due to seismic excitations.

A separate multi-mass model was employed in the seismic analysis of the pressurizer.

5A.3.2.1 Piping

For the design of Seismic Category I piping and equipment, coefficients were based on the floor response-spectrum curves. These curves were generated using the time-history technique for both horizontal and vertical direction, for various damping values, and at designated floor elevations in the Category I structures. This method is based on a dynamic analysis of multi-degree-of-freedom system. Code Case N-411 of the ASME Boiler and Pressure Vessel (B&PV) Code may be used to take advantage of the flexibility in piping systems (Section 5A.3.2.2).

Buried Pipes

All Category I buried pipes are designed for bending stresses due to ground motion. At the joints, where direction of pipe changes, a cushion of compressible material is provided to accommodate any rotation of the pipe joint.

Above-Ground Pipes

Piping systems are anchored and restrained to floors and walls of buildings. The relative seismic displacements between buildings, between floors in buildings, and between major components are applied to the piping, anchors and restraints. When appropriate, seismic movements are considered to be out of phase between structures and/or major components, thereby evaluating the piping systems for the maximum hypothetical relative displacements. The resulting stresses are classified as secondary and are combined with other secondary stresses. The sum of secondary stresses is held within the limits of the applicable piping code.

Local stress analysis of welded attachments using Code Case N-392-1 (12-11-89) is documented in Table 5A-7 as required by Regulatory Guide 1.84.

5A.3.2.2 Routing of Seismic Category I Piping

The routing of Category I piping is typically confined within and/or attached to Category I structures, such as the Containment Structure or the Auxiliary Building. Also, some Category I piping is routed underground between Category I system components, such as the Fuel Oil Storage Tank, and a Category I structure or component. In addition, classification reevaluations and upgrades have occurred. As a result, a few of the upgraded Category I piping segments, such as the Saltwater Ram's Heads, are located in Category II structures, such as the Turbine Building. In each case the upgraded components and the relocations have been evaluated and found to be acceptable (as-is or with already completed modifications) to perform their required safety functions.

Category II piping such as instrument and plant air, plant heating system water, nitrogen, wash water service, fire protection, and roof drain lines are primarily 2" and smaller piping. The 2" and smaller Category II pipe runs which are routed in close proximity of Category I piping do not have the potential to inflict damage on the Category I piping. Physical separation of larger Category II piping is routed such that its failure would not pose a hazard. Where larger Category II piping whose rupture could pose a hazard is routed near Category I piping, adequate pipe restraints are provided to preclude the possibility of pipe whip damage to the Category I piping.

Category I piping was designed in accordance with B31.1 1967, Power Piping, or B31.7 1969, Nuclear Power Piping. Exceptions are noted in relevant sections of the UFSAR for specific systems and components. Effective August 6, 1985, ASME Code Case N-411, "Alternative Damping Values for Seismic Analysis of Piping, Section III Div. 1 Class 1, 2 and 3 Construction," may be used for new analyses or for reconciliation work on new or existing systems (Reference 2). This case takes advantage of piping system flexibility. See the provisions in the NRC letter dated August 6, 1985, when using this code case. All Category I piping, with the exception of 2" and smaller B31.1 and B31.7 Nuclear Class 2 and 3 piping, was originally designed by Bechtel Power Corporation and included the location of restraints and supports, and determination of loads. The building structure connections were checked by the structural engineering group. The piping support contractor was given all necessary information to design and locate pipe supports, and indicates the location of the supports on Bechtel's piping fabrication isometric erection sketch. These drawings, as well as the support design drawings and field installation were checked by Bechtel Engineering. For 2" and smaller Category I piping, a Bechtel field installation manual was provided so that field engineers could properly design and locate pipe supports and restraints. When Bechtel field engineers had completed their design, drawings were submitted to Bechtel engineering for review. The field did not locate any of the seismic supports or restraints for Category I system equipment or components. This work was done at the CE and Bechtel engineering offices.

5A.3.2.3 Equipment, Personnel, and Escape Locks

The equipment, personnel and escape locks are Category I equipment and are designed for the following accelerations: (OBE)

Lock	Vertical Acceleration	Horizontal Acceleration
Equipment Lock	0.07 g	0.11 g
Personnel Lock	0.08 g	0.12 g
Escape Lock	0.07 g	0.10 g

The acceleration values are multiplied by the normal operating weight of the lock or parts of the lock to obtain the horizontal and vertical components of the earthquake force. Both components are considered acting simultaneously with normal operating loads without exceeding code allowable, at a temperature of 120°F.

The earthquake forces due to the SSE are obtained by multiplying the accelerations above by 1.90. The locks are designed to withstand the simultaneous action of SSE components and accident loads as stated in Chapter 5, at a temperature of 276°F, without exceeding material yield stress nor loss-of-lock function.

5A.3.2.4 <u>Seismic Adequacy Verification of Equipment</u>

Because of the extent of changes in seismic design requirements from the 1960's to the 1980's the NRC initiated Unresolved Safety Issue (USI) A-46, "Seismic Qualification of Equipment in Operating Plants," to address the concern that a number of older operating nuclear power plants contained equipment which may not have been qualified to meet newer, more rigorous seismic design criteria. As a result of the technical resolution of USI A-46, the NRC concluded that the seismic adequacy of certain equipment in operating nuclear plants must be reviewed against seismic criteria not in use at the time the plant was licensed. As described in the licensing basis, some Calvert Cliffs electrical equipment was designed to the seismic criteria in IEEE-344 1971 (Reference 3). The NRC issued Generic Letter 87-02 (Reference 4), and encouraged licensees to participate in a generic program to resolve the seismic verification issues associated with USI A-46. The Generic Letter provided the following assumptions for review of the seismic adequacy of structures, systems, and components required during and following a SSE:

- The seismic event does not cause a LOCA, main steam line break to high energy line break and does not occur simultaneously with these events.
- Offsite power may be lost during or following a seismic event.
- The plant must be capable of being brought to a safe shutdown condition following an SSE.

The equipment included in the evaluation is generally limited to active mechanical and electrical components and cable trays. Tanks and heat exchangers that are required to achieve and maintain safe shutdown are also included. Masonry walls were addressed in Bulletin 80-11 (Reference 5) and are not included. The seismic adequacy of the AFW system was addressed in Generic Letter 81-14 (Reference 6) and not included here. Calvert Cliffs participated in the USI A-46 program as described by the Generic Letter and is considered an USI A-46 plant.

As a result, the Seismic Qualification Utility Group (SQUG) developed the Generic Implementation Procedure, Revision 2 (GIP-2) (Reference 7) approved by the NRC via Generic Letter 87-02, Supplement 1 (Reference 8). Reference 8 also required that licensees commit to the commitments and implementation guidance in GIP-2, as supplemented by Reference 8. We committed to the GIP-2 guidance in 1992 (Reference 9).

GIP-2 provided an approved approach for verifying the seismic adequacy of mechanical and electrical equipment needed to maintain the plant in a safe shutdown condition following an SSE as identified in 10 CFR Part 100. The GIP-2 sets forth an approach for verifying seismic adequacy of equipment using earthquake experience data supplemented by test results, as necessary. The four major steps outlined in the GIP were to:

1. Identify seismic evaluation personnel,
2. Identify safe shutdown equipment,
3. Perform screening and verification walkdowns, and
4. Identify outliers and resolve them.

As described in Reference 10, CCNPP's response to each of the major steps was found acceptable.

- The seismic evaluation was performed by a multi-discipline project team with appropriate training and experience.
- Safe shutdown is based on the following safety functions: reactor reactivity control, pressure control, inventory control, and decay heat removal. Primary and alternate safe shutdown success paths with their support systems and instrumentation were identified for each of these safety functions. The plant must be capable of being brought to a hot standby (Mode 3) condition and maintained there for 72 hours following an SSE. Hot standby (Mode 3) is considered the safe shutdown condition for CCNPP. The decay heat removal function was clarified to mean the equipment needed to bring the plant to a hot standby condition and maintain it there for 72 hours. The safe shutdown essential equipment list was developed based on a review of operating, abnormal, and emergency procedures. The 72 hour period is sufficient for inspection of equipment and minor repairs, if necessary following an SSE or to provide additional sources of water for decay heat removal, if needed, to extend the time at safe shutdown.
- A safe shutdown equipment list was developed and walkdowns were performed using the SQUG experience database to determine the degree of seismic ruggedness. Outliers were identified and issues were mitigated. Tanks and heat exchangers, cable and conduit raceways, and relays were evaluated using guidance specific to the equipment type.
- The USI A-46 walkdowns identified a total of 266 items as outliers. Modifications and evaluations resolved the outliers. A completion letter was provided to the NRC (Reference 11).

The NRC determined that the GIP-2 approach may be used as an alternate method, in place of existing methods, for verifying the seismic adequacy of mechanical and electrical equipment for which seismic verification is required (Reference 10). The GIP-2 methodology applies to new and replacement equipment, and existing equipment which has been walked down in accordance with the GIP. The GIP-2 methodology applies to all 20 classes of equipment discussed in the GIP, tanks and heat exchangers, and cable and conduit raceways covered by the GIP, except for the following:

1. Auxiliary Feedwater Actuation System (IEEE 344-1975)
2. Portions of the Engineered Safety Features Actuation System, specifically the maintenance bypass switches and module, isolation module fault indication and auctioneered 15V DC power supplies for the logic modules (see FCR 87-0087 for specific equipment) (IEEE 344-1975)
3. Regulatory Guide 1.97 Category I (PAM 1) Instrumentation (IEEE 344-1975)

4. All safety-related items/equipment for EDG No. 1A and its associated building. (IEEE 344-1987)
5. Wide Range Noble Gas Monitor (IEEE 344-1975)
6. MPT equipment (IEEE 344-1975)
7. Equipment and instrumentation designed to a specific seismic standard later than IEEE 344-1971 (Reference 3)

In the case of tanks and heat exchangers, the GIP-2 methodology may be used only for existing tanks and heat exchangers, not for new installations.

5A.3.3 CATEGORY II

5A.3.3.1 Structure Design

Category II structures are designed in accordance with design methods of accepted codes and standards insofar as they are applicable. Wind design (25 psf zone) is in accordance with the Uniform Building Code, with a one-third increase in the allowable stresses. Seismic design is in accordance with the Uniform Building Code. Seismic forces are based on Seismic Probability Zone 3 multiplied by a ratio of 0.08/0.30. A one-third increase in allowable stresses is not allowed.

5A.3.3.2 Systems and Equipment Design

Category II systems and equipment are designed in accordance with design methods of accepted codes and standards. Wind loads (25 psf zone) and seismic loads, where applicable, conform to the requirements of the Uniform Building Code as stated in Section 5A.3.3.1.

5A.3.4 COUPLED CONTAINMENT, CONTAINMENT INTERNAL STRUCTURE, AND REACTOR COOLANT SYSTEM (RCS) SEISMIC ANALYSIS

Seismic response spectra were developed for the Containment Building shell, Containment Building internal structure, and RCS attachment nozzles and connection points using a coupled Containment Building, containment internal structure/RCS model.

Seismic analyses were performed using "state-of-the-art" methodologies, which included the following:

a. Seismic ground motion based on Regulatory Guide 1.60, Revision 1 recommendations.
b. Structural damping based on Regulatory Guide 1.61, Revision 0 recommendations.
c. Guidance as provided in NUREG-0800 (Standard Review Plan) Sections 3.7.1 and 3.7.2 and NUREG/CR-5347.
d. Development of floor response spectra based on Regulatory Guide 1.84, Revision 31, Regulatory Guide 1.122, Revision 1, and ASCE 4-86 guidance.
e. Three-dimensional (3-D) representation of the structures and the RCS.
f. Soil structure interaction analysis.

Because of the axial symmetry of the containment shell, the existing two-dimensional (2-D) model is the same as the 3-D model. Thus, there is no difference between analysis results obtained during the 2-D model versus the 3-D model.

Although the containment shell is considered symmetrical, the interior structure of the Containment exhibits some degree of asymmetry, which affects the overall dynamic properties, resulting in added torsional response. A revised 3-D model was generated for the containment internal structure.

Response spectra were generated at 14 different locations, including the reactor vessel nozzle, SG supports, containment shell, basemat, 45-foot and 69-foot floor elevations.

The response spectra are generated for two earthquake levels, OBE and SSE. Operating basis earthquake response spectra are generated for constant damping values of 1%, 2%, 3%, 4%, 5%, and for the variable damping of Code Case N-411. Safe shutdown earthquake response spectra are generated for constant damping values of 2%, 3%, 4%, 5%, 7%, and for the variable damping of Code Case N-411.

5A.3.4.1 Methodology

Seismic Ground Motion:

The basic seismic input to the building structure was the Regulatory Guide 1.60 ground motion spectra in two horizontal and one vertical direction. The horizontal spectra were normalized to 0.15g for SSE and 0.08g for OBE in both horizontal directions, and to 2/3 of the horizontal acceleration in the corresponding vertical direction as specified in Section 2.6. Uncorrelated acceleration time history functions based on enveloping the 2%, 5%, and 7% shaped Regulatory Guide 1.60 spectra were generated for each of the three directions.

Structural Damping:

The damping values used in the coupled Containment Building (including internal structure)/RCS analysis were in compliance with Regulatory Guide 1.61 and are as follows:

	OBE	SSE
Containment Structure – shell	2%	5%
Containment Structure - internal structures	4%	7%
RCS	2%	3%

Development of 3-D Model:

For this coupled Containment Building, containment internal structure/RCS seismic analysis, the original 2-D model of the Containment Building was revised to create a 3-D stick model. The stiffness and mass properties of the new 3-D model were developed based on information contained in design basis calculations and drawings. The shear center and mass center for each floor elevation were based on design drawings showing the structural details of the walls and floors of the internal structure of the Containment Building. This model includes the 3-D representation of the RCS attached at the appropriate elevations.

The new 3-D models are described as follows:

1. Containment shell stick model – Since the containment shell is axisymmetric, the 3-D containment shell model is the same as the original 2-D model.
2. Containment internal structure stick model – This dynamic model of the internal structures is a multi-branch 3-D stick model. The first stick represents the primary shield walls only. The second stick represents the secondary shield walls only without the SG pedestals. The secondary shield walls stick splits into three branches above Elevation 69'-0". Two branches correspond to the SG boxes and the third branch corresponds to the pressurizer box. The third stick includes two branches modeling the SG pedestals. This 3-D model is based on the actual wall and floor stiffness and masses, allowing for differences between centers of mass

and shear centers at each major floor elevation. This modeling captures the torsional response of the containment internal structure.

3. RCS stick model – This is a 3-D stick representation of the RCS incorporating SG dynamic properties. A composite 3-D lumped-mass ANSYS model of the reactor vessel, two SGs, four reactor coolant pumps, and main coolant loop piping is included. In addition, representations of the reactor vessel and SG assemblies used in this model include sufficient detail of the reactor internals and replacement SGs internals to account for possible dynamic interaction between those internal components and the RCS. The RCS stick model is coupled to the reactor building internal structure stick model at appropriate support or restrained elevations. The number of masses and dynamic degrees of freedom are consistent with NRC guidance.

Seismic Analysis:

The seismic analysis is performed using the program SUPER SASSI/PC. The SASSI code uses the substructuring method for analysis, including soil structure interaction.

The following analyses were performed:

1. A fixed-base frequency analysis to determine the fundamental frequencies and associated mode shapes of the 3-D containment stick model, including the RCS model, without the effects of the supporting soil. The frequency results were used as a basis for initial specification of frequencies to perform the soil structure interaction analysis.
2. Time-history analyses, including soil structure interaction, for both OBE and SSE using the Regulatory Guide 1.60 as control motion and Regulatory Guide 1.61 damping values for the structures. The model of the Containment Building was subjected to excitation in the three orthogonal directions of the N–S, E-W, and the vertical direction applied separately. Soil properties for input to SUPER SASSI were determined using the computer program SHAKE91.

From the time-history analyses, time histories of response are generated at the points of interest for the containment structure and containment internal structure. Time histories for these points for each of the three earthquake directions are added algebraically at each time point. These in-structure time history results are then used to generate in-structure response spectra with the computer program SPECTRA, at all points that floor response spectra were originally provided, plus the upper feedwater nozzles of the SGs, for the two earthquake levels of OBE and SSE. The resulting spectra for the three soil cases (best estimate, lower bound, and upper bound properties) were enveloped and then peak broadened. In accordance with Regulatory Guide 1.122 and ASCE 4-86, a broadening factor of ± 15% was used to account for the effects of uncertainties. Additionally, prior to broadening, the peaks were reduced 15%, as directed by ASCE 4-86, in order to prevent the introduction of considerable conservatism within the broadened peak region.

At the base node for the primary shield wall, for each earthquake level of OBE and SSE, three translational and three rotational time histories of acceleration response corresponding to the three orthogonal directions are generated. Three sets of time histories are developed corresponding to the three soil cases (i.e., best estimate, lower bound, and upper bound). These time histories are then used in the Westinghouse detailed RCS model.

5A.3.4.2 Results

The 3-D Containment Building, containment internal structure/RCS seismic analysis generated three sets of time histories at the base node that represent the three soil cases. These time histories are utilized for analyses of the RCS. Also generated were numerous floor response spectra. The peaks of these response spectra are 50-60% lower than the response spectra from the original 2-D containment model. However, some of the areas on either side of the peaks contain a somewhat higher response. The damping values to be utilized for the new time histories and floor response spectra are shown in Table 5A-9.

5A.3.5 REFERENCES

1. T.C. Waters and N.T. Barret, "Prestressed Concrete Pressure Vessels for Nuclear Reactors," J. Brit. Nuclear Society 2, 1963
2. Letter from H. R. Denton (NRC) to A. E. Lundvall, Jr. (BGE), dated August 6, 1985, Use of ASME Code Case N-411
3. IEEE 344 – 1971, Trial Use - Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations
4. Generic Letter 87-02, dated February 19, 1987, Verification of Seismic Adequacy of Mechanical and Electrical Equipment in Operating Reactors, Unresolved Safety Issue A-46
5. Bulletin 80-11, dated July 3, 1980, Masonry Wall Design
6. Generic Letter 81-14, dated February 10, 1981, Seismic Qualification of Auxiliary Feedwater Systems
7. Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Power Plant Equipment, Revision 2, corrected February 14, 1992, Seismic Qualification Utility Group
8. Generic Letter 87-02, Supplement 1, dated May 22, 1992, Supplemental Safety Evaluation Report No. 2 on Seismic Qualification Utility Group's Generic Implementation Procedure, Revision 2, corrected February 14, 1992
9. Letter from G. C. Creel (BGE) to Document Control Desk (NRC), dated September 18, 1992, Response to Generic Letter 87-02, Supplement 1 on Seismic Qualification Utility Group (SQUG) Resolution of USI A-46
10. Letter from A. W. Dromerick (NRC) to C. H. Cruse (BGE), dated January 20, 1999, Plant-Specific Safety Evaluation Report for USI A-46 Program Implementation at CCNPP
11. Letter from C. H. Cruse (BGE) to Document Control Desk (NRC), dated February 28, 2001, Completion of USI A-46 Program Implementation at CCNPP

TABLE 5A-1

CONTAINMENT SEISMIC ANALYSIS MODEL POINTS

POINT	DESCRIPTION	ELEVATION OF POINT
1	East SG Main Steam Nozzle	EL 87'6"
2	East SG to Snubbers Attachment	EL 75'0"
3	East SG	EL 55'0"
4	East SG Cold Nozzle	EL 37'6"
5	East SG Snubbers to Concrete Attachment	EL 75'0"
6	East Portion of Operating Slab	EL 67'6"
7	East SG Bottom Support Anchorage	EL 26'0"
8	Interior Concrete Structures	EL 44'0"
9	Reactor Vessel Nozzles	EL 37'6"
10	Reactor Vessel Supports	EL 28'0"
11	Primary Shield Wall	EL 18'6"
12	Bottom Slab	EL 8'6"
13	West SG Snubbers to Concrete Attachment	EL 75'0"
14	West Portion of the Operating Slab	EL 67'6"
15	West SG Bottom Support Anchorage	EL 26'0"
16	West SG Main Steam Nozzle	EL 87'6"
17	West SG to Snubbers Attachment	EL 75'0"
18	West SG	EL 55'0"
19	West SG Cold Nozzle	EL 37'6"
20	Containment Shell	EL 163'0"
21	Containment Shell	EL 144'6"
22	Containment Shell	EL 127'0"
23	Containment Shell	EL 108'0"
24	Containment Shell	EL 90'0"
25	Containment Shell	EL 71'0"
26	Containment Shell	EL 50'0"
27	Containment Shell	EL 38'0"
28	Containment Shell	EL 26'0"

TABLE 5A-2

CONTAINMENT RESPONSES BY RESPONSE-SPECTRUM TECHNIQUE FOR OBE

GROUND RESPONSE SPECTRA FOR A MAX HORIZONTAL ACCEL. OF 8% GRAVITY
RESULTS OF SPECTRUM RESPONSE TECHNIQUE
SQUARE ROOT OF THE SUM OF THE SQUARES

POINT	INERTIAL FORCE kips	ACCELERATION g	DISPLACEMENT feet
1	0.32025E 02	0.18838E 00	0.42611E-01
2	0.74922E 02	0.16951E 00	0.37214E-01
3	0.74056E 02	0.14521E 00	0.28714E-01
4	0.10632E 03	0.13203E 00	0.21545E-01
5	0.73381E 02	0.15798E 00	0.36830E-01
6	0.50756E 03	0.14677E 00	0.33584E-01
7	0.12000E 03	0.13487E 00	0.17282E-01
8	0.11608E 04	0.12826E 00	0.24061E-01
9	0.31474E 03	0.12941E 00	0.21693E-01
10	0.25159E 03	0.13154E 00	0.18321E-01
11	0.13270E 03	0.13502E 00	0.15379E-01
12	0.41119E 04	0.14175E 00	0.12919E-01
13	0.76053E 02	0.16373E 00	0.37036E-01
14	0.55810E 03	0.15071E 00	0.33738E-01
15	0.11879E 03	0.13350E 00	0.17168E-01
16	0.33569E 02	0.19746E 00	0.42900E-01
17	0.77786E 02	0.17599E 00	0.37441E-01
18	0.75445E 02	0.14793E 00	0.28839E-01
19	0.10657E 03	0.13234E 00	0.21565E-01
20	0.12940E 04	0.14220E 00	0.91545E-01
21	0.12917E 04	0.11566E 00	0.81372E-01
22	0.39734E 03	0.92098E-01	0.71656E-01
23	0.31449E 03	0.71909E-01	0.61140E-01
24	0.27536E 03	0.62963E-01	0.51247E-01
25	0.32372E 03	0.68468E-01	0.40949E-01
26	0.34375E 03	0.88127E-01	0.29918E-01
27	0.29112E 03	0.10262E-00	0.23957E-01
28	0.53101E 03	0.11822E-00	0.18528E-01

TABLE 5A-3

CONTAINMENT RESPONSES BY RESPONSE-SPECTRUM TECHNIQUE FOR SSE

GROUND RESPONSE SPECTRA FOR A MAX HORIZONTAL ACCEL. OF 15% GRAVITY
RESULTS OF SPECTRUM RESPONSE TECHNIQUE
SQUARE ROOT OF THE SUM OF THE SQUARES

POINT	INERTIAL FORCE kips	ACCELERATION g	DISPLACEMENT feet
1	0.47902E 02	0.28178E 00	0.72303E-01
2	0.11243E 03	0.25437E 00	0.63029E-01
3	0.11158E 03	0.21878E 00	0.48381E-01
4	0.16016E 03	0.19888E 00	0.35937E-01
5	0.11044E 03	0.23776E 00	0.62445E-01
6	0.76546E 03	0.22135E 00	0.56879E-01
7	0.18036E 03	0.20270E 00	0.28302E-01
8	0.17517E 04	0.19354E 00	0.40419E-01
9	0.47427E 03	0.19500E 00	0.36239E-01
10	0.37838E 03	0.19784E 00	0.30227E-01
11	0.19927E 03	0.20276E 00	0.24861E-01
12	0.61672E 04	0.21261E 00	0.20134E-01
13	0.11430E 03	0.24607E 00	0.62761E-01
14	0.84083E 03	0.22706E 00	0.57113E-01
15	0.17854E 03	0.20066E 00	0.28125E-01
16	0.50122E 02	0.29484E 00	0.72745E-01
17	0.11656E 03	0.26372E 00	0.63377E-01
18	0.11361E 03	0.22276E 00	0.48572E-01
19	0.16053E 03	0.19934E 00	0.35968E-01
20	0.20993E 04	0.23069E 00	0.15681E 00
21	0.21253E 04	0.19029E 00	0.13943E 00
22	0.66603E 03	0.15438E 00	0.12281E 00
23	0.53725E 03	0.12285E 00	0.10481E 00
24	0.46568E 03	0.10648E 00	0.87843E-01
25	0.51936E 03	0.10985E 00	0.70126E-01
26	0.52673E 03	0.13504E 00	0.51016E-01
27	0.44071E 03	0.15535E 00	0.40561E-01
28	0.79903E 03	0.17789E 00	0.30840E-01

TABLE 5A-4

CONTAINMENT RESPONSES BY TIME HISTORY METHOD FOR OBE

	MAXIMUM TIME HISTORY VALUE							
POINT	RELATIVE DISPLACEMENT feet	TIME second	RELATIVE VELOCITY feet/sec	TIME second	RELATIVE ACCELERATION g	TIME second	ABSOLUTE ACCELERATION g	TIME second
1	0.57680E-01	11.790	0.47323E 00	11.560	-0.27655E 00	4.100	-0.31580E 00	4.100
2	-0.50162E-01	12.340	0.41154E 00	11.560	-0.24317E 00	4.100	-0.28241E 00	4.100
3	-0.38167E-01	12.340	-0.32876E 00	6.050	-0.19179E 00	4.100	-0.23403E 00	4.090
4	0.27781E-01	5.670	-0.28418E 00	6.050	0.15859E 00	6.130	-0.19794E 00	4.080
5	-0.49854E-01	12.330	0.39791E 00	11.560	-0.22220E 00	4.100	-0.26458E 00	4.090
6	-0.45353E-01	12.330	0.35948E 00	11.560	-0.20160E 00	4.090	-0.24403E 00	4.090
7	0.23268E-01	5.670	-0.26505E 00	6.050	0.15375E 00	6.130	-0.19224E 00	4.080
8	-0.31748E-01	12.340	-0.29188E 00	6.050	0.15832E 00	6.130	-0.19701E 00	4.080
9	-0.28062E-01	12.340	-0.28193E 00	6.050	0.15599E 00	6.130	-0.19448E 00	4.080
10	0.24335E-01	5.670	-0.26695E 00	6.050	0.15250E 00	6.130	-0.19083E 00	4.080
11	0.20990E-01	5.670	-0.25350E 00	6.050	0.15020E 00	6.130	-0.18880E 00	4.080
12	0.17374E-01	5.670	0.24315E 00	5.580	0.15282E 00	5.500	-0.19016E 00	4.080
13	-0.50036E-01	12.340	0.40461E 00	11.560	-0.23077E 00	4.100	-0.27223E 00	4.090
14	-0.45491E-01	12.340	0.36433E 00	11.560	-0.20717E 00	4.090	-0.24959E 00	4.090
15	0.23099E-01	5.670	-0.26280E 00	6.050	0.15229E 00	6.130	-0.19071E 00	4.080
16	-0.57940E-01	12.340	0.48288E 00	11.560	-0.28807E 00	4.100	-0.32731E 00	4.100
17	-0.50365E-01	12.340	0.41881E 00	11.560	-0.25184E 00	4.100	-0.29109E 00	4.100
18	-0.38275E-01	12.340	-0.33140E 00	6.050	-0.19602E 00	4.100	-0.23776E 00	4.090
19	0.27618E-01	5.670	-0.28459E 00	6.050	0.15892E 00	6.130	-0.19835E 00	4.080
20	0.13972E-00	11.800	-0.81571E 00	12.020	-0.20422E 00	11.810	0.23952E 00	24.830
21	0.12381E-00	11.800	-0.72925E 00	12.020	-0.17635E 00	11.820	0.21776E 00	1.870
22	0.10856E-00	11.800	-0.64666E 00	12.020	-0.15072E 00	11.830	0.20146E 00	1.870
23	0.91971E-01	11.800	-0.55685E 00	12.020	-0.12578E 00	11.850	0.18373E 00	1.870
24	0.76249E-01	11.800	-0.47181E 00	12.030	-0.10575E 00	11.860	0.16690E 00	1.870
25	0.59671E-01	11.800	-0.38238E 00	12.030	-0.10468E 00	5.690	0.14913E 00	1.870
26	-0.41472E-01	12.330	-0.28361E 00	12.030	-0.12066E 00	5.690	0.15232E 00	25.310
27	-0.32545E-01	12.330	-0.25791E 00	6.050	-0.12941E 00	5.690	-0.16089E 00	4.070
28	0.23989E-01	5.670	-0.25166E 00	6.050	0.13807E 00	6.130	-0.17276E 00	4.070

TABLE 5A-5

ASSESSMENT OF PROBABILITY OF EXPOSURES IN EXCESS OF 10 CFR PART 100 FOR EQUIPMENT LOCATIONS WITHOUT TORNADO-GENERATED MISSILE RESISTANT BARRIERS

NUMBER	DESCRIPTION	LOCATION	PROBABILITY OF EXPOSURE IN EXCESS OF 10 CFR PART 100 GUIDELINES PER YEAR PER UNIT
1	EDG Nos. 1B, 2A, and 2B engine intake air filter and exhaust piping and muffler	Exposed components located on the roof of the Auxiliary Building	< 5E-08
2	AFW turbine exhaust piping	Portion of piping running from the floor of the 27’ in the Turbine Building out through the roof of the Auxiliary Building on the associated Unit.	< 1E-08
3	MSSV and ADV vent stacks	Portion of vent stack from floor of the 69’ Elevation in the main plant exhaust equipment room out through the roof of the Auxiliary Building on the associated Unit.	< 5E-08
4	SRW head tanks and exposed piping	SRW head tanks in the main plant exhaust equipment room (69’ Elevation).	< 1E-08
5	Saltwater pumps and piping	Exposure from watertight doors on north end of the Intake structure.	< 1E-08
6	21 FOST vent	On roof of 21 FOST Building	<5E-09
7	EDG No. 1A exhaust ducts	EDG 1A Building	<5E-09
8	2A EDG power cables	Truck Bay (Rm 419) overhead	≤1E-10
9	4kV and 480V Buses 14 and 24	U1 and U2 45' Swgr Rooms through roll up and personnel access doors	≤5E-08
10	Saltwater Air Compressor air tubing	West wall of U1 and U2 45' Swgr Rooms	≤1E-10
11	No. 2 DFO Header	DFO loading station near 11 FOST	≤1E-09
Aggregate Probability (per Section 5A.3.1.9, acceptable if less than < 1E-06.)			< 5E-07

TABLE 5A-6

TABLE OF LOADING COMBINATIONS AND PRIMARY STRESS LIMITS FOR NUCLEAR CLASS 2 AND 3 PIPING

LOADING COMBINATIONS	**PRIMARY STRESS LIMITS** Vessels[d]	Piping	Supports
1. Design Loading + OBE	$P_M \leq S_M$ $P_B \leq P_L \leq 1.5\ S_M$	$P_M \leq 1.2\ S_h$ $P_B + P_M \leq 1.2\ S_h$	Working Stress
2. Normal Operating Loadings + Safe Shutdown + Earthquake	$P_M \leq S_D$ $P_B \leq 1.5\left[1-\frac{(P_M)^2}{(S_D)^2}\right]S_D$ (b)	$P_M \leq S_D$ $P_B \leq \frac{4}{\pi}S_D Cos\left(\frac{\pi}{2}\bullet\frac{P_M}{S_D}\right)$ (c)	Within Yield
3. Normal Operating Loadings + Pipe Rupture + Safe Shutdown Earthquake	$P_M \leq S_L$ $P_B \leq 1.5\left[1-\frac{(P_M)^2}{(S_L)^2}\right]S_L$ (b)	$P_M \leq S_L$ $P_B \leq \frac{4}{\pi}S_L Cos\left(\frac{\pi}{2}\bullet\frac{P_M}{S_L}\right)$ (a),(c)	Deflection of supports limited to maintain supported equipment within limits shown in columns (1) and (2)

(a) These stress criteria are not applied to the piping run within which a pipe break is considered to have occurred.

(b) For loading combinations 2 and 3, stress limits for vessel, with symbol P_M changed to P_L, should also be used in evaluating the effects of local loads imposed on vessels and/or piping.

(c) The tabulated limits for piping are based on a minimum "shape factor." These limits may be modified to incorporate the shape factor of the particular piping being analyzed.

TABLE 5A-6

TABLE OF LOADING COMBINATIONS AND PRIMARY STRESS LIMITS FOR NUCLEAR CLASS 2 AND 3 PIPING

Legend:

P_M	=	Calculated Primary Membrane Stress
P_B	=	Calculated Primary Bending Stress
P_L	=	Calculated Primary Local Membrane Stress
S_M	=	Tabulated Allowable Stress Limit at Temperature from ASME Boiler and Pressure Vessel Code, Section III or ANSI B31.7
S_Y	=	Tabulated Yield at Temperature, ASME B&PV Code, Section III
S_D	=	Design Stress
	=	S_Y (for ferritic steels)
	=	1.2 S_M (for austenitic steels)
S_L	=	$S_Y + 1/3\ (S_u - S_Y)$
S_u	=	Tensile Strength of Material at Temperature
S_h	=	Tabulated Yield at Temperature, from U.S.A. Standard B31.1

The following typical values are selected to illustrate the conservatism of this approach for establishing stress limits Units are 10^3 lbs/in^2. These values are illustrative and not necessarily those to be used in design.

Material	**S_y[1]**	**S_u**	**S_D**	**S_L**
A-106B	24.5	60.0 [2]	25.4	36.9
SA-533B	41.4	80.0 [2]	41.4	54.3
304 SS	17.0	54.0 [3]	18.35	29.3
316 SS	18.5	58.2 [3]	22.2	31.7

(1) From ASME B&PV Code, Section III, at 650°F.

(2) Minimum value at room temperature which is approximately the same at 650°F for ferritic materials.

(3) Estimated.

TABLE 5A-7

USE OF CODE CASE N-392-1 (12-11-89) – EVALUATION OF THE DESIGN OF HOLLOW CIRCULAR CROSS-SECTION WELDED ATTACHMENTS ON CLASS 2 AND 3 PIPING

PIPING CLASS	DESCRIPTION	DRAWING NO.	LOCATION (SUPPORT)	METHOD OF ATTACHMENT
GC-8-1006	1-RV-200 Discharge Piping	91-098 Sh 0002	R-9	Welded
GC-8-1003	1-RV-200 Discharge Piping	91-098 Sh 0002	A-1	Welded
GC-8-1006	1-RV-200 Discharge Piping	91-098 Sh 0002	H-2	Welded
GC-8-1006	1-RV-200 Discharge Piping	91-098 Sh 0002	H-1	Welded
GC-8-1013	1-RV-201 Discharge Piping	91-098 Sh 0001	R-11	Welded
GC-8-1013	1-RV-201 Discharge Piping	91-098 Sh 0001	R-15	Welded
GC-8-1013	1-RV-201 Discharge Piping	91-098 Sh 0001	R-16	Welded
GC-8-1013	1-RV-201 Discharge Piping	91-098 Sh 0001	A-2	Welded

NOTE: Per Regulatory Guide 1.84, Revision 31, May 1999, Design and Fabrication Code Case Acceptability, ASME Section III, Division 1, Code Case N-392-1 is acceptable subject to the following conditions in addition to those conditions specified in the Code Case: Applicants should identify in their Safety Analysis Report: (1) the method of lug attachment, (2) the piping system involved, and (3) the location in the system where the Case is to be applied. Accordingly, this table should be updated as necessary to use the Code Case.

TABLE 5A-8

DAMPING VALUES FOR ALL[1] SEISMIC CATEGORY I STRUCTURES, SYSTEMS, AND COMPONENTS

(Percent of Critical Damping)

STRUCTURE OR COMPONENT	OBE	SSE
Welded steel plate assemblies	1	1
Welded steel framed structures	2	2
Bolted or riveted steel framed structures	2.5	2.5
Reinforced concrete equipment supports	2	3
Reinforced concrete frame and buildings	3	5
Prestressed concrete structures	2	5
Steel piping[2]	0.5	0.5
Soil	2	3
Rocking motion for prestressed concrete structures	5	7
Rocking motion for reinforced concrete structures	5	7

(1) See Table 5-9 for Damping Values of the 1A Diesel Generator Structures, Systems, and Components.

(2) In lieu of these values, ASME Code Case N-411 may be used as described in Section 5A.3.1.6.

TABLE 5A-9

DAMPING VALUES FOR CONTAINMENT SEISMIC CATEGORY I STRUCTURES, SYSTEMS, AND COMPONENTS (FOR USE WITH 3-D CONTAINMENT MODEL GENERATED TIME HISTORIES AND RESPONSE SPECTRA)

(Percent of Critical Damping)[1]

STRUCTURE OR COMPONENT	OBE[2]	SSE
Equipment and large-diameter piping systems[3][4], pipe diameter > 12”	2	3
Small-diameter piping systems[4], pipe diameter ≤ 12”	1	2
Welded steel structures	2	4
Bolted steel structures	4	7
Prestressed concrete structures	2	5
Reinforced concrete structures	4	7

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(1) Damping values higher than the ones delineated above within the Table may be used in a dynamic seismic analysis if documented test data exists to support the use of higher values.

(2) In the dynamic analysis of active components, as defined in Regulatory Guide 1.48, these values should also be used for SSE.

(3) Includes both material and structural damping. If the piping system consists of only one or two spans with little structural damping, use values for small-diameter piping.

(4) In lieu of these values, ASME Code Case N-411 may be used as described in Section 5A.3.1.6.