3.6 PROTECTION AGAINST DYNAMIC EFFECTS ASSOCIATED WITH THE POSTULATED RUPTURE OF PIPING

Two inputs to Section 3.6 are provided. Section 3.6A is applicable to the Stone & Webster Engineering Corporation (SWEC) scope of supply. Section 3.6B is applicable to the GE scope of supply.

With regard to design for protection against dynamic effects associated with the postulated rupture of piping, the respective GE and SWEC responsibilities are as follows:

1. GE's responsibility includes the reactor recirculation piping only. For the recirculation piping, GE determines the postulated break locations and the blowdown reactions resulting from each postulated break, and provides the restraints to restrict pipe whip in the event that a postulated break occurs.

2. SWEC's responsibility includes the balance of piping inside and outside containment. For all piping, except recirculation piping, SWEC determines the break locations and the resulting blowdown reactions, and provides the required pipe whip restraints. In addition, for all piping including the recirculation piping, SWEC analyzes the jet impingement effects resulting from each postulated break.

This section describes the design for protection against postulated piping failures both inside and outside containment including all high- and moderate-energy piping systems. This section includes or references plant layout drawings and system piping and arrangement drawings, and a description of how the plant structures, systems, and components conform to related design criteria and bases. It also demonstrates the ability to perform a safe shutdown after a postulated piping failure of a high- or moderate-energy system.

This section is consistent with pipe rupture criteria submitted to the NRC by NMPC on July 31, 1978[1]. An excerpt from this letter showing a comparison of the Unit 2 criteria to RG 1.46 and Branch Technical Positions (BTP) MEB 3-1 and APCSB 3-1 is provided in Table 3.6A-1.

3.6A PROTECTION AGAINST EFFECTS ASSOCIATED WITH THE POSTULATED RUPTURE OF PIPING (SWEC SCOPE OF SUPPLY)

3.6A.1 Postulated Piping Failures in Fluid Systems Inside and Outside the Containment

3.6A.1.1 Design Bases

Criteria

The pipe failure protection conforms to Appendix A of 10CFR50, GDC 4, Environmental and Missile Design Bases. The overall design for this protection is in compliance with BTP APCSB 3-1 and MEB 3-1. Compliance to RG 1.46 is discussed in Table 1.8-1.

Design Objectives

Protection against pipe failure effects is provided to fulfill the following objectives:

1. To ensure that the reactor can be shut down safely and maintained in a safe shutdown condition.

2. To ensure that radiological doses resulting from a postulated piping failure remain below the limits of 10CFR100.

3. To ensure that containment integrity is maintained.

4. To ensure that a pipe break that is not a loss of reactor coolant does not cause a loss of reactor coolant.

5. To ensure that a postulated piping failure with its direct consequences and a single active component failure do not result in unacceptable consequences except as noted below.

   Where the postulated piping failure is assumed to occur in one of two or more redundant trains of a dual-purpose, moderate-energy, essential system (i.e., one required to operate during normal plant conditions as well as to shut down the reactor and mitigate the consequences of the piping failure), single failures of components in the other train or trains of that system are not assumed since:

   a. The system is designed to Category I standards.

   b. Power is provided from both offsite and onsite sources.

   c. Construction, operation, and inspection are done in accordance with quality assurance, testing, and in-service requirements appropriate for nuclear safety systems.

   Examples of systems that qualify as moderate-energy, dual-purpose, essential systems are the SWP and RHR systems.

6. To ensure that minimum core cooling requirements are maintained for pipe break events, their direct consequences, and any single active failure as

specified in Section 3.6A.1.1, the following break areas and break combinations are not exceeded:

a. For breaks involving recirculation piping, the total effective area of all broken pipes (including the effective area of the recirculation line break) does not exceed the total effective area of the design basis double-ended recirculation line break (DBA).

b. Breaks involving one recirculation loop do not result in a loss of coolant from the other recirculation loop in excess of that which would result from a break of the attached cleanup connections on the suction side of the loops.

Design Assumptions

The following assumptions are used to determine the protection requirements:

1. Pipe breaks or cracks are postulated to occur during normal plant operation (i.e., reactor startup, operation at power, hot standby, or reactor cooldown to a cold shutdown).

2. Only high-energy piping, as defined in Section 3.6A.2.1.1, is capable of producing breaks. Moderate-energy piping, as defined in Section 3.6A.2.1.2, is capable of producing only cracks.

3. Each longitudinal or circumferential break in high-energy fluid system piping, or leakage crack in moderate-energy fluid system piping, is considered separately as a single postulated initial event occurring during normal plant conditions.

4. Pipe failures (breaks or cracks) inside the containment are not postulated concurrently with pipe failures outside the containment.

5. Offsite power is assumed to be unavailable when a trip of the turbine generator system or RPS is a direct consequence of the postulated piping failure, unless it is more conservative to assume that offsite power is available (e.g., a feedwater line break with offsite power available leads to a larger inventory of water for flooding considerations).

6. All available systems, including those initiated by Operator actions, are employed to mitigate the consequences of a postulated piping failure.

7. A whipping pipe is not considered capable of rupturing impacted pipes of equal or greater nominal pipe diameter and equal or greater thickness.

8. Pipe whip is assumed to occur in the plane defined by the piping geometry, and to cause movement in the direction of the jet reaction, unless shown to be otherwise by analysis.

9. The fluid internal energy associated with the pipe break reaction takes into account any line restrictions (e.g., flow limiter) between the pressure source and break location and absence of energy reservoirs, as applicable.

10. Damage to the RPV from the surface impact effects of pipe rupture does not occur due to its location relative to piping systems.

11. Initial pipe break events are not assumed to occur in pump and valve bodies because of their greater wall thicknesses.

Approach

Systems, components, and equipment required to safely shut down the plant and mitigate the consequences of postulated piping failures (hereinafter called essential systems, components, and equipment) are reviewed to determine their susceptibility to the pipe failure.

Pipe breaks are evaluated for the effects of pipe whip, jet impingement, flooding, room pressurization, and other environmental effects such as temperature.

Pipe cracks are evaluated for wetting from spray, flooding, and other environmental effects.

Piping system break and crack locations are determined in accordance with Section 3.6A.2. A flow chart of activities, sample model, and typical restraint design are shown on Figures 3.6A-1 through 3.6A-11.

3.6A.1.2 Description of Piping Failures

A list of essential systems, components, and equipment, or portions thereof, is provided in Table 3.6A-72. A list of high-energy lines, as discussed in Section 3.6A.2.1.1, is given in Table 3.6A-73. Moderate-energy piping (Section 3.6A.2.2) is not listed.

Composite drawings (Figures 3.6A-52 through 3.6A-60) show the routing of high-energy piping in relation to compartments. Nearby essential items are discussed in Section 3.6A.2.5.

Pressure response analyses are performed for the subcompartments containing high-energy piping. For a detailed discussion of the line breaks selected, vent paths, room volumes, analytical methods, pressure results, etc., refer to Section 6.2.1.2 for containment subcompartments, and Appendix 3B for subcompartments located outside the containment.

The effects of pipe whip, jet impingement, spraying, and flooding on essential systems, components, and equipment are discussed in Appendix 3C.

There are no high-energy lines located near or in the main control room. Therefore, effects upon the habitability of the main control room from pipe break, including pipe whip, jet impingement, and transport of steam, are not considered. Further discussion of main control room habitability systems is provided in Section 6.4.

There are no high-energy lines located near or in the diesel generator building.

3.6A.1.3 Safety Evaluation

3.6A.1.3.1 Approach

An analysis of pipe failures is performed to identify those safety-related systems, components, and equipment that provide protective actions required to mitigate the consequences of the postulated pipe failure.

Design features such as separation, barriers, and pipe whip restraints are incorporated to ensure that pipe breaks and cracks do not damage essential items to an extent that would impair the integrity or operability of essential systems and components.

Specific design features used for protecting the essential systems, components, and equipment, and the ability of specific safety-related systems to withstand a single active failure concurrent with a postulated event, are discussed in Appendix 3C.

When the pipe layout and plant arrangement drawings show that the effects of postulated breaks/cracks are isolated, physically remote, or restrained by plant design features from essential systems or components, no further evaluation is performed.

3.6A.1.3.2 Protection Methods

The effects associated with a particular break/crack must be mechanistically consistent with the failure. Thus, actual pipe dimensions, piping layouts, material properties, and equipment arrangements are considered in defining the specific measures for protection against actual pipe movement and other associated consequences of postulated failures. Protection against the

dynamic effects of pipe failures is provided in the form of pipe whip restraints, jet impingement shields, barriers, compartments, and physical separation of piping, equipment, and instrumentation. Pipe supports are used as protective measures in isolated cases. The specific method chosen depends on physical limitations such as accessibility, maintenance, and proximity to other essential systems, components, and equipment. Protective measures utilized to meet these requirements consider access requirements for conducting the in-service examinations specified in the ASME Boiler and Pressure Vessel Code, Section XI.

Separation and Enclosure

Separation is achieved to the extent practicable by plant physical layouts that provide sufficient distances so that essential systems and components are separated from other fluid systems. Fluid systems that are not physically separated from essential systems and components are enclosed, when practical, within structures or compartments designed to protect nearby essential systems and components. Alternatively, essential systems and components may be enclosed within structures or compartments designed to withstand the effects of postulated piping failures in nearby fluid systems.

Barriers and Shields

In many cases protection requirements are met by walls, floors, columns, abutments, and foundations. Where adequate protection does not exist due to separation, additional barriers, deflectors, or shields are provided as necessary.

Piping Restraint Protection

Pipe restraints for protection against pipe whip as a result of high-energy pipe breaks are provided except in cases described as follows:

1. The piping is either physically separated (or isolated) from any essential safety-related structure, system, or component required to place the plant in a safe shutdown condition, or the piping is restrained from whipping by plant design features such as concrete encasement or other structures or compartments designed for jet impingement and pipe whip loads. These loads are applied either at the most critical locations of the protective structures or at locations determined in Appendix 3C.3.1.

2. Following a single break, unrestrained pipe movement of either end of the ruptured pipe could not damage, to an unacceptable level, any structure, system, or component required to place the plant in a safe shutdown condition.

3. The energy associated with the whipping pipe is demonstrated to be insufficient to impair, to an unacceptable level, the safety function of any structure, system, or component required to place the plant in a safe shutdown condition.

The design criteria for restraints are given in Section 3.6A.2.3.2.

3.6A.1.3.3 Specific Protection Measures

Nonessential systems and system components are not required for the safe shutdown of the reactor, nor are they required for the limitation of the offsite release in the event of a pipe rupture. However, while none of this equipment is needed during or following a pipe break event, pipe whip protection is provided in specific cases where the broken pipe in the nonessential system could adversely impact an essential system or component.

The pressure, water level, and flow sensor instrumentation for those essential systems required to function during or after accident conditions are protected from pipe rupture effects.

High-energy fluid system piping restraints and protective measures are designed in such a way that a postulated break in one pipe could not, in turn, lead to rupture of other nearby pipes or components if the secondary rupture could result in consequences that would be considered unacceptable for the initial postulated break.

Pipe rupture restraints are located such that the unrestrained portion will not form a plastic hinge at the restraint. This criterion precludes tip deflection from adversely affecting safety-related equipment.

A postulated rupture in the piping in Unit 1 does not affect the capability for safe shutdown of Unit 2 and vice versa. There are no shared safety systems between Unit 1 and Unit 2.

For any postulated pipe rupture, the structural and leak-tight integrity of the containment structure is maintained.

To maintain the ability to scram the reactor in the event of a pipe rupture, the CRD withdraw lines are protected from pipe break events so that no more than one in any rod array is allowed to be completely crimped (totally blocked). Complete severance of withdraw lines will not affect the scram function. Protection for the CRD insert lines is not required during normal reactor operation since a reactor pressure of 450 psig or higher (CRD insert lines principal backup) could adequately scram the control rods even with a complete loss of insert lines. Routing of high-energy lines in the vicinity of the CRD withdraw lines is strictly controlled by design measures.

The escape of steam, water, combustible, or corrosive fluids, gases, and heat in the event of a pipe rupture does not preclude:

1. Habitability of the main control room.

2. The ability of essential instrumentation, electric power supplies, components, and controls to perform their safety function.

The potential for damage to both independent high-pressure ECCS systems (ADS and HPCS) in the event of a partial break in the pressurized portion of the HPCS has been considered. No portion of the normally-pressurized HPCS system is located within jet impingement distance of any component considered essential to the operation of the ADS, and vice versa.

3.6A.2 Determination of Break Locations and Dynamic Effects Associated with the Postulated Rupture of Piping

3.6A.2.1 Criteria Used to Define Break and Crack Location and Configuration

3.6A.2.1.1 Definition of High-Energy Fluid System

High-energy fluid systems are defined as those systems or portions of systems that during normal plant conditions are either in operation or are maintained pressurized under conditions where either or both of the following are met:

Maximum temperature exceeds 200°F, or

Maximum pressure exceeds 275 psig.

Normal plant conditions are defined as the plant operating conditions during reactor startup, power plant operation, and reactor cold shutdown, but excluding test modes.

3.6A.2.1.2 Definition of Moderate-Energy Fluid System

Moderate-energy fluid systems are defined as those systems or portions of systems that during normal plant conditions are either in operation or are maintained pressurized under conditions where both of the following are met:

Maximum temperature is 200°F or less, and

Maximum pressure is 275 psig or less.

Piping systems are classified as moderate-energy systems when they operate as high-energy piping for only short operational periods in performing their system functions, but for the major operational period qualify as moderate-energy fluid systems. An

operational period is considered "short" if the total fraction of time that the system operates within the pressure-temperature (P-T) conditions specified for the high-energy fluid system is less than 2 percent of the total operating time for which the system is designed.

3.6A.2.1.3 Postulated Pipe Breaks and Cracks

A postulated pipe break is defined as a sudden, gross failure of the pressure boundary either in the form of a complete circumferential severance (guillotine break) or as the development of a sudden longitudinal, uncontrolled crack (longitudinal split), and is postulated for the high-energy fluid system only. For moderate-energy fluid systems, pipe breaks are confined to the postulation of controlled cracks in piping and branch runs. These cracks affect the surrounding environmental conditions only, and do not result in whipping of the cracked pipe.

Portions of piping systems that are isolated from the source of the high-energy fluid during normal plant conditions are exempted from consideration of postulated pipe breaks. This includes portions of piping systems beyond a normally closed valve. Pump and valve bodies are also exempted from consideration of pipe break because of their greater wall thickness. Internal missiles that might be generated from failures of these components are evaluated as discussed in Section 3.5.1.

A high-energy piping system break is not postulated simultaneously with a moderate energy piping system crack.

Effects of moderate-energy leakage cracks inside the containment are bounded by DBA (large- and small-break LOCA) considerations. Environmental effects, including effects from spraying and flooding as well as from jet impingement and pipe whip, are less severe for moderate-energy pipe cracks than for LOCA events. Therefore, safety-related equipment which is qualified to function post-LOCA is also available to mitigate the consequences of moderate-energy line cracks.

3.6A.2.1.4 Exemptions from Pipe Whip Protection Requirements

Protection from pipe whip need not be provided if any one of the following exists:

1. Piping is classified as moderate-energy piping.

2. Following a single postulated pipe break, the piping for which the unrestrained movement of either end of the ruptured pipe in any feasible direction cannot impact any structure, system, or component important to safety.

3. Piping for which the internal energy level associated with whipping is insufficient to impair the safety function of any structure, system, or component to an unacceptable level. Any line restrictions (e.g., flow limiters) between the pressure source and break location, and the effects of either a single-ended or double-ended flow condition are accounted for in the determination of the internal fluid energy level associated with the postulated pipe break reaction. The energy level in a whipping pipe is considered insufficient to rupture an impacted pipe of equal or greater nominal pipe size and equal or heavier wall thickness.

3.6A.2.1.5 Postulated Pipe Break Locations

Criteria for Inside the Containment

For ASME Section III, Safety Class 1 piping systems within the containment, design basis piping break locations are selected using the following criteria:

1. At the terminal ends including:

    a. Piping, pressure vessel, or equipment nozzle intersections.

    b. High energy-moderate energy boundary.

    c. A branch connection to a main run unless all the following are met:

        (1) The branch and main runs are of comparable size and fixity (i.e., the nominal size of the branch run is at least one-half that of the main run),

        (2) The intersection is not rigidly constrained by the building structure, and

        (3) The branch and main runs are modeled as a common piping system during the pipe stress analysis.

2. At the intermediate locations between the terminal ends selected by either of the following criteria:

    a. At each fitting (e.g., elbow, tee, cross, flange, and nonstandard fitting), welded attachment, and valve, or

    b. At locations where the maximum stress range for the normal and upset plant conditions and for an OBE exceeds 2.4 S_m, calculated by Equation (10)

and either Equation (12) or (13) in ASME Section III, Paragraph NB-3653; and at locations where the cumulative usage factor (CUF)[1], U, derived from the piping fatigue analysis, under the loadings associated with OBE and operational plant conditions exceeds 0.1. S_m is the allowable stress intensity as specified in ASME Section III, Subparagraph NB-3213.1.

If, as a result of piping reanalysis of the original design configuration, the highest stress or CUF location shifts for any initially determined arbitrary intermediate break location, the original postulated arbitrary break location need not change unless a redesign of the piping resulting in a change in pipe parameters (diameter, wall thickness, routing) is required, or the dynamic effects from the new (as-built) intermediate break locations are not mitigated by the original pipe whip restraints and jet shields.

For ASME Section III, Safety Class 2 and 3 piping systems, break locations are postulated by the following criteria:

1. At the terminal ends.

2. At the intermediate locations between the terminal ends selected by either of the following criteria:

   a. At each pipe fitting (e.g., elbow, tee, cross, flange, and nonstandard fitting), welded attachment, and valve, or

   b. At each location where the stress associated with normal and upset plant conditions and an OBE event calculated by Equations (9) plus (10) in Paragraph

      NC-3652 of ASME Section III, exceeds 0.8 (1.2 S_h+S_A).

      S_h = Allowable stress at the elevated temperature calculated according to ASME Section III, Subarticles NC-3600 and ND-3600 for Safety Class 2 and 3 components, respectively.

      S_A = Allowable stress range for the expansion stress calculated according to ASME Section III, Subarticle NC-3600 and ANSI B31.1.

[1] Specified in ASME Section III, Subparagraph NB-3222.4.

3. If a fatigue analysis is performed at any intermediate locations between the terminal ends where the CUF under the loading associated with OBE and operational plant conditions exceeds 0.1.

<u>Criteria for Outside the Containment</u>

<u>High-Energy Fluid Systems</u> The following criteria are used to define break and crack locations in high-energy fluid systems outside the containment:

1. <u>Fluid Systems Separated from Essential Structures, Systems, and Components</u> Breaks are not postulated in high-energy piping at locations that are isolated or physically remote from essential equipment, structures, and the containment.

2. <u>Fluid System Piping in Containment Penetration Areas</u> Breaks are not postulated in the portions of high-energy piping between the containment isolation valves, outside and inside containment. Breaks are not postulated in the portions of high-energy piping between the isolation valve and the first restraint or groups of restraints designed to protect these portions of piping. Containment isolation valve pipe whip restraints are capable of resisting bending and torsional moments produced by a postulated piping failure outboard of the first restraint or group of restraints beyond the containment isolation valves.

   Restraints are designed to withstand the loadings resulting from a postulated piping failure, so that neither the isolation valve operability nor the leak-tight integrity of the associated containment penetration will be impaired. These portions of piping are designed to meet the requirements of ASME Section III, Subarticle NE-1120, and the following additional design requirements, which are in conformance with Revision 1 (July 1981) of SRP 3.6.2 and BTP MEB 3-1, the documents applicable at the time the analysis was performed:

   a. The following design stress and fatigue limits are not exceeded for Safety Class 1 piping:

      (1) The maximum stress range between any two load sets (including the zero load set), calculated by Equation (10) in Paragraph NB-3653, ASME Section III, for those loads and conditions thereof for which Level A and Level B stress limits have been specified in the design specification, including an OBE event transient, do not exceed 2.4 S_m. If the calculated maximum stress range of

Equation (10) exceeds 2.4 S_m, the stress ranges calculated by both Equations (12) and (13) in Paragraph NB-3653 will meet the limit of 2.4 S_m.

(2) The CUF is less than 0.1.

(3) The maximum stress, as calculated by Equation (9) in Paragraph NB-3652, under the loading resulting from a postulated piping failure beyond these portions of piping does not exceed 2.25 S_m, except that following a failure outside containment, the pipe between the outboard isolation valve and the first restraint may be permitted higher stresses provided a plastic hinge is not formed.

b. The following design stress limits are not exceeded for Safety Class 2 and nonnuclear piping in the break exclusion area:

(1) The maximum stress ranges do not exceed 0.8 (1.2 S_h+S_A), as calculated by Equations (9) and (10) in Paragraph NC-3652, ASME Code Section III, considering normal and upset plant conditions (i.e., sustained loads, occasional loads, and thermal expansion) and an OBE event.

(2) The maximum stresses do not exceed 1.8 S_h, as calculated by Equation (9) in Paragraph NC-3652 under the loadings resulting from a postulated piping failure of fluid system piping beyond these portions of piping.

c. The following design stress limits are not exceeded for Safety Class 3 piping between the outboard isolation valve and the first restraint:

(1) The maximum stress ranges do not exceed 0.8 (1.2 S_h+S_A), as calculated by Equations (9) and (10) in Paragraph NC-3652, ASME Code Section III, considering normal and upset plant conditions (i.e., sustained loads, occasional loads, and thermal expansion) and an OBE event.

(2) Following a pipe failure outside the containment, the formation of a plastic hinge is not permitted in the piping between the outboard isolation valve and the first pipe whip restraint, or group of restraints, to assure the operability of the isolation valve.

d. Welded attachments for pipe supports or other purposes to these portions of piping are avoided, except where detailed stress analysis demonstrates compliance with the limits discussed in Items 2a and 2b.

e. The number of circumferential and longitudinal piping welds and branch connections is minimized.

f. The length of these portions of piping is reduced to the minimum length practicable.

g. The design of pipe anchors or restraints (e.g., connections to containment penetrations and pipe whip restraints) does not require welding directly to the outer surface of the piping (e.g., flued integrally-forged pipe fittings are used), except where such welds are capable of 100-percent volumetric ISI. This criterion is also applicable to the portion of piping between the containment and the inside containment isolation valves.

h. For these portions of high-energy fluid system piping, preservice and subsequent inservice examinations are performed in accordance with the requirements specified in ASME Section XI. During each inspection interval, as defined in IWA-2400, an ISI is performed on all nonexempt ASME Code Section XI circumferential and longitudinal welds within the break exclusion region for high-energy fluid system piping. These inspections consist of augmented volumetric examinations (nominal pipe size greater than or equal to 4 in) and augmented surface examinations (nominal pipe size less than 4 in) such that 100 percent of the previously defined welds are inspected at each interval, or as required per the risk-informed process for piping outlined in EPRI Topical Report TR-1006937 and Nuclear Engineering Report NER-2A-025 NMP2 RI-ISI BER Evaluation. The break exclusion zone consists of those portions of high-energy fluid system piping between the moment limiting restraint(s) outside the outboard containment isolation valve and the moment limiting restraint(s) beyond the inboard containment isolation valve. The choice of the restraint(s) that define the limits of the break exclusion zone is based upon those restraint(s) which are necessary to ensure the operability of the primary containment isolation valves.

i. Regardless of the fact that all conditions above have been met, a crack is postulated in the main

steam or feedwater piping in the main steam tunnel. The crack in the pipe, equal in area to a single-ended pipe rupture, is considered a singular event. Pipe whip and jet impingement are not considered, and a single active failure is not taken as a concurrent event.

3. <u>Balance of Piping Outside the Containment</u>

a. Breaks in ASME Section III, Safety Class 2 and 3 piping and in nonnuclear class piping that is seismically analyzed and supported are postulated at the following locations in each piping and branch run (except those portions of fluid system piping identified in Items 1 and 2):

(1) At terminal ends of the pressurized portions of the runs.

(2) At intermediate locations selected by either of the following criteria:

(a) At each pipe fitting (e.g., elbow, tee, cross, and nonstandard fitting), welded attachment, and valve, or, if the run contains no fittings, at one location at each extreme of the run within the protective structure (a terminal end, if located within a protective structure, may substitute for one intermediate break).

(b) At each location where the stresses associated with normal and upset plant conditions and an OBE event exceed 0.8 (1.2 S_h+S_A), as calculated by Equations (9) and (10), Paragraph NC-3652 of ASME Section III, for Safety Class 2 and 3 piping.

b. Breaks in nonnuclear safety class piping not seismically qualified are postulated at the following locations in each piping or branch run:

(1) At terminal ends of the pressurized portions of the runs.

(2) At each pipe fitting, welded attachment, and valve.

These breaks are sufficient to establish the worst pipe break effects since:

(1) The piping is physically remote from essential equipment and structures, or

(2) There are a large number of pipe breaks postulated on the same line in the same area due to a large number of pipe fittings or attachments on the pipe, or

(3) For nonseismic piping in close proximity to safety-related systems, components, or structures, either the piping is seismically analyzed and supported or other protection is provided.

Moderate-Energy Fluid Systems The following criteria are used to define crack locations in moderate-energy fluid systems outside the containment:

1. For the purpose of satisfying the separation provisions of plant arrangement, a review of the piping layout and plant arrangement drawings is conducted. Safe shutdown systems are isolated or located physically remote from the effects of through-wall leakage cracks, to the extent this is practical.

2. Leakage cracks are not postulated in those portions of piping between the isolation valve and the containment, provided they meet the requirements of ASME Section III, Subsubarticle NE-1120, and are designed so that the maximum stress range associated with normal and upset plant conditions and an OBE event does not exceed 0.4 (1.2 S_h+S_A) (as calculated by Equations (9) and (10), Paragraph NC-3652 of ASME Section III, Safety Class 2 piping).

3. Cracks are not postulated in moderate-energy fluid system piping located in an area in which a break in high-energy piping occurs. Where a postulated leakage crack in the moderate-energy fluid system piping results in more limiting environmental conditions than a break in proximate high-energy fluid system piping, the provisions identified under Item 4 below are applied.

4. Through-wall leakage cracks are postulated in fluid system piping, except where exempted under Items 1, 2, and 3 above, or where the maximum stress range, associated with normal and upset plant conditions and an OBE event, in these portions of ASME Section III, Safety Class 2 or 3 piping and nonnuclear piping is less than 0.4 (1.2 S_h+S_A) (as calculated by Equations (9) and (10), Paragraph NC-3652 of ASME Section III). The cracks are postulated to occur individually at locations that result in the maximum effects from fluid

spray and flooding. Only environmental effects that develop from these cracks are considered.

5. Through-wall leakage cracks, instead of breaks, are postulated in the piping of those fluid systems that qualify as high-energy fluid systems for only short operational periods, but qualify as moderate-energy fluid systems for the major operational period. An operational period is considered short if the fraction of time that the system operates within the P-T conditions specified for high-energy fluid systems is less than 2 percent of the time that the system operates as a moderate-energy fluid system (e.g., systems such as the reactor RHR system qualify as moderate-energy fluid systems).

3.6A.2.1.6 Design Basis Break/Crack Types and Orientation

Circumferential Pipe Breaks

Circumferential breaks are postulated in high-energy fluid system piping at the locations specified in Section 3.6A.2.1.5 except:

1. For nominal pipe size of 1 in or less.

2. Where it is determined by detailed stress analysis that the stress in the circumferential direction is at least 1.5 times that in the axial direction at the location of maximum stress.

Where break locations are selected at pipe fittings without the benefit of stress calculations, breaks are postulated at the piping weld to each fitting, valve, or welded attachment. If detailed stress analyses or tests are performed, the maximum stressed location in the fitting may be selected instead of the pipe-to-fitting weld.

Circumferential breaks are assumed to result in pipe severance and separation amounting to at least one-diameter lateral displacement of the ruptured piping sections unless physically limited by piping restraints, structural members, or piping stiffness as may be demonstrated by inelastic analysis.

The dynamic force of the jet discharge at the break location is based on the effective cross-sectional flow area of the pipe and on a calculated fluid pressure as modified by an analytically or experimentally determined thrust coefficient. Limited pipe displacement at the break location, line restrictions, flow limiters, positive pump-controlled flow, and the absence of energy reservoirs is taken into account, as applicable, in the reduction of jet discharge.

Pipe whipping is assumed to occur in the plane defined by the piping geometry and configuration, and is assumed to cause pipe

movement in the direction of the jet reaction, unless shown to be otherwise by analysis.

Longitudinal Pipe Breaks

Longitudinal breaks are postulated in high-energy fluid system piping at the locations specified under circumferential pipe breaks, except:

1. For nominal pipe sizes smaller than 4 in.

2. Where it is determined by detailed stress analysis that the stress in the axial direction is at least 1.5 times that in the circumferential direction at the location of maximum stress.

3. At terminal ends.

4. At intermediate locations where the criterion for a minimum number of break locations must be satisfied, except where it is determined by detailed stress analysis that the stress in the circumferential direction is at least 1.5 times that in the axial direction.

Longitudinal breaks are assumed to result in an axial split without pipe severance. Splits are located (but not concurrently) at two diametrically opposed points on the piping circumference in such a way that a jet reaction causing out-of-plane bending of the piping configuration results. Alternately, a single split may be assumed at the section of highest stress as determined by detailed stress analysis.

The dynamic force of the fluid jet discharge is based on a circular break area equal to the effective cross-sectional flow area of the pipe at the break location, and on a calculated fluid pressure modified by an analytically or experimentally determined thrust coefficient as determined for a circumferential break at the same location. Line restrictions, flow limiters, positive pump-controlled flow, and the absence of energy reservoirs are taken into account, as applicable, in the reduction of jet discharge.

Pipe movement is assumed to occur in the directions defined by the stiffness of the piping configuration and jet reaction forces, unless limited by structural members or piping restraints.

Through-Wall Leakage Cracks (Outside the Containment Only)

Through-wall leakage cracks are postulated in main steam or feedwater piping systems in containment penetration areas as stated in Section 3.6A.2.1.5 under High-Energy Fluid Systems, Item 2h. The following through-wall leakage cracks are

postulated in Moderate-Energy Fluid System piping at the locations specified in Section 3.6A.2.1.5 under Moderate-Energy Fluid Systems:

1. Cracks are postulated in moderate-energy fluid system piping and branch runs exceeding a nominal pipe size of 1 in.

2. Fluid flow from a crack is based on a circular opening of area equal to that of a rectangle one-half the nominal pipe diameter in length and one-half the pipe wall thickness in width.

3. The flow from the crack is assumed to result in an environment that wets all unprotected components within the compartment, with consequent flooding in the compartment and communicating compartments. Flooding effects are determined on the basis of a conservatively estimated time period required to effect corrective actions.

3.6A.2.2 Analytical Methods to Define Forcing Functions and Response Models

3.6A.2.2.1 Introduction

Pipe rupture analyses consist of calculations to determine the fluid forces generated by the blowdown of pressurized lines, complemented by dynamic or energy balance analyses to determine pipe motion and impact effects (Figure 3.6A-1). Restraints for lines 6 in and less in diameter are usually qualified on a generic basis using an energy balance. However, restraints for larger lines are engineered individually for each system, usually using standard design concepts and worst-case dynamic analysis to qualify several similar restraints in different locations. The response of unrestrained lines is analyzed by either inelastic dynamic analysis or energy balance analysis.

Criteria for the response analyses are as follows:

1. An analysis of the pipe run or branch is performed for each postulated longitudinal and circumferential rupture or, alternatively, for a worst case. Worst cases are selected on the basis of gap, fluid force, and piping system stiffness.

2. The loading condition of a pipe run or branch prior to postulated rupture in terms of internal pressure, temperature, and stress state is that condition associated with reactor operation at 100-percent power.

3. For a circumferential rupture, pipe whip dynamic analyses are only performed for that end (or ends) of the pipe or branch that is (are) connected to a

contained fluid energy reservoir having sufficient capacity to develop a jet stream.

4. Dynamic analytical methods, used for calculating the piping or piping/restraint system response to the jet thrust developed after a postulated rupture, adequately account for the effects of the following:

   a. Mass inertia and stiffness properties of the system.

   b. Impact and rebound (if any) as permitted by gaps between piping and restraint.

   c. Elastic and inelastic deformation of piping and/or restraint.

   d. Support boundary conditions.

5. An allowable design strain limit of 0.5 ultimate uniform strain of the restraints is used for tensile energy-absorbing components. For compressive energy-absorbing components, a design limit of 80 percent of energy-absorbing capacity is used.

6. A 10-percent increase of minimum specified yield strength (S_y) may be used to account for strain rate effects in inelastic nonlinear analyses. Alternatively, experimental data may be used to determine the strain rate parameters for use in nonlinear codes that monitor strain rate.

3.6A.2.2.2 Time-Dependent Blowdown Force

The blowdown force calculations are based on the transient pressures, velocities, and other thermodynamic properties of the fluid[2]. To provide the time history of pressure, velocity, etc., the method of characteristics is used to solve the continuity and momentum equations simultaneously. A general description of the method can be found in most gas dynamics textbooks[3-6]. For these one-dimensional fluid mechanics analyses, the pipe is regarded as straight, despite numerous bends. The calculated momentum and pressure forces are applied wherever there is a change in flow direction or cross section of the piping to provide time-dependent loads for pipe dynamic analysis.

The transient forces result from wave propagation and fluid momentum. It is assumed that pipe bends and elbows neither attenuate the traveling pressure waves nor cause reflections. Immediately following the rupture of a pipe, a decompression wave travels from the break at the speed of sound relative to the fluid. The fluids ahead of and behind the wave are at different states. This initial blowdown condition will last until a return

signal from a pressure reservoir reaches the break. Repeated wave reflections between the reservoir and break prevail until a steady-state flow condition is established. Boundary conditions that govern the flow at the break end and at the inlet from the vessel to the pipe are applied.

The time histories of transient pressure, mass flow rate, and other thermodynamic properties of the fluid are based on the following equation, which includes static and dynamic effects, to calculate the blowdown force:

$$F = \left[P_e - P_a + \frac{RU_e^2}{144g} \right] A \qquad (3.6A\text{-}1)$$

Where:

F = Blowdown force, lb_f

P_e = Pressure at exit plane, psia

P_a = Ambient pressure, psia

U_e = Velocity of fluid at exit plane, fps

R = Density of fluid, lb_m/ft^3

A = Pipe break area, sq in

g_c = Gravitational constant, lb_m-ft/lb_f sec^2

The effects of line friction are included in the evaluation of steady-state blowdown. For the calculation of the transient fluid response, however, friction may or may not be considered.

Subcooled Nonflashing Waterline Blowdown

Transient Flow Immediately following the rupture, a flow disturbance propagates from the break at a speed of sound relative to the fluid, leaving the fluid behind the wave at a thermodynamic state of U_o and P = P_2. The governing equation across the wave is:

$$\Delta P = \pm \frac{RC}{g} \Delta U \qquad (3.6A\text{-}2)$$

Where:

ΔP = Differential pressure across wave, psia

ΔU = Differential velocity across wave, fps

C = Speed of sound in fluid, fps

When the disturbance reaches a pressure reservoir, it is reflected and travels toward the break end. The boundary conditions that govern the flow at the break location and at the inlet to the pipe (from the reservoir) are:

$P_e = P_a$

$$P_i = P_o - \frac{RU_i^2}{2g} \qquad (3.6A\text{-}3)$$

Where:

P_i = Pressure at pipe inlet, psia

U_i = Velocity of fluid at pipe inlet, fps

P_e, P_a = Pressure at the break location, psia

P_o = Reservoir pressure, psia

The initial blowdown flow remains constant until the disturbance, which is reflected from the vessel, reaches the break end. Then it is reflected again, and that brings a change of blowdown flow. These repeated wave transmissions and reflections continue until the steady-state flow is established.

<u>Steady-State Flow</u> For steady-state flow, the blowdown forcing function calculations become:

$$F = \left[\frac{2(P_o - P_a)}{P_o} \frac{1}{1 + \frac{fL_e}{D}} \right] P_o A \qquad (3.6A\text{-}4)$$

which is derived by applying Bernoulli's equation across the pipe and by using the expression for the forcing function calculation.

Where:

L_e = Total equivalent length of pipe friction, ft

f = Friction factor (Reynolds number and pipe surface roughness dependent)

D = Pipe inside diameter, ft

<u>Steamline Blowdown Transient Flow</u> Steam is treated as an ideal, single-phase gas with a constant specific heat ratio, k, of 1.3.

Except for the case of steady-state blowdown flow, the flow is assumed to be isentropic with negligible pipe friction. The

characteristic method, which is a finite difference approximation using the principle of characteristics, is used as a basis for the numerical solution of the continuity and momentum equations[7,8]. The transient pressure, mass flow rate, and other thermodynamic properties are then used to calculate the transient-state forcing function.

Immediately following the break, a decompression wave travels into the pipe toward the pressure reservoir. The fluid in front of the wave is at a state:

$$U_i = O$$

$$C_i = C \qquad (3.6A\text{-}5)$$

Where:

U_i = Velocity of fluid in front of the wave, fps

C_o = Speed of sound in fluid with respect to the state of fluid at the reservoir, fps

C_i = Speed of sound in fluid in front of wave

The fluid state at the exit is at the sonic condition, because the initial pressure was sufficiently high[9]:

$$\frac{U_e}{C_o} = \frac{C_e}{C_o} = \frac{2}{k+1} = 0.8695 \; for \; k = 1.3 \qquad (3.6A\text{-}6)$$

The blowdown force can be calculated as:

$$F = \left[\frac{P_e}{P_o} + \frac{R_e C_e^2}{gP_o}\right] P_o A = \left[\frac{P_e}{P_o} + \frac{R_e}{R_o}\left(\frac{C_e}{C_o}\right)^2 \frac{R_o c_o^2}{gP_o}\right] P_o A$$

(3.6A-7)

The pressure ratio across the wave is:

$$\frac{P_e}{P_o} = \left(\frac{T_e}{T_o}\right)^{\frac{k}{k-1}} = \left(\frac{C_e}{C_o}\right)^{\frac{2k}{k-1}} = \left(\frac{2}{k+1}\right)^{\frac{2k}{k-1}} = 0.298 \qquad (3.6A\text{-}8)$$

Where:

T = Temperature, and the density ratio is:

$$\frac{R_e}{R_o} = \left(\frac{P_e}{P_o}\right)^{\frac{1}{k}} = \left(\frac{C_e}{C_o}\right)^{\frac{2k}{k-1}\left(\frac{1}{k}\right)} = \left(\frac{2}{k+1}\right)^{\frac{2}{k-1}}$$

Therefore, the blowdown force can be reformulated as:

$$F = (1+k)\left(\frac{2}{k+1}\right)^{\frac{2k}{k-1}} P_o A = 0.685\ P_o A \qquad (3.6A\text{-}9)$$

The blowdown force is constant until a return signal from the pressure source reaches the break. When the wave reaches the reservoir, it is reflected as a compression wave. The boundary condition at the pressure lies on the steady-state ellipse:

$$\left(\frac{C_i}{C_o}\right)^2 + \frac{k-1}{2}\left(\frac{U_i}{C_o}\right)^2 = 1 \qquad (3.6A\text{-}10)$$

which is the energy equation applying across the vessel-pipe inlet. The boundary condition for this case is:

$$T_o = T_i + \frac{U_i^2}{2C_p} \qquad (3.6A\text{-}11)$$

Where:

C_p = Constant pressure specific heat of a fluid, Btu/lbm °F

i = State at the inlet to the pipe

J = 778 x 32.2 = 25,052 $\frac{Pbm \bullet ft^2}{Btu \bullet Sec^2}$

<u>Steady-State Flow</u> If the steady state is reached, the flow in the pipe is uniform, and if the pressure in the pressure vessel remains high, then the boundary condition at the break always lies on the sonic line, that is:

$$\frac{U^*}{C_o} = \frac{C^*}{C_o} \qquad (3.6A\text{-}12)$$

Then from the critical flow condition:

$$\frac{U^*}{C_o} = \frac{C^*}{C_o} = \sqrt{\frac{2}{k+1}} = 0.9325 \qquad (3.6A\text{-}13)$$

Where:

* = Critical flow condition

Then, the steady-state blowdown force is:

$$F = \left(\frac{P^*}{P_o} + \frac{R^*(U^*)^2}{g} \right) P_o A = (1+k) \left(\frac{2}{k+1} \right)^{\frac{k}{k-1}} P_o A = 1.255\, P_o A$$

(3.6A-14)

For steady-state flow with friction losses, the analysis is based on the theory of compressible flow with friction[9]. The pipe friction is the chief factor bringing about the change of fluid properties in the flow. A curve that describes the variation of steady-state steam blowdown force versus friction parameter fL/D is shown on Figure 3.6A-2.

3.6A.2.2.3 Simplified Blowdown Analysis

A conservative steady-state forcing function may be used for calculations based on the energy balance method. The function has a magnitude of:

$$T = KPA \qquad (3.6A\text{-}15)$$

Where:

P = System pressure prior to pipe break, psia

A = Pipe break area, sq in

K = Thrust coefficient

Theoretical maximum K values are as follows:

1.26 for saturated steam, water, and steam/water mixture

2.00 for nonflashing subcooled water

Where pipe rebound may occur upon impact on the restraint, an amplification factor between 1.0 and 1.1 is applied to the above force. Justification for the use of amplification factors less than 1.1 can be found in Reference 13. In this reference, the results of nonlinear dynamic analysis were compared to the results of the corresponding analyses based on the energy balance method, which assumes constant blowdown forces and instantaneous development of a dynamic plastic hinge. The pipe will rotate around the hinge as a rigid body. The appropriate correlation factors required to match the energy balance results with the dynamic analyses results were correlated to design parameters such as restraint location from the break, yield strength of energy-absorbing components, and restraint gap. It was shown that the correlation factors were related to a single, nondimensional parameter and could be limited to a value of 1.0 if appropriate design parameters were chosen. Since actual results and designs for Unit 2 were used in the reference

analysis, this validates the use of factors less than 1.1. Alternatively, the maximum fluid force during the energy input phase, as determined by the detailed methods of Section 3.6A.2.2.2, may be used. In determining this maximum, a brief initial force of PA may be ignored since the initial pipe velocity is low and the resulting work input is inconsequential. The above amplification factor for rebound is also included.

3.6A.2.2.4 Lumped-Parameter Dynamic Analysis

The piping system is modeled mathematically as a series of beam elements connected at nodes. The geometry of the model matches that of the pipe. The distributed mass of the pipe and contained fluid is modeled as lumped masses located at the nodes. The beam elements have the stiffness properties of the pipe in the elastic range and approximate the plastic behavior after yield.

Before a rupture, the pipe is stressed by internal pressure, but remains in static equilibrium. When initial conditions have a significant effect on the parameters being calculated, such as stresses in break exclusion regions or loads on attached components, this effect is considered.

As a circumferential break propagates before the pipeline breaks up into two sections, the load-carrying metal area at the break of the pipe decreases so that a force unbalance results. The force initially transmitted across the break is assumed to drop linearly to zero in 1 msec. After the break, the forces exerted on the pipe by the fluid are determined by the time-dependent blowdown force derived in Section 3.6A.2.2.2. Similarly, for a longitudinal split, the crack propagation speed limits the rate at which the split opens, so a 1-msec force rise time is assumed. Other break opening times may be used if justified.

Subsequent to a postulated rupture, the inelastic system response is analyzed by the use of an elastic-plastic lumped-mass beam element computer code such as DINASAW or LIMITA (Appendix 3A). The analysis considers the free motion of the pipe through a gap, if one exists, using the appropriate initial conditions and the fluid blowdown forces as calculated in Section 3.6A.2.2.2. The mathematical model includes the restraint or barrier, and sometimes a member simulating the local crush resistance of the pipe. Rebound effects are considered by automatically connecting and disconnecting that member for impact and rebound, respectively.

<u>Sample Dynamic Analysis</u>

Pipe rupture restraint 2RHS*PRR004 inside the containment limits the motion of the RHR line following a circumferential break at the elbow. The restraint is an omnidirectional-type restraint (Figure 3.6A-10) with a 1.815-in gap between the hot pipe and the restraint.

The analysis of the pipe-restraint interaction used the LIMITA computer codes. The finite-element model is shown on Figure 3.6A-3. The fluid forces depicted on Figure 3.6A-4 were applied to the pipe elbow as shown on Figure 3.6A-3.

The restraint reaction load is shown on Figure 3.6A-5. The maximum restraint load is 388 kips and the maximum deformation of the honeycomb panel is 2.62 in. The corresponding working strain is 0.524 in/in, which is less than the allowable strain of 0.56 in/in.

3.6A.2.2.5 Energy Balance Analysis

The energy balance technique for analyzing pipe impact equates the work done by the escaping fluid to the energy absorbed in deforming the ruptured pipe and the impacted target. A steady-state blowdown force is used for the energy balance analysis. The magnitude of the force is described in Section 3.6A.2.2.3.

The input energy of the system is determined by multiplying the pipe displacement at the break end by the component of the fluid blowdown force in the direction of the displacement.

The input energy is:

$$E = F \times D \qquad (3.6A\text{-}16)$$

Where:

F = Component of blowdown force in direction of pipe displacement, lb
D = Displacement of break end of pipe, in

The strain energy absorbed during pipe whip and impact consists of the energy absorbed by pipe bending, E_{pb}, the energy absorbed by pipe crush during impact, E_{pc}, and the energy absorbed by deformation of the target, E_f.

To determine postimpact target deformation and the peak reaction force, the input energy is equated to the strain energy absorbed by the pipe and target. The energy absorption characteristics of the pipe crush and target deformation are calculated on the basis of the displacement integral of the appropriate force-deformation curves.

Sample Energy Balance Analysis

The same RHR restraint, 2RHS*PRR004 (Figure 3.6A-6), is analyzed here by the energy balance technique. The energy input from the fluid blowdown force is:

$$E_{input} = F_b \ (g + d) \left(\frac{L_h}{L_h - L} \right) \qquad (3.6A\text{-}17)$$

Where:

F_b = Fluid blowdown force, kips

g = Acceleration gap of restraint, in

d = Restraint deflection, in

L_h = Length from break to plastic hinge, in

L = Length from break to restraint, in

The ratio $L_h/(L_h\text{-}L)$ represents the increased pipe displacement at the break, compared to displacement at the restraint, due to the assumed pipe rotation about a plastic hinge.

The fluid force is calculated:

$$F_b = k_r k_f P_o A = 287 \text{ kips} \qquad (3.6A\text{-}18)$$

Where:

k_r = Rebound factor (1.1)

k_f = Thrust coefficient (0.92), reduced from 1.26 after accounting for friction between the source and break

P_o = Initial pressure (1,028 psi)

A = Pipe flow area (252.719 sq in)

This energy may be absorbed in plastic bending of the pipe and in crush of the restraint. The energy absorbed by bending at the plastic hinge is:

$$E_b = M_p = M_p(g+d)/(L_h\text{-}L)$$

$$E_b = 932 \text{ in-kips} \qquad (3.6A\text{-}19)$$

Where:

M_b = Plastic moment of the pipe

= $\sigma_y \ (D_o^3 - D_i^3)/6$

= $27{,}510 \ (20^3\text{-}17.938^3)/6$

= 1.0216×10^4 in-kips

L_h = $3\ M_p/F_b = 106.8$ in

L = 54 in

g = 1.815 in

d = Allowable crushing of the honeycomb panel

= 0.8 (total crushable depth of honeycomb panel)

= 0.8 x 0.7 x 5.0 = 3.0 in

The energy absorbed in the honeycomb panel is:

$$E_a = F \times d \tag{3.6A-19}$$

By equating E_{in} to (E_a+E_b), the restraint reaction force F is found to be in 621 kips, and the honeycomb panel is crushed 3.0 in.

3.6A.2.2.6 Local Pipe Indentation

The local shell indentation stiffness of the pipe is usually considered where other energy-absorbing mechanisms are not available at the point of impact. Examples include impacts into rigid displacement-limiting bumpers, concrete walls, and the omnidirectional restraint weldment (the latter interposes a significant mass between the impacting pipe and the energy absorbers).

Two methods have been used to determine the shell indentation stiffness. The earlier was analytical and tended to overpredict conservatively the indentation stiffness. The other was a series of pseudostatic pipe crush tests covering several crush geometries and a sufficient range of pipe thicknesses and diameters to develop parametric scaling laws[10,11]. This was augmented by analyses to determine the sensitivity to material strength, dynamics, and variations in loading geometry.

3.6A.2.2.7 Concrete Barrier Impact

In a pipe whip impact, the force on the barrier is a complex function of time depending primarily on the sudden deceleration of the pipe wall at the impact point (slug impact), the shell indentation of the pipe as it locally crushes against the wall, and the force transmitted to the impact point by the more gradual deceleration of the adjacent run of pipe. After impact, the pipe also transmits a more enduring force resulting from the continuing fluid blowdown. The concrete is affected in the same way as in any other missile impact event, the only significant difference being the long-term fluid force. To evaluate this postulated event, the pipe is transformed into an equivalent missile and the concrete is analyzed for scabbing and structural response using the procedure described in Section 3.5.3. The analysis for structural response includes the impulse of the

initial impact as well as the subsequent fluid blowdown force and other concurrent loads.

Four basic parameters must be determined to define the equivalent missile: kinetic energy (or impulse), impact velocity, pipe crush stiffness, and bearing area. The kinetic energy and velocity can be found by either of two methods:

1. Simplified Method Use the total input energy (fluid blowdown force times distance of pipe travel) less the energy absorbed in pipe bending prior to impact. Compute the velocity using approximate formulas.

2. Lumped-Parameter Dynamic Analysis (Section 3.6A.2.2.4) This method is especially suited for evaluating the impact of piping systems with complex geometries and can even consider multiple impact points. As an alternative to the kinetic energy, the impact force history (impulse) can be computed.

Regardless of which analysis method is used, the crush resistance of the equivalent missile and the bearing area are derived from the experimental data described in Section 3.6A.2.2.6. These data are modified to account for the effect of dynamics and internal pressure.

3.6A.2.3 Dynamic Analysis Methods to Verify Integrity and Operability

Pipe rupture loads to determine the integrity of mechanical components are determined using the analytical methods described in Section 3.6A.2.2. The applicable load combinations for the components and for break exclusion regions are presented in Sections 3.9A and 3.6A.2.1.5 (Criteria for Outside the Containment, Item 2), respectively. Criteria for rupture restraints are presented in Section 3.6A.2.3.1.

3.6A.2.3.1 Jet Impingement Analysis

Jet impingement loadings are determined as follows:

1. Jet forces are represented by time-dependent forcing functions. The effects of the piping geometry, capacity of the upstream energy reservoir, source pressure, and fluid enthalpy are considered in these forcing functions.

2. The steady-state jet force has a magnitude of:

$$T_j = K_j P A \qquad (3.6A\text{-}20)$$

Where:

P = System pressure prior to pipe break, psia

A = Pipe break area, sq in

K_j = Jet coefficient

The following K_j values are used whenever the reservoir pressure is constant, pipe friction is negligible, and there are no upstream flow restrictions:

1.26 for saturated steam, saturated water, and steam/water mixture.

2.00 for nonflashing subcooled water

3. In calculating the jet impingement load on a distant object or target, the retarding action of the surrounding air along the jet path is neglected. The jet impingement pressure on an effective target is calculated by taking the jet force as being constant at all effective distances from, and normal to, the break area and by assuming that the jet stream diverges conically at a solid angle of 20 deg. However, for those cases where the 20-deg divergence assumption is shown to be unnecessarily conservative for the blowdown of steam or steam-water mixtures, Moody's asymptotic jet expansion model is adopted[12].

4. The proportion of the total jet force acting on the target is determined from the fraction of the jet intercepted and by the shape factor of the target. For a target with its flat surface area normal to the center of the jet stream, the impingement load is the product of the pressure and the intercepted jet area. For those cases where the target area is such that the intercepted jet stream is deflected rather than totally stopped, a shape factor that is less than unity and is a function of the target geometry is used in calculating the total jet impingement load.

Since the jet impingement force is a dynamically applied load, the target is analyzed either by static methods using an appropriate dynamic load factor, or dynamically using elastic structural response computer codes such as STARDYNE or inelastic structural response computer codes such as DINASAW or LIMITA (Appendix 3A). The load combinations and design allowables are given in Sections 3.8 and 3.9A.

3.6A.2.3.2 Pipe Rupture Restraints

Two basic restraint types are used, elastic and energy-absorbing. Energy-absorbing restraints are used where the primary objective is to dissipate the energy of a ruptured pipe.

The elastic portions of pipe rupture restraints and intermediate structures (auxiliary steel) are designed in accordance with the loads, loading combinations, and stress limits outlined in Section 3.8.4.3 for steel structures except for the following: for abnormal loading conditions, the allowable stress for combination 7 is 1.6S, and for combination 8 the allowable stress is 1.7S.

Only energy-absorbing components (e.g., honeycomb panels and studs) of pipe rupture restraints are designed for strain beyond the elastic limit.

The design of pipe rupture restraints and supporting structures includes the forces induced by SSE; therefore, failure cannot occur during a seismic event.

The portion of the pipe rupture restraint that contacts and supports the process pipe is designed to meet the pipe support requirements of ASME Section III (Section 3.9).

Elastic Restraints

Since elastic restraints are used to minimize displacements of the broken pipe, they are close-gapped. For some applications, this requires that they contact the pipe during conditions other than a postulated rupture, in which case they are designed as a pipe support. If an elastic restraint only contacts the pipe following a rupture, it is designed according to the criteria for structural steel (Section 3.8.3).

Energy-Absorbing Restraints

Several approaches are used for energy absorption in pipe rupture restraints. In tension, stainless steel studs or straps are used, with a design limit of 50 percent of uniform ultimate strain. In compression, honeycomb panels or pipe are used. Compressive components are designed to 80 percent or less of their energy absorption capacity. Other energy-absorbing devices that may be used are designed to these limits.

One or more of the above energy-absorbing mechanisms are utilized in each of the typical restraints described below. When a single energy-absorbing mechanism is utilized, the design limits are met for the design range of loading directions.

Elastic components of energy-absorbing restraints are designed to the criteria for structural steel (Section 3.8).

Pipe Crush Bumper The pipe crush bumper absorbs impact energy in a direction toward the supporting structure. The energy absorber is a length of pipe placed normal to the axis of the process pipe. Subsequent to a rupture, the bumper pipe is crushed between its support structure and the moving process pipe. This absorbs energy and forms a retaining recess in the bumper pipe.

The bumper pipe is attached to its support by welding and bolting (Figure 3.6A-7).

Laminated Strap Restraint The laminated strap restraint is capable of absorbing impact loads in the outward direction from the supporting structure (Figures 3.6A-8 and 3.6A-9). The energy-absorbing component is a U-shaped strap that consists of multiple strips (depending on energy to be absorbed) of highly ductile material (Type 304 stainless steel).

This laminated design results in great flexibility. If the process pipe contacts the sides of the restraint during an event other than pipe rupture, only negligible loads are transmitted. The design also minimizes bending strains, permitting the strap to act mainly as a membrane during the rupture event.

Omnidirectional Restraint The omnidirectional restraint is capable of absorbing impact loads applied in any direction in the plane of the restraint (Figure 3.6A-10). This restraint consists of a base weldment, an arch, ductile holddown studs on each side of the base weldment, and a honeycomb panel. The primary function of the studs is to absorb energy from impact loads acting outward from the support structure. The honeycomb panel absorbs energy from impact loads acting in an inward direction. Side load impacts are absorbed by the combined action of the studs and honeycomb. A limit stop (Figure 3.6A-11) is a restraint whose design varies slightly with that of an omnidirectional restraint. This restraint is designed primarily to absorb energy from the impact load in the inward impact direction.

Combinations of pipe crush bumpers and laminated straps may also be used to achieve energy absorption over a range of impact directions up to a full 360 deg.

3.6A.2.4 Guard Pipe Assembly Design Criteria

Guard pipes are not used at Unit 2, so this section is not applicable.

3.6A.2.5 Material for the Operating License Review

Pipe break and crack locations are obtained in accordance with the criteria of Section 3.6A.2.1. High-energy piping with break locations identified are provided in isometric drawings (Reference 14). High-energy piping composites (Figures 3.6A-52 through 3.6A-60) have been provided to show graphically the pipes in relation to their branch piping. The stress results that were originally utilized to determine the break types and locations were submitted to the NRC in References 14, 15, and 16. If there are changes in the pipe routing, restraint locations, or stress analysis as a result of plant modifications, the effects on pipe break analyses are evaluated in accordance with established

programs and procedures. The locations of cracks are adequately defined in the FMEA (Appendix 3C).

The augmented ISI plan is discussed in Section 6.6. Pipe whip restraints are designed as discussed in Section 3.6A.2.3.1. The restraint locations and orientation and break locations are shown on Figures 3.6A-12 through 3.6A-49. Jet thrust and impingement forces are determined in accordance with Section 3.6A.2.3.

The effects of breaks and cracks are discussed in detail in Appendix 3C. The results of this appendix are based on the protection evaluation criteria of Section 3.6A.1. Any protective measures to assure a safe shutdown (i.e., barriers, separation, and restraints) are also discussed.

3.6A.3 References

1. Letter from Mr. James Bartlett, NMPC, to Mr. Harold Denton, NRC, dated July 31, 1978.

2. Moody, J. F. Time Dependent Pipe Forces Caused by Blowdown and Flow Stoppage. ASME Paper 73-FE-23, 1973.

3. DeHaller, P. The Application of Graphical Method to Some Dynamic Problems in Gases. Sulzer Technical Review No. 1, pp 6-24, 1945.

4. Rudinger, G. Nonsteady Duct Flow-Wave Diagram Analysis. Dover Publications, Inc., 1969.

5. Owzarek, J. A. Fundamentals of Gas Dynamics. International Textbook Company, Second Printing, Scranton, PA, 1968.

6. Streeter, V. L. and Wylie, E. G. Hydraulic Transients. McGraw-Hill Book Co., New York, NY, pp 32-37, 1967.

7. Jonssen, V. K.; Matthews, L.; and Spalding, D. B. Numerical Solution Procedure for Calculating the Unsteady, One-Dimensional Flow of Compressible Fluid with Allowance for the Effect of Heat Transfer and Friction. ASME Paper 73-FE-23, 1973.

8. Hartree, D. R. Some Practical Methods of Using Characteristics in the Calculation of Non-Steady Compressible Flow. Los Alamos Report LA-HU-1, 1952.

9. Shapiro, A. H. The Dynamics and Thermodynamics of Compressible Fluid Flow, Vol. I and II. Ronald Press, New York, NY, 1953.

10. Gerbert, T. L. Plastic Deformation of Piping Due to Pipe Whip Loading. ASME Paper 74-NE-1, 1974.

11. Peech, J. M.; Roemer, R. E.; Pirotin, S. D.; East, G. H.; and Goldstein, N. A. Local Crush Rigidity of Pipes and Elbows. Paper F3/8, Transactions of the Fourth International Conference on Structural Mechanics in Reactor Technology, San Francisco, CA, August 1977.

12. Moody, F. J. Prediction of Blowdown and Jet Thrust Forces. ASME Paper 69-HT-31, August 6, 1969.

13. Kuo, H. and Durkee, M. Correlation of Energy Balance Method to Dynamic Analysis, SMIRT Paper F5/3, presented at Seventh Structural Mechanics in Reactor Technology Conference, Chicago, IL, August 1983.

14. Letter from Mr. T. E. Lempges, NMPC, to Ms. Elinor G. Adensam, NRC, dated July 18, 1986 (NMP2L 0787).

15. Letter from Mr. C. V. Mangan, NMPC, to Ms. Elinor G. Adensam, NRC, dated August 8, 1986 (NMP2L 0809).

16. Letter from Mr. C. V. Mangan, NMPC, to Ms. Elinor G. Adensam, NRC, dated August 21, 1986 (NMP2L 0832).

TABLE 3.6A-1
(Sheet 1 of 4)

COMPARISON OF PIPE RUPTURE CRITERIA

Inside Containment

Item	Regulatory Guide 1.46	Criteria for Unit 2	Comments
Definition of high-energy line (breaks postulated)	Design temperature >200°F or Design pressure >275 psig	Maximum operating temperature >200°F or Maximum operating pressure >275 psig	BTP APCSB 3-1, Appendix A, p 3.6.1-16. The NRC has accepted this position in the SWESSAR, Section 3.6, p 3.6-1.
Break Location Safety Class 1 piping	1. Terminal ends 2. Intermediate break locations where stress limit S_n >2.0 S_m for ferritic steel S_n >2.4 S_m for austenitic steel (Eq. 10) 3. Cumulative usage factor (U) >0.1 4. Minimum of two intermediate breaks	1. Terminal ends 2. Intermediate break locations where stress limit S_n >2.4 S_m (Eq. 10 and 12 or 13) 3. Cumulative usage factor (U) >0.1	Item 2 reflects latest industrial position and has been submitted to the NRC by other utilities. The NRC has accepted this position in the GESSAR, Section 3.6, pp 3.6.1A, 3.6.2. NRC Generic Letter 87-11 no longer requires a minimum number of breaks.
Safety Class 2 and 3 piping	1. Terminal ends 2. At intermediate locations where stresses >0.8 (S_h+S_A) 3. Minimum of two intermediate breaks	1. Terminal ends 2. At intermediate locations where stresses >0.8 (1.2 S_h + S_A)	BTP MEB 3-1, Paragraph B.1.b.(1).(e), p 3.6.2-10. The NRC has accepted this position in the SWESSAR, Section 3.6, p 3.6-3. NRC Generic Letter 87-11 no longer requires a minimum number of breaks.
Type of Break Circumferential	Not required if pipe diameter ≤1 in	Not required: 1. If pipe diameter ≤1 in 2. At break locations where the circumferential stress is ≥ 1.5 the axial stress	BTP MEB 3-1, Paragraph B.3.a, p 3.6.2-15. The NRC has accepted this position in the SWESSAR, Section 3.6, p 3.6-4D.

TABLE 3.6A-1
(Sheet 2 of 4)

COMPARISON OF PIPE RUPTURE CRITERIA

Inside Containment

Item	Regulatory Guide 1.46	Criteria for Unit 2	Comments
Longitudinal	Not required if pipe diameter <4 in	Not required: 1. If pipe diameter <4 in 2. At terminal ends 3. At break locations where axial stress is ≥1.5 the circumferential stress	BTP MEB 3-1, Paragraph B.3.b, p 3.6.2-15. The NRC has accepted this position in the SWESSAR, Section 3.6, pp 3.6-4E, 4F. NRC Generic Letter 87-11 no longer requires a minimum number of breaks.
Longitudinal break area	Break area is equal to the sum of flow area upstream and downstream of the break	Break area equals flow area	BTP MEB 3-1, Paragraph B.3.b(3), p 3.6.2-16. The NRC has accepted this position in the SWESSAR PSAR, Section 3.6, p 3.6-4F BTP MEB 3-1, Paragraph B.3.b(3), p 3.6.2-16.
Orientation of longitudinal breaks	Parallel to the pipe axis and oriented at any point around the pipe circumference	Axial splits to occur at two diametrically opposed points (but not concurrently) on the piping circumference such that the jet reaction is normal to the plane of the fitting or piping and causes out-of-piping configuration	The NRC has accepted this position in the SWESSAR, Section 3.6, p 3.6-4F.
Definition of terminal end at branch piping connections	Branch connection to a main piping run is a terminal end of the branch piping	If a branch connection to a main piping run is analyzed in the same model, it is not considered a terminal end*	BTP APCSB 3-1, Appendix A, p 3.6.1-17. The NRC has accepted this position in the GESSAR, Section 3.6, p 3.6-2a, Note 4.

TABLE 3.6A-1
(Sheet 3 of 4)

COMPARISON OF PIPE RUPTURE CRITERIA

Outside Containment

Item	Branch Technical Positions MEB 3-1 and APCSB 3-1	Criteria for Unit 2	Comments
Definition of high-energy line (breaks postulated)	Temperature >200°F or Pressure >275 psig	Same as BTP, APCSB 3-1	
Definition of moderate-energy line (cracks postulated)	Temperature <200°F and Pressure <275 psig	Same as BTP, APCSB 3-1	
Break Location Safety Class 1 piping between containment isolation valves (stress limits)	No breaks postulated: 1. If S_n <2.4 S_m (Eq. 10) 2. If 2.4 S_m <S_n <3.0 S_m and (U) <0.1 or 3. If S_n >3.0 S_m (Eq. 10) but S_n <2.4 S_m (Eq. 12 and 13) and (U) <0.1 4. Eq: 9<2.25 S_m	No breaks are postulated. Stress values must be maintained below the limits set by BTP MEB 3-1	
Break Locations Safety Class 2 and 3 piping (stress limits)	Breaks postulated: 1. At terminal ends 2. At intermediate locations where stresses >0.8 (1.2 S_h + S_A)	Same as BTP MEB 3-1 NRC Generic Letter 87-11 no longer requires a minimum number of breaks	
Safety Class 2 and 3 piping between containment isolation (stress limits)	No breaks postulated if stresses <0.8 (1.2 S_h + S_A) Eq: 9<1.8 S_h	No breaks postulated. Stress values must be maintained below the limits set by BTP MEB 3-1	
Nonnuclear class piping	Breaks postulated: 1. At terminal ends 2. At each intermediate pipe fitting, welded attachment, and valve	1. At terminal ends 2. At each intermediate pipe fitting, welded attachment, and valve 3. For piping designed for seismic loads, at intermediate locations where stresses >0.8 S_A	NRC Generic Letter 87-11 no longer requires a minimum number of breaks.
Type of Break Circumferential	Not required: 1. If pipe diameter ≤1 in 2. If stress >0.8 (1.2 S_h+S_A) and the circumferential stress is ≥1.5 the axial stress	Same as BTP MEB 3-1	

TABLE 3.6A-1
(Sheet 4 of 4)

COMPARISON OF PIPE RUPTURE CRITERIA

Outside Containent (Cont'd.)

Item	Branch Technical Positions MEB 3-1 and APCSB 3-1	Criteria for Unit 2	Comments
Longitudinal	Not required: 1. If pipe diameter <4 in 2. At terminal ends 3. At intermediate locations where the criteria for a minimum number of break locations must be satisfied 4. If stress >0.8 (1.2 S_h+S_A) and the axial stress is $\geq$1.5 the circumferential stress	Same as BTP MEB 3-1	NRC Generic Letter 87-11 no longer requires a minimum number of breaks.
Orientation of longitudinal breaks	Axial splits to occur at two diametrically opposed points (but not concurrently) on the piping circumference such that the jet reaction is normal to the plane of the fitting or piping and causes out-of-plane bending of the piping configuration	Same as BTP MEB 3-1	
Cracks (environmental study only)	Pipe diameter >1 in Stresses >0.4 (1.2 S_h+S_A)	Same as BTP MEB 3-1	
Definition of terminal end at branch piping connections	If a branch connection to a main piping run is analyzed in the same model, it is not considered as a terminal end.	Same as BTP APCSB 3-1*	

* The criteria have been clarified in Section 3.6A.2.1.5 to incorporate the current NRC understanding of the definition as follows:

A branch connection to main run is considered a terminal end unless the following are met:

1. The branch connection and main runs are of comparable size and fixity (i.e., the nominal size of the branch run is at least one-half that of the main run).
2. The intersection is not rigidly constrained by the building structure.
3. The branch and main runs are modeled as a common piping system during pipe stress analysis.

NOTE: This table has been extracted from the Reference 1 letter.

TABLES 3.6A-2 THROUGH 3.6A-71 HAVE BEEN DELETED.

TABLE 3.6A-72
(Sheet 1 of 2)

ESSENTIAL SYSTEMS/COMPONENTS/EQUIPMENT EVALUATED FOR PIPE FAILURES

PART I

Inside Containment
(QA Category I Portions Only Unless Otherwise Noted)

1. Reactor coolant pressure boundary (up to and including the containment isolation valves)

2. Containment isolation system and containment boundary (including liner plate)

3. Reactor protection system (instruments associated with SCRAM signals)

4. Emergency core cooling systems (for LOCA only)

    a. HPCS (CSH)

    b. LPCS (CSL)

    c. LPCI (RHS)

    d. ADS, including SVV valves and discharge lines (SVV)

5. Core cooling systems (other than ECCS)

    a. RCIC (ICS)

    b. RHR shutdown cooling mode (RHS)

6. Control rod drive system

7. Containment heat removal systems

    a. RHR suppression pool cooling mode (RHS)

    b. RHR containment spray mode (RHS)

8. Neutron monitoring system (NMS)

9. Reactor recirculation system including hydraulic lines to hydraulic actuators (includes Class 4 portions) (RCS)

10. Hydrogen recombiner system (HCS)

11. Containment atmosphere monitoring system (CMS)

12. Reactor vessel instrumentation (ISC)

13. The following equipment/systems, or portions thereof, are required to ensure the proper operation of those essential items listed in Items 1 through 12:

    a. Class 1E electrical system

    b. Reactor plant component cooling water (CCP) (excluding Class 4 portions)

    c. Instrument air to ADS (IAS)

    d. Safety-related instrumentation and instrumentation piping

TABLE 3.6A-72
(Sheet 2 of 2)

ESSENTIAL SYSTEMS/COMPONENTS/EQUIPMENT EVALUATED
FOR PIPE FAILURES

PART II

Outside Containment
(QA Category I Portions Only Unless Otherwise Noted)

1. Containment isolation system and containment boundary up to and including the outermost containment isolation valves

2. Reactor protection system (instruments associated with SCRAM signals)

3. Core and containment cooling systems

    a. HPCS (CSH)

    b. RCIC (ICS)

    c. LPCS (CSL)

    d. LPCI (RHS)

    e. RHR shutdown cooling mode (RHS)

    f. RHR containment spray mode (RHS)

    g. RHR suppression pool cooling mode (RHS)

4. Containment atmosphere monitoring system (CMS)

5. Control rod drive system (including Class 4 portions) (RDS)

6. Spent fuel pool cooling system (SFC)

7. Reactor water cleanup (ASME Classes 1, 2, and 3 only) (WCS)

8. Reactor vessel instrumentation system (ISC)

9. The following equipment/systems, or portions thereof, are required to ensure the proper operation of those essential items listed in Items 1 through 8:

    a. Class 1E electrical systems, ac and dc (including diesel generator system, emergency buses, motor control centers, switchgear, batteries, auxiliary shutdown control panel, and distribution systems)

    b. Service water system (SWP) including reactor plant component cooling water (CCP) (excluding Class 4 portions)

    c. Safety-related environmental systems (HVK, HVN, HVR, HVP, HVY, HVC)

    d. Safety-related instrumentation and instrument piping/tubing, including leak detection instrumentation

    e. Instrument air (IAS) and neutron monitoring (GSN) supply to ADS

10. Reactor recirculation system hydraulic lines to hydraulic actuators (includes Class 4 portion) (RCS)

TABLE 3.6A-73
(Sheet 1 of 1)

HIGH-ENERGY PIPING

Inside Containment

PART I

Piping System

Main steam system (MSS)

Main steam drains

Reactor core isolation cooling (RCIC) system - steam

Reactor core isolation cooling (RCIC) system - head spray

Feedwater system (FWS)

Recirculation system (RCS)

High-pressure core spray system (HPCS) (reactor pressure vessel (RPV) to first check valve)

Low-pressure core spray system (LPCS) (reactor pressure vessel (RPV) to first check valve)

Reactor water cleanup (RWCU) system

Main steam vent lines

Residual heat removal (RHR) system (shutdown suction)

Residual heat removal (RHR) system - low-pressure core injection (LPCI)

Control rod drive (CRD) system

Standby liquid control system (SLCS)

Main steam safety relief valve (SRV) piping (between the main steam line and the safety relief valve)

TABLE 3.6A-73
(Sheet 1 of 1)

HIGH-ENERGY PIPING

Outside Containment

PART II

Piping System

Main steam system (MSS)

Main steam drains

Feedwater system (FWS)

Reactor core isolation cooling (RCIC) system (steam to RCIC turbine)

Control rod drive (CRD) system

Reactor water cleanup (RWCU) system (including residual heat removal (RHR) feed to feedwater)

3.6B PROTECTION AGAINST DYNAMIC EFFECTS ASSOCIATED WITH POSTULATED RUPTURE OF PIPING (GE SCOPE OF SUPPLY)

See Section 3.6A for an explanation of GE/SWEC scope of supply. The following high-energy systems are in the scope of Section 3.6B.

3.6B.1 Postulated Piping Failures in Fluid Systems Outside Containment

See Section 3.6A.1.

3.6B.2 Determination of Break Locations and Dynamic Effects Associated with the Postulated Rupture of Piping

Information concerning break and crack location criteria and methods of analysis is presented in this section. The location criteria and methods of analysis are needed to evaluate the dynamic effects associated with postulated breaks and cracks in high- and moderate-energy fluid system piping inside and outside the primary containment. This information confirms that the requirements for the protection of structures, systems, and components relied upon for safe reactor shutdown or to mitigate the consequences of a postulated pipe break have been met.

3.6B.2.1 Criteria Used to Define Break and Crack Location and Configuration

The following section establishes the criteria for the location and configuration of postulated breaks and cracks.

3.6B.2.1.1 Criteria for Recirculation Piping System Inside Containment

3.6B.2.1.1.1 Definition of High-Energy Fluid System

High-energy fluid systems are defined as those systems, or portions of systems, that during normal plant conditions[2] are either in operation or are maintained pressurized under conditions where either or both of the following are met:

Maximum operating temperature exceeds 200°F.

Maximum operating pressure exceeds 275 psig.

3.6B.2.1.1.2 Definition of Moderate-Energy Fluid System

Moderate-energy fluid systems are defined as those systems, or portions of systems, that during normal plant conditions are either in operation or are maintained pressurized under conditions where both of the following are met:

---

[2]Normal plant conditions are defined as the plant operating conditions during reactor startup, power, operation hot standby, or reactor cold shutdown.

Maximum operating temperature is 200°F or less, and

Maximum operating pressure is 275 psig or less.

Piping systems are classified as moderate-energy systems when they operate as high-energy piping for only short operational periods in performing their system function, but for the major operational period qualify as moderate-energy fluid systems. An operational period is considered "short" if the total fraction of time that the system operates within the P-T conditions specified for high-energy fluid systems is less than 2 percent of total operating time the system is designed for.

3.6B.2.1.1.3 Postulated Pipe Breaks and Cracks

A postulated pipe break is defined as a sudden, gross failure of the pressure boundary either in the form of a complete circumferential severance (guillotine break) or as development of a sudden longitudinal crack (longitudinal split), and is postulated for high-energy fluid systems only. For moderate-energy fluid systems, pipe breaks are confined to postulation of leakage cracks in piping and branch runs. These cracks affect the surrounding environmental conditions only and do not result in whipping of the cracked pipe.

The following high-energy piping systems (or portions of systems) are considered to have potential for initiation of a postulated pipe break during normal plant conditions, and are analyzed for potential damage due to dynamic effects:

1. All piping that is part of the RCPB and subject to reactor pressure continuously during plant operation.

2. All piping that is beyond the second isolation valve but is subject to reactor pressure continuously during plant operation.

3. In addition to piping under 1 and 2, all other piping systems or portions of piping systems considered high-energy systems.

Portions of piping systems that are isolated from the source of the high-energy fluid during normal plant conditions are exempted from consideration of postulated pipe breaks. This would include portions of piping systems beyond a normally closed valve. Pump and valve bodies are also exempted from consideration of pipe break because of their greater wall thickness.

A high-energy piping system break is not postulated simultaneously with a moderate-energy piping system crack, nor is any pipe break or crack outside containment postulated concurrently with a postulated pipe break inside containment. Only the high-energy piping system break is postulated except where a postulated leakage crack in the moderate-energy fluid

system piping results in more severe environmental conditions than the break in the approximate high-energy fluid piping system.

3.6B.2.1.1.4 Exemptions from Pipe Whip Protection Requirements

Protection from pipe whip need not be provided if any one of the following conditions exists:

1. Piping is classified as moderate-energy piping.

2. Following a single postulated pipe break, piping for which the unrestrained movement of either end of the ruptured pipe in any feasible direction about a plastic hinge, formed within the piping, cannot impact any structure, system, or component important to safety.

3. Piping for which the internal energy level associated with whipping is insufficient to impair the safety function of any structure, system, or component to an unacceptable level. Any line restrictions (e.g., flow limiters) between the pressure source and break location, and the effects of either a single-ended or double-ended flow condition are accounted for, in the determination of the internal fluid energy level associated with the postulated pipe break reaction. The energy level in a whipping pipe is considered insufficient to rupture an impacted pipe of equal or greater nominal pipe size and equal or heavier wall thickness.

3.6B.2.1.1.5 Location for Postulated Pipe Breaks (ASME Safety Class 1 Piping)

Postulated pipe break locations are selected in accordance with the intent of RG 1.46, BTP APCSB 3-1, Appendix B, and as expanded in BTP MEB 3-1. For ASME Section III, Safety Class 1 piping systems classified as high energy, the postulated break locations are:

1. The terminal ends of the pressurized portions of the run. (Terminal ends are extremities of piping runs that connect to structures, components, or pipe anchors that act as rigid constraints to piping motion and thermal expansion. A branch connection to a main piping run is a terminal end for a branch run, except when the branch run is modeled as a part of the piping system in the stress analysis, and is shown to have a significant effect on the main run behavior.)

2. At intermediate locations between the terminal ends, where the maximum stress range between any two load sets (including zero load set) according to ASME Section III Subarticle NB-3600 for upset plant

conditions and an independent OBE event transient, exceeds the following:

a. If the stress range calculated using Equation 10 of the Code exceeds 2.4 S_m but is not greater than 3 S_m, no breaks are postulated unless the CUF exceeds 0.1.

b. The stress ranges, as calculated by Equations 12 or 13 of the Code, exceed 2.4 S_m or if the CUF exceeds 0.1 when Equation 10 exceeds 3 S_m.

3.6B.2.1.1.6 Other High-Energy Piping and Moderate-Energy Piping

There are no piping components in this section other than ASME Safety Class 1.

3.6B.2.1.1.7 Regulatory Guide 1.46

RG 1.46 describes an acceptable basis for selecting the design locations and orientations of postulated breaks in fluid system piping within the reactor containment, and for determining the measures that should be taken for restraint against pipe whipping that may result from such breaks.

GE-supplied NSSS analysis, design, and/or equipment utilized in this facility is in compliance with the intent of RG 1.46 through the incorporation of the following alternate approach. See the regulatory guide commitment matrix in Section 1.8 for commitment, revision number, and scope.

The recirculation piping also has been analyzed for the effects of hydrodynamic loads and the pipe break criteria of NUREG-0800. The analysis shows that the SRP criteria do not result in any additional pipe breaks beyond those using the design basis criteria (Section 3.6B.2.1.1.5).

3.6B.2.1.1.8 Types of Breaks to Be Postulated in Fluid System Piping

The following types of breaks are postulated in high-energy fluid system piping:

1. No breaks need be postulated in piping having a nominal diameter less than or equal to 1 in.

2. Circumferential breaks are postulated in piping exceeding a 1-in nominal pipe diameter.

3. Longitudinal splits are postulated only in piping having a nominal diameter equal to or greater than 4 in.

4. Circumferential breaks are to be assumed at all terminal ends and at intermediate locations chosen to

satisfy the minimum break location criteria (Section 3.6B.2.1.1.5) for Safety Class 1 piping systems. At each of the intermediate postulated break locations identified to exceed the stress and usage factor limits of the criteria for Safety Class 1 piping systems, either a circumferential or a longitudinal break, or both, are postulated in accordance with the following:

a. Circumferential breaks are postulated at fitting joints.

b. Longitudinal breaks are postulated in the center of the fitting at two diametrically opposed points (but not concurrently), located so that the reaction force is perpendicular to the plane of the piping and produces out-of-plane bending.

c. Consideration is given to the occurrence of either a longitudinal or circumferential break. Examination of the state of stress in the vicinity of the postulated break location may be used to identify the most probable type of break. If the maximum stress range in the longitudinal direction is greater than 1.5 times the maximum stress range in the circumferential direction, only the circumferential break may be postulated, and conversely if the maximum stress range in the circumferential direction is greater than 1.5 times the stress range in the longitudinal direction, only the longitudinal break may be postulated. If no significant difference between the circumferential and longitudinal stresses is determined, then both types of breaks are considered.

5. For design purposes, a longitudinal break area is assumed to be the equivalent of one circumferential pipe area.

6. For both longitudinal and circumferential breaks, after assessing the contribution of upstream piping flexibilities, pipe whipping is assumed to occur in the plane defined by the piping geometry and configuration for circumferential breaks and out-of-plane for longitudinal breaks, and to cause pipe movement in the direction of the jet reaction.

7. For a circumferential break, the dynamic force of the jet discharge at the break location is based upon the effective cross-sectional flow area of the pipe and on a calculated fluid pressure as modified by an analytically or experimentally determined thrust coefficient. Justifiable line restrictions, flow limiters, and the absence of energy reservoirs are

used, as applicable, in the reduction of the jet discharge.

The through-wall leakage cracks are postulated in moderate-energy fluid systems (or portions of systems). There are no moderate-energy piping components in this section.

3.6B.2.1.2 Criteria for Piping System in Area of Containment Isolation Valves

There are no containment penetrations associated with this section on the reactor recirculation piping system.

3.6B.2.2 Analytical Methods to Define Blowdown Forcing Functions and Response Models

3.6B.2.2.1 Analytical Methods to Define Blowdown Forcing Functions

Rupture of a pressurized pipe causes the flow characteristics of the system to change, creating reaction forces that can dynamically excite the piping system. The reaction forces are a function of time and space and depend upon fluid state within the pipe prior to rupture, break flow area, frictional losses, plant system characteristics, piping system, and other factors. The methods used to calculate the reaction forces for various piping systems are presented in the following sections.

3.6B.2.2.1.1 Recirculation Piping System

The criteria used for calculation of fluid blowdown forcing functions include:

1. Circumferential breaks are assumed to result in pipe severance and separation amounting to at least a one pipe diameter lateral displacement of the ruptured piping sections unless physically limited by piping restraints, structural members, or piping stiffness as may be demonstrated by the inelastic pipe whip analysis (Section 3.6B.2.2.2).

2. For a circumferential break, the dynamic force of the jet discharge at the break location is based upon the effective cross-sectional flow area of the pipe and on a calculated fluid pressure as modified by an analytically or experimentally determined thrust coefficient. Justifiable line restrictions, flow limiters, and the absence of energy reservoirs are used, as applicable, in the reduction of the jet discharge.

   For both longitudinal and circumferential breaks, after assessing the contribution of upstream piping flexibilities, pipe whip is assumed to occur in the

plane defined by the piping geometry and configuration for circumferential breaks and out-of-plane for longitudinal breaks, and to cause pipe movement in the direction of the jet reaction.

3. All breaks are assumed to attain full area instantaneously. A rise time not exceeding 1 msec is used for the initial pulse.

Blowdown forcing functions are determined by the following method: The predicted blowdown forces on pipes fed by a pressurized vessel can be described by transient (time dependent) and steady-state forcing functions. The forcing functions used are based on methods described in Reference 1. These may be simply described as follows:

1. The transient forcing functions occur at points along the pipe from the propagation of waves (wave thrust) along the pipe, and at the broken end from the reaction force due to the momentum of the fluid leaving the end of the pipe (blowdown thrust).

2. The waves cause various sections of the pipe to be loaded with time-dependent forces. It is assumed that the pipe is one-dimensional, in that there is no attenuation or reflection of the pressure waves at bends, elbows, and the like. Following the rupture, a decompression wave is assumed to travel from the break at a speed equal to the local speed of sound within the fluid. Wave reflections occur at the break end and the pressure vessel end, until a steady blowdown condition is established. Free space and vessel conditions are used as boundary conditions. The blowdown thrust that is caused by fluid acceleration from the break and static pressure in the break itself causes a time-dependent reaction force perpendicular to the pipe break, reaching a final steady-state value.

3. The initial blowdown force on the pipe is taken as the sum of the wave and blowdown thrusts and is equal to the vessel pressure (Po) times the break area (A). After the initial decompression period (i.e., the time it takes for a wave to reach the first change in direction), the force is assumed to drop off to the value of the blowdown thrust (i.e., 0.7 PoA).

4. Time histories of transient pressure, flow rate, and other thermodynamic properties of the fluid can be used to calculate the blowdown force on the pipe using the following equation:

$$F = \left[(P - P_a) + \frac{\rho v^2}{g_c} \right] A$$

(3.6B-1)

Where:

F = Blowdown force

P = Pressure at exit plane

P_a = Ambient pressure

V = Velocity at exit plane

ρ = Density at exit plane

A = Area of break

g_c = Newton's constant

5. Following the transient period, a steady-state period is assumed to exist. Steady-state blowdown forces are calculated considering frictional effects. ANS-58.2[1] is the base document used for determining thrust coefficients in evaluating the dynamic force due to jet discharge.

   For frictionless flow, the theoretical maximum value of thrust coefficient for subcooled water is 2.0. Frictional effects are then considered to calculate the blowdown forces from this theoretical maximum value.

   The steady-state thrust coefficient for frictionless flow of subcooled water based on the Henry-Fauske model[5] results in the following expression:

   $C_T = 3.0 - 0.861h^{*2}; \; 0 \le h^* \le 0.75$

   $C_T = 3.22 - 3.0h^* + 0.97h^{*2}; \; 0.75 < h^* \le 1.0$

   Where:

   $h^* = (h_o - 180)/(h_{saturated} - 180)$

   h_o = stagnation enthalpy (Btu/lbm)

   Where:

   $h_{saturated}$ = saturated water enthalpy at the stagnation pressure (Btu/lbm)

This model was confirmed by the experimental comparison work of Hanson.[6] For all values of h*, C_T is no greater than 2.0. For recirculation line break, in general, C_T = 2.0 is used for conservatism unless otherwise justified by documented evaluation of the empirical equation.

3.6B.2.2.2 Pipe Whip Dynamic Response Analyses

The prediction of time-dependent and steady-thrust reaction loads caused by blowdown of subcooled, saturated, and two-phase fluid from a ruptured pipe is used in design and evaluation of dynamic effects of pipe breaks. A detailed discussion of the analytical methods employed to compute these blowdown loads is given in Section 3.6B.2.2.1. A detailed discussion of analytical methods used to account for this loading is discussed below.

3.6B.2.2.2.1 Recirculation Piping System

The criteria used for performing the pipe whip dynamic response analyses include:

1. A pipe whip analysis is performed for each postulated pipe break. However, a given analysis can be used for more than one postulated break location if the blowdown forcing function, piping and restraint system geometry, and piping and restraint system properties are conservative for other break locations.

2. The analysis includes the dynamic response of the pipe in question and the pipe whip restraints that transmit loading to the structures.

3. The analytical model adequately represents the mass/inertia and stiffness properties of the system.

4. Pipe whip is assumed to occur in the plane defined by the piping geometry and configuration, and to cause pipe movement in the direction of the jet reaction.

5. Piping contained within the broken loop is no longer considered part of the RCPB. Plastic deformation in the pipe is considered a potential energy absorber. A limit of strain is imposed similar to that on the pipe whip restraint material (Section 3.6B.2.3.3.1, Type 1 restraint design limits).

6. Components such as vessel safe ends and valves that are attached to the broken piping system and do not serve a safety function, or whose failure would not further escalate the consequences of the accident, are not designed to meet ASME Code imposed limits for essential components under faulted loading. However, if these components are required for safe shutdown, or if they

serve a safety function to protect the structural integrity of an essential component, then these components are designed to ASME Code limits for faulted conditions and for limits necessary to ensure operability.

The pipe whip analysis was performed using the PDA (pipe dynamic analysis) computer program to determine the response of a pipe subjected to the thrust force occurring after a pipe break[2]. The program treats the situation in terms of generic pipe break configuration, which involves a straight, uniform pipe fixed at one end and subjected to a time-dependent thrust force at the other end. A typical restraint used to reduce the resulting deformation is also included at a location between the two ends. Nonlinear and time-dependent stress-strain relations are used for the pipe and the restraint. Similar to the popular plastic-hinge concept, bending of the pipe is assumed to occur only at the fixed end and at the location supported by the restraint.

Shear deformation is also neglected. The pipe bending moment-deflection (or rotation) relation used for these locations is obtained from a static nonlinear cantilever beam analysis. Using the moment-rotation relation, nonlinear equations of motion of the pipe are formulated using an energy consideration and the equations are numerically integrated in small time steps to yield time history information about the deformed pipe.

A comprehensive verification program has been performed to demonstrate the conservatisms inherent in the PDA pipe whip computer program and the analytical methods utilized. Part of this verification program includes an independent analysis by Nuclear Services Corporation (NSC), under contract to GE, of the recirculation piping system for the 1969 Standard Plant Design. The recirculation piping system was chosen for study due to its complex piping arrangement and assorted pipe sizes. The NSC analysis included elastic-plastic pipe properties, elastic-plastic restraint properties, and gaps between the restraint and pipe, and is documented in Reference 3. The piping/restraint system geometry and properties and fluid blowdown forces were the same in both analyses. However, a linear approximation was made by NSC for the restraint load-deflection curve supplied by GE. This approximation is demonstrated on Figure 3.6B-1. The effect of this approximation is to give lower energy absorption of a given restraint deflection. Typically, this yields higher restraint deflections and lower restraint-to-structure loads than the GE analysis. The deflection limit used by NSC is the design deflection at one-half of the ultimate uniform strain for the GE restraint design. The restraint properties used for both analyses are provided in Table 3.6B-1.

A comparison of the NSC analysis with the PDA analysis (Table 3.6B-2) shows that PDA predicts higher loads in 15 of the 18 restraints analyzed. This is due to the NSC model including

energy-absorbing effects in secondary pipe elements and structural members. However, PDA predicts higher restraint deflections in 50 percent of the restraints. The higher deflections predicted by NSC for the lower loads are caused by the linear approximation used for the force-deflection curve rather than by differences in computer techniques. This comparison demonstrates that the simplified modeling system used in PDA is adequate for pipe rupture loading, restraint performance, and pipe movement predictions within the meaningful design requirements for these low-probability postulated accidents.

A comprehensive test program was performed to develop the restraint properties such as the load-deflection power relationships shown in Table 3.6B-1. A series of static and dynamic deformation tests of model restraints were conducted. The model restraints were scaled down from the restraints suitable for 26-in size pipe. Also, the static and dynamic material properties were obtained from tensile tests of bar specimens. The results of these tests were studied and analyzed for use in the development of an analytical model that predicts the behavior of a restraint when loaded by a moving pipe. Tests were performed on some full-scale restraints that showed that the pipe whip restraints will perform their designated functions adequately.

3.6B.2.3 Dynamic Analysis Methods to Verify Integrity and Operability

3.6B.2.3.1 Jet Impingement Analyses and Effects on Safety-Related Components

The methods used to evaluate the jet effects resulting from the postulated breaks of high-energy piping are presented in Section 3.6A.

3.6B.2.3.2 Pipe Whip Effects on Safety-Related Components

This section provides the criteria and methods used to evaluate the effects of pipe displacements on safety-related structures, systems, and components following a postulated pipe rupture.

3.6B.2.3.2.1 Pipe Whip Effects Following a Postulated Rupture of the Recirculation Piping System

Pipe whip (displacement) effects on safety-related structures, systems, and components (nozzles, valves, tees, etc.) that are in the same piping run as the one in which the break occurred are determined by:

1. The criteria used for determining the effects of pipe displacements on the in-line components are as follows:

    a. Components such as vessel safe ends and valves

that are attached to the broken piping system and do not serve a safety function, or whose failure would not further escalate the consequences of the accident, need not be designed to meet ASME Section III imposed limits for essential components under faulted loading.

b. If these components are required for safe shutdown, or serve a safety function to protect the structural integrity of an essential component, limits to meet the Code requirements for faulted conditions and limits to ensure operability, if required, are met.

2. The methods used to calculate the pipe whip loads on piping components in the same run as the postulated break are described in Section 3.6B.2.2.2.

3.6B.2.3.3 Load Combinations and Design Criteria for Pipe Whip Restraints

Pipe whip restraints, as differentiated from piping supports, are designed to function and carry load for an extremely low probability gross failure in a piping system carrying high-energy fluid. The piping integrity does not usually depend on the pipe whip restraints for any load combination. When the piping integrity is lost because of a postulated break, the pipe whip restraint acts to limit the movement of the broken pipe to an acceptable distance. The pipe whip restraints (i.e., those devices that serve only to control the movement of a ruptured pipe following gross failure) are subjected to once-in-a-lifetime loading. The pipe break event is considered to be an abnormal condition for the ruptured pipe, its restraints, and the structure to which the restraint is attached. The design and analysis of these components for this event are described in Section 3.6B.2.2 and the following sections.

3.6B.2.3.3.1 Recirculation Piping System Pipe Whip Restraints

The pipe whip restraints designed, tested, and fabricated by GE for the recirculation loop piping utilize energy-absorbing U-rods to attenuate the kinetic energy of a ruptured pipe. A typical pipe whip restraint is shown on Figure 3.6B-2. A principal feature of these restraints is that they are installed with several inches of annular clearance between them and the process pipe. This allows for installation of normal piping insulation and unrestricted pipe thermal movements. Select critical locations inside primary containment are also monitored during hot functional testing to provide verification of adequate clearances prior to plant operation.

The specific design objectives for the restraints are:

1. The restraints will in no way increase the RCPB

stresses by their presence during any normal mode of reactor operation or condition.

2. The restraint system will function to stop the movement of a pipe failure (gross loss of piping integrity) without allowing damage to critical components or missile development.

3. The restraints should provide minimum hindrance to ISI of the process piping.

For the purposes of design, the pipe whip restraints are designed for the following dynamic loads:

1. Blowdown thrust of the pipe section that impacts the restraint.

2. Dynamic inertia loads of the moving pipe section that is accelerated by the blowdown thrust and subsequent impact on the restraint.

3. Design characteristics of the pipe whip restraints are included and verified by the pipe whip dynamic analysis described in Section 3.6B.2.2.2.

4. Since the pipe whip restraints are not in contact with the piping during normal plant operation, the postulated pipe rupture event is the only design loading condition.

The recirculation loop pipe whip restraints are composed of several components, each of which performs a different function. These components are categorized as Types I, II, III, and IV, as follows:

Type I Restraint energy absorption members - Members that, under the influence of impacting pipes (pipe whip), absorb energy by significant plastic deformation (e.g., U-rods).

Type II Restraint connecting members - Components that form a direct link between the restraint plastic members and the structure (e.g., clevises, brackets, pins).

Type III Restraint connecting member structural attachments - Fasteners that provide the method of securing the restraint connecting members to the structure (e.g., weld attachments, bolts).

Type IV Structural and civil components - Steel and concrete structures that ultimately must carry the restraint load (e.g., sacrificial shield, trusses).

Each of these components is typically constructed of a different material, with a different design objective in order to perform the overall design function. Therefore, the material and inspection requirements and design limits for each are somewhat different. These requirements for each component are given below:

1. Type I Restraint (e.g., U-rods)

   a. Materials All materials used to absorb energy through significant plastic deformation conform to:

      (1) ASME Section III, Subsection NB, Boiler and Pressure Vessel Code for Safety Class I components, or

      (2) ASTM Specifications with consideration for brittle fracture control, or

      (3) ASME Section III, Subsection NF, Boiler and Pressure Vessel Code, if applicable.

      (4) GE Material Specifications.

   b. Inspection Inspection and identification of materials conform to:

      (1) ASME Section III, Subsection NB, Boiler and Pressure Vessel Code for Safety Class I components (Section V, Nondestructive Examination Methods), or

      (2) ASTM Specifications procedures including volumetric and surface inspection, or

      (3) ASME Section III, Subsection NF, Boiler and Pressure Vessel Code, if applicable.

      (4) GE Methods and Acceptance Standards.

   c. Design Limits

      (1) Design local strain The permanent strain in metallic ductile materials is limited to:

         (a) 50 percent of the minimum actual ultimate uniform strain (at the maximum stress on an engineering stress-strain curve) based on restraint material tests, or

         (b) One-half of minimum percent elongation as specified in the applicable ASME

Section III Boiler and Pressure Vessel Code or ASTM Specifications, when demonstrated to be as or more conservative than the above.

(2) Design steady-state load The maximum restraint load is limited to 80 percent of the minimum calculated static ultimate restraint strength at the drywell design temperature. This strain is less than 50 percent of the ultimate uniform strain for all materials used for Type I components.

(3) Dynamic material mechanical properties The material selected exhibits tensile and impact properties not less than:

(a) 70 percent of the static percent elongation, or

(b) 80 percent of the statically determined minimum total energy absorption.

2. Type II Restraint (e.g., clevises, brackets, pins)

a. Materials Material selection conforms to:

(1) ASTM Specifications including consideration for brittle fracture control, or

(2) ASME Section III, Subsection NF, Boiler and Pressure Vessel Code, if applicable.

(3) GE Material Specifications.

b. Inspection Inspection conforms to:

(1) ASME/ASTM requirements or process qualification and finished part surface inspection in accordance with ASTM methods, or

(2) ASME Section III, Subsection NF, Boiler and Pressure Vessel Code, if applicable.

(3) GE Methods and Acceptance Standards.

c. Design Limits Design limits are based on the following stress limits:

(1) Primary stresses (in accordance with definitions in ASME Section III) are limited to the higher of:

(a) 70 percent of S_u where S_u = minimum ultimate strength by tests or ASTM specification

(b) $S_y + 1/3 (S_u - S_y)$ where S_y = minimum yield strength by test or ASTM specification, or

(2) Recommended stress limits in accordance with ASME Section III, Subsection NF, for faulted conditions, if applicable.

3. Type III Restraint (fasteners)

a. Materials Fastener material conforms to ASTM, ASME, or MIL requirements.

b. Inspection All fasteners are inspected or certified in accordance with applicable ASTM, ASME, or MIL specifications.

c. Design limits Same as Type II.

4. Type III Restraint Material (welds)

a. Materials Weld materials for attachment to carbon steel structures are limited to low hydrogen types, or processes that are inherently low hydrogen.

b. Inspection Liquid penetrant surface inspection is performed in accordance with:

(1) ASTM Specification E165, or

(2) AWS Structural Welding Codes, AWS-D1.1.

c. Design limits Design limits are based on the following stress limits: the maximum primary weld stress intensity (two times maximum shear stress) is limited to three times AWS or AISC building allowable weld shear stress.

d. Procedures Procedures and welders are qualified in accordance with the latest AWS Code for welding in building structures.

5. Type IV Restraint (structural and civil components) Material, inspection, and design requirements for the structural and civil components are provided by industry standards such as AISC, ACI, and ASME Section III, Division II, along with appropriate requirements imposed for similar loading events. These components are also designed for other operational and accident

loadings, seismic loadings, wind loadings, and tornado loadings.

The design basis approach of categorizing components is consistent in allowing less stringent inspection requirements for those components subject to lower stresses. Considerable strength margins exist in Type II through IV components even to the limit of load capacity (fracture) of a Type I component. Impact properties in all components are considered since brittle type failures could reduce the restraint system effectiveness.

In addition to the design considerations discussed above, strain rate effects and other material property variations have been considered in the design of the pipe whip restraints. The material properties utilized in the design have included one or more of the following methods:

1. Code minimum or specification yield and ultimate strength values for the affected components and structures are used for both the dynamic and steady-state events.

2. Not more than a 10-percent increase in Code or specification values is used when designing components or structures for the dynamic event. Code minimum or specification yield and ultimate strength values are used for the steady-state loads.

3. Representative or actual test data values are used in the design of components and structures.

4. Representative or actual test data are used for any affected component(s) and the minimum Code or specification values for the structures for the dynamic and the steady-state events.

3.6B.2.4 Material to be Submitted for the Operating License Review

3.6B.2.4.1 Implementation of Criteria for Pipe Break and Crack Location and Orientation

<u>Postulated Pipe Breaks in Recirculation Piping System</u>

The criteria for selection of postulated pipe breaks in the recirculation piping system inside containment are provided in Section 3.6B.2.1.

3.6B.2.4.2 Implementation of Special Protection Criteria

<u>Pipe Whip Restraints for Recirculation Piping System</u>

The pipe whip restraint locations for the recirculation piping system are provided to preclude any likelihood of loss of an

essential system or structural integrity of the containment. This system of restraints is provided to prevent unrestrained pipe whip at break locations postulated in accordance with the criteria provided in Section 3.6B.2.1.

3.6B.2.4.3 Summary of Jet Effects Analyses Results

Jet Effects for Postulated Ruptures of Recirculation Piping System

The fluid jet thrust for each of the recirculation piping postulated break locations is calculated in accordance with Section 3.6B.2.2. The jet effects are evaluated in accordance with Section 3.6A.2.3.1 and results are summarized in Appendix 3C.

3.6B.3 References

1. American National Standards Institute (ANSI). ANS-58.2 (ANSI N176), Proposed American National Standard Design Basis for Protection of Light Water Nuclear Power Plants Against Effects of Postulated Pipe Rupture, 1980.

2. GE Report NEDE-10313. PDA - Pipe Dynamic Analysis Program for Pipe Rupture Movement (Proprietary Filing).

3. Nuclear Services Corporation Report No. GEN-02-02, Final Report Pipe Rupture Analysis of Recirculation System for 1969 Standard Plant Design.

4. Shapiro, A. H. The Dynamics and Thermodynamics of Compressible Fluid Flow, Vol. 1. Ronald Press, New York, 1965.

5. Webb, S. W. Evaluation of Subcooled Water Thrust Forces. Nuclear Technology, Vol. 31, October 1976.

6. Hanson, G. H. Subcooled-Blowdown Forces on Reactor System Components: Calculation Method and Experimental Confirmation. Idaho Nuclear Corporation Report IN-1354, June 1970.

TABLE 3.6B-1
(Sheet 1 of 1)

PDA VERIFICATION RESTRAINT DATA

General restraint property for 1 bar of a restraint

$F = C_2 \ (\Delta \ \text{restraint})^N$

Where, Δ restraint^N = δ pipe - total clearance (Figure 3.6B-1)

Pipe Size (in)	Rest Load Direction (deg)	C_2	N	Limit Restraint	Initial Clearance	Effective Clearance	Total Clearance
12	0	27,733	0.24	6.129	4	1.941	5.941
12	90	14,795	0.401	9.063	4	12.247	16.247
16	0	109,265	0.24	6.278	4	1.934	5.934
16	90	62,599	0.377	8.978	4	12.187	16.187
24	0	102,228	0.24	8.222	4	1.984	5.984
24	90	55,531	0.375	11.972	4	13.685	17.685
24	38*	109,888	0.24	5.588	4	5.698	9.698
24	52*	109,835	0.24	5.473	4	8.462	12.462

* Applies to restraint RCR 3 only. See Figure 3.6B-3.

TABLE 3.6B-2
(Sheet 1 of 1)

COMPARISON OF PDA AND NSC CODE

Break ID No.*	Restraint ID No.*	No. Bars		Load (kips)		Restraint Deflection (in)		Percent of Design Restraint Deflection		Pipe Deflection (in)	
		PDA	NSC	PDA	NSC	PDA	NSC	PDA	NSC	PDA	NSC
RC1$_J$	RCR1	5	5	803.2	788.3	6.57	7.926	79.93	96.4	17.72	15.58
RC2$_{LL}$	RCR1	5	5	766.4	458.4	14.99	7.495	125	62.6	35.83	24.52
RC3$_{LL}$	RCR2	6	6	747.0	639.7	2.27	3.73	27.65	45.35	17.16	20.11
RC3$_{LL}$	RCR2	6	6	796.6	780.3	10.22	10.54	57.8	59.6	41.48	43.0
RC4$_{LL}$	RCR3	5	5	846.0	838.4	7.64	8.05	92.95	97.98	18.87	16.43
RC4$_{LL}$	RCR3	8	8	1,319.0	1,073.9	5.43	4.206	99.23	76.85	23.38	17.25
RC4C$_V$	RCR3	8	8	1,260.7	1,275.0	4.49	5.58	80.37	99.89	22.56	18.73
RC6A$_V$	RCR3	8	8	928.5	722.5	1.22	1.77	22.46	31.7	23.68	95.39
RC7$_J$	RCR7	6	6	953.3	801.6	6.28	5.76	76.4	70.12	16.46	21.63
RC8$_{LL}$	RCR6 RCR7	4 6	4 6	599.0 895.0	0 0	8.28 8.16	0 0	112.46 110.76	0 0	26.76 29.316	 8.39
RC9C$_V$	RCR6	4	4	575.8	520.16	4.16	5.53	50.63	67.33	13.2	14.56
RC9$_{LL}$	RCR8	6	6	830.2	546.8	11.408	6.815	95.29	56.9	36.612	26.24
RC11A	RCR8	6	6	818.3	493.6	10.98	5.99	91.72	50.07	31.404	23.71
RC13	RCR10	4	4	668.4	478.0	5.87	3.66	93.5	58.39	13.37	10.44
RC16	RCR11	4	4	687.4	518.4	6.59	4.38	105	69.86	15.37	10.22
RC14C$_V$	RCR20	8	8	285.0	309.6	2.83	5.88	46.3	95.92	15.45	13.96
RC14$_{LL}$	RCR20	8	8	116.3	129.9	0.96	3.36	10.5	37.1	22.13	23.56

* Figure 3.6B-3.

3.7 SEISMIC DESIGN

This section is composed of two parts. Section 3.7A is applicable to the seismic design applied to structures, systems, and components within the SWEC scope of supply. Section 3.7B is applicable to the seismic design of structures, systems, and components within the GE scope of supply.

3.7A SEISMIC DESIGN (SWEC SCOPE OF SUPPLY)

3.7A.1 Seismic Input

3.7A.1.1 Design Response Spectra

The design response spectra are developed in accordance with published procedures[1] and RG 1.60 (Section 3.7A.2.5). In this method, the critical parameter is the maximum expected ground acceleration. Associated with the maximum ground acceleration, the maximum ground displacement is determined by linear scaling from Figures 1 and 2 of RG 1.60 in proportion to the maximum expected ground acceleration. Detailed smooth spectra for any given value of damping are obtained by locating critical points and joining them by straight lines in a tripartite logarithmic plot. These critical points are obtained from given amplification factors and cutoff frequencies. The design value of the maximum ground acceleration is 0.15 g for the SSE and 0.075 g for the OBE. Figures 3.7A-1 and 3.7A-2 show the smooth design response spectra for horizontal and vertical earthquakes associated with the SSE. These design response spectra are not related to any site-dependent ground motion time history.

As shown in Figures 3.7A-1 and 3.7A-2, the ratios of vertical design response spectral values to the horizontal design response spectral values comply with the position of NRC RG 1.60. The ratio varies for different frequencies as required by RG 1.60.

3.7A.1.2 Design Response Spectrum Derivation

An artificial earthquake is generated to give response spectra enveloping the design response spectra. The artificial earthquake is generated and checked by comparing spectral values at 80 periods between 0.02 and 5.0 sec. The periods are spaced according to the rule:

$$T_n = \lambda T_{n-1} \qquad (3.7A\text{-}1)$$

Where:

T_n = Period n in spectrum computation

T_{n-1} = Period (n-1) in spectrum computation

$$\lambda = \left(\frac{T_f}{T_i}\right)\left(\frac{1}{n-1}\right) = 1.0724$$

T_i = Initial period = 0.02 sec

T_f = Final period = 5.0 sec

N = Total number of periods = 80

The acceleration time history yields ground response spectra at damping values of 1, 2, 5, 7, and 10 percent that envelop the smoothed site design ground response spectra (SSE) for damping values as shown on Figures 3.7A-3 through 3.7A-17. The calculated response spectra and design response spectra of RG 1.60 are compared. Based on this comparison, the artificial earthquake is used as the design time history for structural analysis.

Details of the artificial acceleration record and its development are presented in Section 3.7A.2.5.

3.7A.1.3 Critical Damping Values

3.7A.1.3.1 Structures

Seismic analysis is performed using total system damping characterized by modal damping. The modal damping value is calculated as a ratio of the sum of the energy dissipated in each component element (based upon the assigned damping ratio of each element) to the total available modal energy. Further discussion of modal damping appears in Section 3.7A.2.15.

In determining the modal damping ratios, component damping values consistent with the stress intensities are used. For example, component damping for welded structural steel is assigned a value of 2 percent for OBE and 4 percent for SSE.

The damping ratios in RG 1.61 and Table 3.7A-1 for various components are used in the design.

3.7A.1.3.2 Equipment

The percentages of critical damping values assigned to Category I systems and components are in accordance with RG 1.61 and are presented in Table 3.7A-1.

3.7A.1.3.3 Piping

The percentage of critical damping values used for the analysis of all piping are consistent with RG 1.61 and are presented in Table 3.7A-1.
The alternative damping values used for Unit 2 will be those described in ASME Code Case N-411 (Figure 3.7A-34) or as shown on Figure 1 of RG 1.61, Revision 1. The potentially increased piping displacements (resulting from using the alternate damping values) will be verified when piping supports are moved, modified, or eliminated. The verification will ensure that there

will be no adverse interaction with adjacent structures, components, and equipment.

3.7A.1.4 Support Media for Category I Structures

Major Category I structures are founded on sound rock. The top of bedrock is encountered at elevations ranging from 246 to 240 ft. The foundation/support media information for Category I structures is summarized in Table 3.7A-2. Rock properties including wave propagation velocities, densities, and shear modulus can be found in Sections 2.5.2.5 and 2.5.4.2. The static properties of Category I structural backfill are discussed in Section 2.5.4.5. The dynamic properties of Category I structural backfill are presented in Section 2.5.4.8.

3.7A.2 Seismic System Analysis

3.7A.2.1 Seismic Analysis Methods

The structural responses of the reactor building and other Category I structures to the application of horizontal and vertical earthquake ground motions are determined by the response spectrum modal analysis method. Seismic responses for all Category I structures are determined from an application of two orthogonal horizontal and one vertical earthquake ground motions, assumed to act simultaneously. The earthquake ground motions are established in the form of response spectra for the SSE and OBE as described in Section 3.7A.1. The computer program STRUDL (Appendix 3A) is used to obtain the mode shapes, natural frequencies, and responses for the major Category I structures. The response spectra for the floor levels are obtained using the TIMHIS6 computer program (Appendix 3A). The combination of design loading conditions with seismic loading and the allowable stress levels are given in Section 3.8.

The dynamic models of the Category I structures consist of generalized systems of lumped masses, each with 6 deg of freedom, connected by massless, linearly elastic springs. The masses consist of floors, tributary walls, columns, equipment, and piping. The lumped mass models of the structures are shown on Figure 3.7A-18 through 3.7A-20.

Since the major structures are founded on rock, the seismic analyses are performed using fixed bases (Section 3.7A.2.4.1). The secondary containment and primary containment structures are connected only at the foundation; therefore, each structure was modeled separately. Each model is constructed so that it properly represents the free vibration of a cantilevered structure in shear, flexure, and torsion. Generally, mass locations are selected at points with a concentration of mass (e.g., floor elevations), or where there is a special interest in the response (equipment locations).

In the dynamic modeling of Category I structures, the floors are treated as rigid plates or diaphragms that transfer earthquake

inertia forces to frames and diaphragm walls, which in turn transfer the loads to the foundation mat. Beam theory, combining the effects of shear, flexure, torsion, and axial deformation, is used to establish the stiffness characteristics of the frame-wall systems.

The criteria used to determine an adequate number of masses in dynamic modeling of all Category I structures are, in general, dictated by the number of floor elevations and the roof elevations of a structure. It is at these points that masses are lumped and include half the walls above and below the floor, the floor itself, and major pieces of equipment resting on the floor or supported from the walls. This is done because this mass distribution closely approximates the real mass distribution of the actual structure. Additional mass points may also be included where mass distribution dictates.

The criterion for establishing the minimum number of masses is one that provides sufficient accuracy for natural frequencies below 33 cps, since above this value there is little amplification of earthquake excitation. Equipment stiffer than 33 cps responds at the mass response, and larger errors in structural frequencies can be tolerated. The number of masses in a model are considered adequate if the number of degrees of freedom is equal to twice the number of modes with frequencies less than 33 cps. A sufficient number of modes were considered so that the inclusion of any additional mode will not result in a 10-percent increase in responses.

The seismic motion of all Category I structures is determined by applying the earthquake ground motions to the appropriate dynamic models. Where non-Category I structures are attached to Category I structures, the effects are analyzed by including the non-Category I structure in the seismic model of the Category I structure. In general, interaction between Category I and non-Category I structures is eliminated by providing structural gaps and separate foundations for the structures. The structural gaps are designed so that seismic motion between the structures is unimpeded and algebraic summation of maximum relative displacements of adjacent structures under most critical conditions is less than the structural gap.

A tabulation of the structural gaps surrounding Category I structures is shown in Table 3.7A-10. To determine worst computed gaps between the structures, out-of-phase deflection (displacement) of the structures (i.e., structures leaning toward each other) is assumed during a SSE event. Allowable construction tolerance, when added to cumulative displacements, will not exceed structural gap provided. See Figure 1.2-2 for the arrangement of plant structures. As can be seen from the tabulation, the cumulative deflection (displacement) under a SSE event does not exceed the structural gap provided in each case. A minimum of 3 in structural gap is provided between the structures as a common design practice.

For the effects of hydrodynamic loads on the structures, refer to the Design Assessment Report for Hydrodynamic Loads (DAR) (Appendix 6A). For the effects of piping on the seismic analysis of the structures, refer to Section 3.7A.2.3.

3.7A.2.2 Natural Frequencies and Response Loads

A summary of natural frequencies of vibration for Category I structures and the corresponding mode shapes is given in Tables 3.7A-3 through 3.7A-6. Typical modal responses for the primary containment at several selected elevations are given in Tables 3.7A-7 and 3.7A-8. Structural response characteristics in the form of building acceleration profiles for the primary containment, reactor building, control building, and diesel generator building are shown on Figures 3.7A-21 through 3.7A-32.

Amplified response spectra (ARS) are generated for all Category I structures at major Category I equipment elevations and points of support to define the seismic environment for the subsystem analyses (Section 3.7A.2.5).

3.7A.2.3 Procedures Used for Modeling

The dynamic models of Category I structures consist of systems of generalized spring-connected lumped masses. The lumped masses and connecting springs of the dynamic model are determined in order to obtain a satisfactory representation of the dynamic behavior of the actual structure. In general, masses are located at floor elevations and include the floor system, a portion of the walls and columns both above and below the floor system, and major components, equipment, and piping. In addition, masses are located at elevations where response values are required. The structural spring elements that connect mass points represent the stiffness characteristics of the walls and column sections. These characteristics are determined from beam theory for concrete sections, which includes the effects of shear, flexure, and torsion, and from frame analysis for steel frameworks.

In the course of analysis, a comparison of relative mass and stiffness properties between connected components is performed to determine whether coupling effects should be considered in the analysis of the supported components. This consideration is pertinent to all Category I systems and equipment. It is a design goal to decouple (dynamically) equipment from its attached components.

Some basic relationships are used to allow the conclusion of decoupling. Failure to meet any one of these requires that either (1) additional restraints are provided to suitably alter stiffness parameters and thereby dynamically uncouple the system, or (2) the analytical model be formulated to include both the supporting and supported components in order to actually determine the coupling effects of the combined system. The principal purpose for such considerations is to define component interface loads for inclusion in component adequacy

documentation, as well as seismic inputs to supported components. The basic relationships used as a guide to conclude decoupling are:

1. If R_m <0.01, decoupling is acceptable for any R_f.

2. If 0.01 $\leq R_m \leq$0.1, decoupling is acceptable if 0.8 $\geq R_f$ $\geq$1.25.

3. If R_m >0.1, an approximate model of the subsystem is included in the primary system model.

Where:

$$R_m = \frac{\text{Total mass of the supported subsystem}}{\text{Total mass of the supporting system}}$$

$$R_f = \frac{\text{Fundamental frequency of the supported subsystem}}{\text{Dominant frequency of the support motion}}$$

If the subsystem is comparatively rigid and also rigidly connected with the primary system, only the mass of the subsystem is included at the support point in the primary system model. On the other hand, in the case of a subsystem supported by very flexible connections (e.g., pipe supported by hangers), the system is not included in the primary model. In most cases, the equipment and components, which come under the definition of subsystems, are analyzed as a decoupled system from the primary structure, and the seismic input for the subsystem is obtained by the analysis of the primary system.

3.7A.2.4 Soil/Structure Interaction

All Category I structures are supported on natural bedrock except that electrical ductline 907, part of ductline 922, and electrical manhole no. 1 are supported by Category I structural fill (Table 3.7A-2). Certain portions of the floor slabs in the diesel generator building beneath which granular fill was placed are shown on Figure 3.7A-33. This fill is used solely as a construction form and, as shown on Figure 3.7A-33, was placed in limited areas. The static and dynamic design of these slabs assumed no load transfer to, or bearing support from, the underlying backfill. Wall design, however, considered lateral soil pressures as shown on Figure 2.5-110. The electrical ductline is a continuously supported underground structure consisting of conduit totally encased in reinforced concrete. The electrical ductlines are evaluated for relative motion between the portions of duct supported on rock and structural fill using ASCE procedures[5]. The electrical ductlines are capable of withstanding the stresses and strains induced by the relative motion between the portions of duct supported on rock and structural fill. Manhole no. 1 is designed for seismic loads using the procedures described in Section 3.7A.3.12 for Category I tunnels. See Figure 3.7A-33 for details of the manhole and ductline.

All Category I structural fill provides a factor of safety against liquefaction as listed in Section 2.5.4.8. For static properties of the Category I fill see Section 2.5.4.5.

3.7A.2.4.1 Rock/Structure Interaction

Dynamic analyses for structures founded on rock are performed using fixed base models because shear wave velocity exceeds 3,500 fps. The details of the geophysical survey are discussed in Sections 2.5.4.2 and 2.5.4.4.

3.7A.2.5 Development of Floor Response Spectra

ARS are defined as plots of the maximum response versus period for single degree-of-freedom systems at various locations in structures subjected to dynamic loading. In the analysis of equipment with masses that are small compared to the masses of the dynamic model of the supporting structure, the response of the structure is independent of the response of the equipment. The problem can then be solved in two parts: the response of the structure due to ground acceleration can be determined, and that response can be applied as support accelerations to the equipment. In such cases, the use of ARS is an acceptable approach to the problem of determining the dynamic loads on equipment. The time history method of analysis, using the TIMHIS6 computer program (Appendix 3A), is used to generate the ARS for Category I piping and equipment.

The undamped equations of motion for an n degree-of-freedom structural model are solved to determine modal eigenvalues, eigenvectors, and participation factors. The modal equation of motion for structural response in mode i can be written:

$$\ddot{X}_i(t) + 2 D_i \omega_i \dot{X}_i(t) + \omega_i^2 X_i(t) = \Gamma_i \ddot{o}_g(t)$$

(3.7A-2)

Where:

$X_i(t)$ = Time-dependent modal amplitude

D_i = Modal damping

ω_i = Modal circular frequency

Γ_i = Modal participation factor

$\ddot{U}_g(t)$ = Ground acceleration time history

Equation 3.7A-2 can be solved numerically for $X_i(t)$. The solution for the structural response in mode i is then:

$$[U_i(t)] = [a_i]\, X_i(t)$$

(3.7A-3)

and:

$$[\ddot{o}_i(t)] = [a_i]\, \ddot{X}_i(t)$$

(3.7A-4)

Where:

$[U_i(t)]$ = Time-dependent displacement vector in mode i for the n degree-of-freedom system

$[\ddot{U}_i(t)]$ = Time-dependent acceleration vector in mode i for the n degree-of-freedom system

$[a_i]$ = Eigenvector for mode i

The significant structural responses may be added numerically to obtain the time history of acceleration which may be applied to the supports of damped single degree-of freedom systems. The maximum values of response of the single degree-of-freedom systems produce the ARS. ARS are developed for two horizontal and one vertical excitation.

The ground acceleration $\ddot{U}_g(t)$ is an artificial time history with a total duration of 15 sec whose ground response spectrum envelops the response spectrum as specified in RG 1.60. An artificial accelerogram of ground excitation that reproduces the frequency content displayed in a response spectrum is simulated statistically in a digital computer by using a multi-P stochastic model. In this model, the earthquake motion is considered to be a wide-band stationary process whose spectral density function, duration, and maximum acceleration are specified.

The artificial motion is generated by matching the target spectrum for several specified percentages of critical damping at 80 oscillator periods distributed logarithmically from 0.02 sec (50 Hz) to 5.0 sec (0.2 Hz). These statistically independent orthogonal ground accelerations (two horizontals and one vertical) are applied simultaneously to the support. The particular response in a direction of interest is obtained by algebraic summation of the response in that direction at each time interval due to each of the three ground accelerations.

3.7A.2.6 Three Components of Earthquake Motion

The spatial components from seismic response analysis are combined in accordance with RG 1.92. All Category I structures are analyzed from the three orthogonal component motions (two horizontal and one vertical) of the prescribed earthquake. When

response spectrum analysis is performed, the representative maximum value of a particular response of interest for design (e.g., stress, strain, moment, shear, or displacement) of a given element of a structure, system, or component subjected to a simultaneous action of the three components of the earthquake is obtained by taking the square root of the sum of the squares (SRSS) of corresponding representative maximum values of the spectrum response to each of the three components calculated independently.

In cases where time history dynamic analysis is used, three statistically independent (maximum correlation factor of 0.2) orthogonal ground accelerations (two horizontal and one vertical) of the prescribed earthquake are input simultaneously. The particular response in a direction of interest is obtained by algebraic summation of the response in that direction at each time interval due to each of the three ground accelerations.

3.7A.2.7 Combination of Modal Responses

In the response spectrum modal analyses, the modal responses are combined using the grouping method or double sum method as described in RG 1.92.

3.7A.2.8 Interaction of Non-Category I Structures with Category I Structures

When Category I and non-Category I structures are integrally connected, the non-Category I structures are included in the model when determining the response of the Category I structures.

All non-Category I structures meet one of the following requirements:

1. The collapse of a non-Category I structure will not cause the non-Category I structure to strike a Category I structure or component.

2. The collapse of a non-Category I structure will not impair the integrity of Category I structures or components.

3. The non-Category I structure is analyzed and designed to prevent its failure under SSE conditions in such a manner that the margin of safety of the structure is equivalent to that of the Category I structures.

3.7A.2.9 Effects of Parameter Variations on Floor Response Spectra

The effects on the calculated value of fundamental structural periods due to expected variations in damping and the structural material properties are taken into account. SWEC-supplied Category I equipment and piping systems designed using floor response spectra and having natural periods within ±15 percent

of the peak resonant period(s) are assigned the peak response value. Outside this range, the broadened peak is bounded on each side by lines that are parallel to lines forming the original spectrum peak. Damping values are assigned to systems as outlined in Section 3.7A.1.3.

In addition to the broadening of peaks, two dynamic models were used in the seismic analysis of the primary containment. This was done to account for the variation in the stiffness of the primary containment members that occur during a LOCA. For one model, the concrete containment is assumed to be completely uncracked, while the other model utilizes cracked section properties for the primary containment wall.

Since two different models are used, two sets of response spectra in each direction are generated at each floor level. The design floor response spectra are obtained from an envelope of the two floor spectra for each direction of excitation. The response spectrum analysis is also performed on each model. The maximum value of a particular response of interest (e.g., moment, shear, axial force, displacement) is used for the design of the primary containment structure.

3.7A.2.10 Use of Constant Vertical Static Factors

Vertical seismic system multimass dynamic models are used to obtain vertical response loads for the seismic design of Category I structures. Therefore, constant vertical load factors are not used to account for vertical response to earthquakes of Category I structures.

3.7A.2.11 Method Used to Account for Torsional Effects

Category I structures may have natural torsional modes of vibration due to eccentricities between the centers of rigidity and centers of mass of the structural elements. The presence of eccentricities generates coupling between translational directions of motion, resulting in torsion. Thus, general three-dimensional models are set up, followed by complete dynamic analyses as described previously. The results of these analyses include torsional effects.

Since the three-dimensional model accounts for the torsional effects, including the effects of eccentricities between the centers of rigidity and centers of mass of the structural components, the additional eccentricity of 5 percent of the maximum building dimension is not considered in the analyses. Additionally, the design of the control building, which is considered representative of Category I structures, was reviewed for an additional torsional moment resulting from additional eccentricity of 5 percent of the maximum building dimension. It was shown that the additional shear stresses resulting from this analysis were not significant and were within the design capacities.

3.7A.2.12 Comparison of Responses

A typical comparison of structural response obtained by response spectrum and time history analyses for selected points throughout the plant is given in Table 3.7A-9.

3.7A.2.13 Methods for Seismic Analysis of Dams

There are no dams that will impact Unit 2.

3.7A.2.14 Determination of Category I Structure Overturning Moments

The overturning moments induced by seismic loading are computed by the spectrum method of analysis (Section 3.7A.2.1.1) for each direction of excitation separately. The applied overturning moment is computed from the SRSS of maximum responses from three individual directions of excitation.

3.7A.2.15 Analysis Procedure for Damping

In order to use modal analysis, damping values in different elements of a coupled system are accounted for by using stiffness as a weighting function in generating the modal damping values[2]. According to this method, for a system vibrating in its ith mode, the ith modal damping can be estimated by evaluating the ratio of the total energy dissipated due to the presence of damping in different elements of the system to the total strain energy stored in the system in its ith mode. The limiting values of damping factors are given in Table 3.7A-1 for various systems and components.

Two types of damping are generally recognized: (1) viscous, in which the energy dissipated per cycle is proportional to frequency, and (2) hysteretic, in which no frequency dependence is seen. Most structural elements display hysteretic behavior, while supporting soils appear to combine both hysteretic and viscous damping mechanisms. Since the dynamic analysis is performed using a fixed base model, only hysteretic damping need be considered. Whitman's Seismic Design for Nuclear Power Plants[3] gives a useful approximation for the damping of each mode when material damping varies from element to element. This expression for the (equivalent viscous) modal damping is obtained by a strain-energy weighting of element damping:

$$B^{j}_{eqv} = \frac{\sum_{i=1}^{N} D_i E^{j}_{i}}{\sum_{i=1}^{N} E^{j}_{i}} \tag{3.7A-5}$$

Where:

N = Number of elements

D_i = Hysteretic damping ratio for element i

E_i^j = Strain energy in element i when deflected into mode shape j

B_{eqv}^j = Equivalent viscous damping ratio (fraction of critical) for structure vibrating in mode j

In particular, when damping is uniform, i.e., $D_i = D$, then:

B_{eqv}^j = D for all modes

When damping is not uniform, modal damping is weighted toward those elements that make the largest contribution to the energy of each mode.

3.7A.3 Seismic Subsystem Analysis

The design of Category I subsystems (i.e., components, equipment, piping, supports) includes OBE and SSE seismic loading conditions.

The SSE produces the maximum vibratory ground motion for which Category I systems and components are designed to remain functional. These systems and components are those necessary to ensure:

1. The integrity of the RCPB.

2. The capability to shut down the reactor and maintain it in a safe shutdown condition.

3. The capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10CFR100.

The OBE produces the vibratory ground motion for which those features of the nuclear power plant that are necessary for continued operation without undue risk to the health and safety of the public are designed to remain functional. System seismic classification is provided in Table 3.2-1.

3.7A.3.1 Seismic Analysis Methods

3.7A.3.1.1 Seismic Qualification of Components

This section provides the qualification methods for equipment affected by seismic loads. The methods for the qualification of equipment affected by hydrodynamic loads associated with SRV

discharge and the postulated LOCA are provided in the DAR, Appendix 6A, Subsection 6A.9.

All Category I equipment is qualified for seismic adequacy. Depending upon equipment location, the basic source of seismic design data is either the ground response spectra or the ARS, derived through a dynamic analysis of the structure. The four principal methods of documenting adequacy for Category I components are static analysis, dynamic analysis, dynamic testing, and static deflection testing. These methods are used singly or in combination to qualify equipment.

Static Analysis

Static analysis is used for equipment that can be modeled as relatively simple structures. This type of analysis involves the multiplication of the component weights by the specified seismic accelerations (direction-dependent loadings) to produce forces that are applied at the centers of gravity in the horizontal and vertical directions. A stress analysis of critical items, such as support points, hold-down bolts, and other structural members, is performed to determine their adequacy. The deflections of critical components are also calculated and compared with specified tolerances.

In the specification of equipment for static analysis, two ranges of acceleration data are provided: a resonant range distinguished by lower frequencies with amplified response accelerations, and a rigid range characterized by higher frequencies and essentially nonamplified response. The division between the two ranges is termed the cutoff frequency. Selection of the appropriate range depends upon the fundamental natural frequency of the equipment. If this value is beyond the resonant range (i.e., higher than the cutoff frequency), the equipment is analyzed to rigid range response accelerations.

Equipment having a fundamental frequency in the resonant range of the ARS is analyzed by using the peak resonant acceleration, increased by a static coefficient of 1.3. This factor accounts for potential multimode response (Section 3.7A.3.5).

Each of the three defined directions of earthquake input (two horizontal and one vertical taken orthogonally) are evaluated separately. The calculated results of the three analyses are superimposed using the SRSS criterion. The particular response values (e.g., acceleration, force, stress) to be combined are optional, but the option selected remains consistent throughout, following the guidelines of RG 1.92.

Dynamic Analysis

A detailed dynamic analysis is performed when component complexity or dynamic interaction precludes static analysis, or when static analysis is too conservative. An infinite number of coordinates would be required to fully describe the behavior of a

component subjected to dynamic loads. Since calculation at every point of a complex model is impractical, the analysis is simplified by the selection of a limited number of mass points. The lumped mass approach is employed in which the main structure is represented in a model with masses interconnected by flexible elements. The nature of the component and the stiffness properties of the corresponding modeling elements determine the minimum spacing of the mass points and the degrees of freedom associated with each point. In cases where some dynamic degrees of freedom do not contribute to the total response, static or kinematic condensation is employed in the analysis.

The normal mode approach is employed for dynamic analysis of components. Natural frequencies, eigenvectors, participation factors, and the required component dynamic responses, such as modal member-end forces and moments of the undamped structure, are calculated. The basis for combination of modal responses is discussed in Section 3.7A.2.7. Documented computer programs in the public domain are used for performing dynamic analysis. However, if proprietary computer programs are used, qualification of the programs is required.

Each of the three defined directions of earthquake input (two horizontal and one vertical taken orthogonally) are evaluated separately. The calculated results of the three analyses are combined by using the SRSS method. The selected response values (e.g., acceleration, force, stress) are combined following the guidelines of RG 1.92.

Testing

Equipment that is too complex to analyze or whose operability cannot be adequately demonstrated by analysis is qualified by dynamic testing. The equipment specification testing requirements supplement the testing methods and acceptance criteria of applicable industry standards (such as IEEE-344-1975, Section 3.10), or provide guidance for testing where no such codes are available.

The minimum acceptance criteria for equipment adequacy are:

1. No loss of function, or ability to function, before, during, or directly after completion of the proposed test.

2. No structural/electrical failure (i.e., connections and anchorages) that would compromise component integrity.

3. No adverse operation or faulty operation before, during, or after completion of the test that could result in an improper safety action.

Equipment vendors and suppliers are required to formulate programs for qualifying the equipment in accordance with the specified seismic requirements.

The base motions used to simulate the seismic loadings consist of either a single frequency or multiple frequencies and are applied either along one axis or along horizontal and vertical axes simultaneously. The choice of the input motion, i.e., frequency and axis, depends on the dynamic characteristics of the equipment and on the frequency content of the seismic loading. The criteria for selecting these specific input test motions are in accordance with IEEE-344-1975 and RG 1.100.

Exploratory tests are run to determine the response characteristics of the equipment and to aid in selecting the method of testing. The exploratory test consists of a low-level sinusoidal sweep over the frequency range of seismic loading (1 to 33 Hz). The sweep rate is 2 octaves/min or lower to excite all the resonances. If the equipment is shown to be nonresonant in the frequency range of seismic loading, it is considered a rigid body and tested accordingly. If the equipment exhibits multiple resonant response, further testing programs, based on multifrequency input, are considered more appropriate and are used in qualifying the equipment.

<u>Multifrequency Testing</u> Multifrequency input, applied biaxially, is the preferred method used for seismic qualification. Other methods are used as justified. Input motion for testing is applied to the vertical and one of the two principal horizontal axes simultaneously, unless it is demonstrated that the equipment response along the vertical direction is not sensitive (coupled) to the vibratory motion along the horizontal direction and vice versa. Phase-incoherent (i.e., statistically independent) inputs in the vertical and horizontal directions are use to avoid purely rectilinear motion. When the test facility limitations do not allow the use of independent inputs, two tests are performed: (1) vertical and horizontal inputs in-phase, and (2) vertical and horizontal inputs 180 deg out-of-phase. This test is repeated with the equipment rotated 90 deg in the horizontal plane. The test setup simulates as closely as possible the actual in-service installation. Equipment is tested in the mode (energized or de-energized) that reflects its design safety function. Equipment operability is verified by performing the appropriate functional cycle during and after the dynamic tests.

The basic objective of qualification or proof testing is to produce a test response spectrum (TRS) that envelops the required response spectrum (RRS). ARS, when properly broadened to account for variations in structural properties, become the RRS for qualification.

For the multifrequency input applied, the testing machine input must, as a minimum, equal the maximum floor acceleration of the RRS. The TRS is adjusted in successive test runs so that it envelops the RRS over the required frequency range. Curves for identical damping are used in comparing TRS and RRS information. Five OBE-level tests are performed prior to SSE qualification testing following the recommendations of IEEE-344-1975.

Multifrequency testing provides broad band test input motion which produces simultaneous response from all the modes of the equipment. Multifrequency motions are derived using any of the following techniques.

1. Time History An acceleration motion in the time domain, at the equipment mounting location, obtained from dynamic analysis of the structure.

2. Random Motion An electrically generated random noise signal that is selectively amplified, or attenuated, in one-third or smaller frequency band-widths. The motion resulting from this modified signal is arranged so that it envelops the TRS. This is the most commonly used input motion for multifrequency testing. The peak acceleration amplitude of this motion equals or exceeds the zero-period acceleration (ZPA) of the RRS. The random motion signal is applied for a minimum duration of 15 sec.

3. Complex Wave A complex wave is a sum of a group of decaying sinusoidal signals spaced at 1/3 octave or narrower frequency intervals over the frequency range of the RRS.

Single Frequency Testing Following the recommendations in IEEE-344-1975, single frequency input for testing is used only if one of the following conditions is met:

1. The characteristics of the required input motion indicate that the motion is dominated by one frequency (i.e., by structural filtering effects).
2. The anticipated response of the equipment is adequately represented by one mode.

The objective is to produce a TRS acceleration at the test frequency at least equal to that given by the RRS. The test table input equals or exceeds the maximum floor acceleration of the RRS. The single-frequency test consists of an exploratory test and a dwell test. In the exploratory test, the table input motion equals or exceeds the maximum floor acceleration, i.e., the ZPA of the RRS. Dwell testing is performed at the natural frequency identified during the exploratory test. The dwell test consists of applying a continuous sinusoidal input motion at the maximum floor acceleration for a minimum duration of 15 sec. Dwell testing is also performed using a sine beat input instead of a continuous sine input. A sine beat consists of a continuous sinusoid at the test frequency, amplitude modulated by a sinusoid of a lower frequency. The duration and peak amplitude of the beat for each particular test frequency are chosen to generate a magnitude of equipment response that is at least equal to that imposed by the RRS at the appropriate damping level. As a minimum, the peak amplitude of the beat should equal the rigid

range acceleration of the RRS. Ten cycles per beat are used, following the recommendations of IEEE-344-1975.

Static Deflection Testing

A static deflection test consists of applying a sustained static load on critical sections of the component in such a way that the deflection caused by this load duplicates or exceeds the calculated SSE deflection. Concurrently, the component is operated in the required manner, and all applicable design loads are superimposed during the test.

3.7A.3.1.2 Seismic Qualification of Piping Systems

This is described in Section 3.7A.3.8.

3.7A.3.1.3 Other Dynamic Loads

Loading combinations and stress limits, including loads due to hydrodynamic effects, are described in Section 3.9A.3.1. A further discussion of hydrodynamic phenomena is included in the DAR (Appendix 6A).

3.7A.3.2 Determination of Number of Equivalent Stress Cycles

The following criteria are applied to all Category I subsystems:

1. A total of five OBE and one SSE are considered.

2. For subsystems, except piping, 20 cycles (full sign reversals) per seismic event, i.e., a total of 120 cycles, are considered.

3. For all piping systems, 10 maximum stress cycles per OBE (i.e., a total of 50 cycles) are postulated.

4. Where time history analysis is performed, a minimum duration of 10 sec is assumed.

3.7A.3.3 Procedure Used for Modeling

The procedure described in the following sections is specifically written for piping systems. Other subsystems are seismically qualified as described in Section 3.7A.3.1.1.

3.7A.3.3.1 Summary

Portions of piping systems that are bounded by anchors or equipment are statically and dynamically independent from the remainder of the piping. Generally, a piping system consists of several such subsystems. The analytical model and its geometric boundaries are described in detail in the following sections.

3.7A.3.3.2 Geometric Boundaries of Analytical Models

For the purpose of analysis, piping systems are subdivided into smaller units (referred to as problems) that are bounded by structural anchors (6 degree-of-freedom constraints) or by other virtually rigid points such as equipment, penetrations, and piping of much larger diameter.

A branch line with a moment of inertia of 1/10 or less of the run pipe may be ignored in the model. However, if the branch line needs to be analyzed, its model includes the effect of the run pipe. Where Category I piping is connected to nonseismic piping, the adjoining portion of the nonseismic piping up to the first anchor is included in the analytical model of the Category I piping, and all supports, up to and including this anchor, are designed seismically (Section 3.7A.3.13).

3.7A.3.3.3 Model

The basic method of analysis used is a finite element computer program (Appendix 3A). In accordance with this method, the continuous piping is mathematically idealized as an assembly of elastic structural members connecting discrete nodal points. Nodal points are placed in such a manner as to isolate particular types of piping elements such as straight runs of pipe, elbows, valves, etc., for which force-deformation characteristics can be categorized. Nodal points are also placed at all discontinuities such as piping supports, concentrated weights, branch lines, and changes in cross section. System loads such as weights, equivalent thermal forces, fluid transient dynamic forces, and inertia forces are applied at the nodal points. Stiffness characteristics of the interconnecting members are related to the effective shear area and moment of inertia of the pipe. The stiffness of piping elbows and certain branch connectors is modified to account for local deformation effects by the flexibility factors specified in ASME Section III, 1974, Subarticles NB-3600 (Safety Class 1 piping analysis) and NC-3600 (Safety Class 2 and 3 piping analysis). The increased stiffness of valve bodies is taken into consideration.

3.7A.3.3.4 Selection of Mass Points

The lumped masses are located to adequately represent the dynamic properties of the piping system. Mass points are generally selected in accordance with the following guidelines:

1. At each node where a concentrated weight is placed (valves, flanges, or other in-line piping components).

2. At each intersection where three or more piping elements are connected (branch connections, tees, and y-fittings).

3. At the end of elbows and turn of direction.

4. At nodes subjected to input of dynamic force excitation.

5. At each terminal (node where only one element is connected such as end caps, valve operators).

6. At least one mass point between two restraints acting in the same direction.

7. Unless there is a concentrated weight, a lumped mass point should not be placed at a dynamic restraint.

When these guidelines are used the number of degrees of freedom in the dynamic model is greater than twice the number of modes with frequencies less than 33 Hz.

3.7A.3.3.5 Number of Modes and Cutoff Frequency

The number of modes used in the dynamic analysis of piping depends upon the number of mass points, dynamic degrees of freedom, and cutoff frequency.

The cutoff frequencies used for different dynamic loads are as follows:

Load Type	Cutoff Frequency (Hz)
Seismic	33
SRV	100
LOCA	100
Hydraulic transients	≥100

3.7A.3.4 Basis for Selection of Frequencies

3.7A.3.4.1 Components

ARS developed for the two orthogonal and vertical direction earthquakes are the basic source of seismic design accelerations. Seismic accelerations are selected from the ARS based on the natural frequency calculations of the components with proper consideration of the frequency characteristics of the component supports. Appropriate amplification factors are included in the seismic loads to insure the adequacy of the design of the components.

3.7A.3.4.2 Piping

Piping systems are generally supported in such a way that the lowest natural frequency of analytical subsystems (piping bounded by components and/or structural anchors) does not occur in the peak range of the applicable ARS. For small size Category I piping, subsystems are supported as outlined in Section 3.7A.3.8.2. The lowest natural frequency for this piping is above the applicable spectrum peak range.

3.7A.3.5 Use of Equivalent Static Load Method of Analysis

Those components that are considered relatively simple or rigid are designed, by virtue of natural frequency calculations, to withstand the effects of amplified seismic acceleration values dependent upon frequency and amplitude ranges associated with the relevant ARS. Analysis of components to the peak value of resonant response is considered conservative, since fundamental natural frequencies do not generally coincide with the frequency at resonance of the relevant response curve. Components having fundamental natural frequencies less than the cutoff frequency (Section 3.7A.3.1.1) are designed to peak acceleration values, increased by a factor of 1.3, or as justified, to account for the contribution of all significant dynamic modes under a resonant condition. Justification for the use of 1.3 as a static coefficient can be found in ASME Paper 74-NE-6[4].

The validity of the use of the 1.3 static coefficient factor is evaluated for equipment where a large number of modes of vibration within the seismic load frequency range are anticipated.

An example justifying the use of a 1.3 factor is demonstrated through a comparison of results obtained for a typical equipment. The equipment selected is an instrument panel, 2DFM-PNL102, for Unit 2. This panel was analyzed by static analysis using the 1.3 static coefficient, and also by a response spectrum modal analysis. The input response spectra used in the modal analysis were modified from the actual Unit 2 spectra, as shown on Figure 3.7A-36 (Sheets 1 and 2). This yields conservative results and allows a wide applicability of the conclusion.

Static Analysis

Reference: Technical Report No. 18609-83N-2, Specification NMP2-C062G, Acton Environmental Testing Corporation/Electro-Mechanics, Inc.

Fundamental Natural Frequency = 18.8 Hz

This frequency is less than 33 Hz; therefore, the static analysis was performed using the following accelerations:

X-1 Direction (Horizontal) = 5.51g (4.24g x 1.3)
X-2 Direction (Horizontal) = 5.51g (4.24g x 1.3)
X-3 Direction (Vertical) = 4.10g (3.15g x 1.3)

Dynamic Analysis

The response spectra modal analysis was performed using the spectra of Figure 3.7A-36 (Sheets 1 and 2). All the modes up to a frequency of 70 Hz were included in this analysis.

Results and Conclusion

The maximum responses (displacements, stresses) from the two analyses and a ratio of these responses are presented in Table

3.7A-13. The margins evident from the ratio of these responses conclude the adequacy of the 1.3 static coefficient.

3.7A.3.6 Three Components of Earthquake Motion

The maximum structural responses (displacements, acceleration, forces, and moments) due to each of the three components of earthquake motion are combined by taking the SRSS of the maximum codirectional responses, caused by each of the three components of earthquake motion at a particular point of the structure or of the mathematical model. This is in conformance with RG 1.92.

3.7A.3.7 Combination of Modal Responses

The basis for computing combined response for use in subsystem analysis is presented in Section 3.7A.2.7.

3.7A.3.8 Analytical Procedures for Piping

3.7A.3.8.1 Introduction

Piping classified as Category I is designed to withstand levels of loading imposed by the OBE and the SSE. The piping systems are classified as:

1. Those governed by the ASME Code as Safety Class 1, 2, or 3 piping.

2. Those governed by the ANSI B31.1 Code and requiring seismic analysis.

The seismic response of piping systems is analyzed by the response spectrum method or the time history method. The response spectrum method requires that seismic loading be combined from the dynamic response of the system based on an ARS, and from the response to a quasi-static differential support movement, also called seismic anchor movement, which represents the out-of-phase movement of portions of the structure to which the system is attached. Computer analysis considers all vibration modes up to at least the mode beyond which the contribution to the overall seismic dynamic response is insignificant.

The structural damping is the same for all modes of the piping system and varies only with pipe size (Section 3.7A.3.15). A response spectrum curve contains a certain damping value implicitly. In time history analysis, the damping value is an input parameter to the analysis. The number of earthquake cycles needed for fatigue analysis is given in Section 3.7A.3.2. Pipe stress analysis classifications are given in Table 3.9A-3.

All safety-related piping systems that have been seismically analyzed are reviewed to verify that engineering input information and as-installed configurations are consistent with the design requirements required by IE Bulletin 79-14. The process that governs this is a part of the design verification

program for all Category I piping. The review consists of two parts: one that examines design inputs such as ARS and anchor motion, and one that compares as-built drawings against the as-analyzed calculations of record.

For large bore piping, the large bore piping as-built drawings are developed based on the installation control drawing for piping. The large bore piping as-built drawings are marked up and checked by an independent organization to show the as-installed configuration in accordance with requirements. For small bore piping, the as-built drawings are the piping design drawings marked up to show the as-installed configuration in accordance with the specification requirements. For both large and small bore pipe supports, the as-built drawings are the engineering pipe support design drawings and associated change documents which have been verified in accordance with the specification requirements. In all cases, the information is compiled by groups responsible for the final analysis where as-built, as-analyzed comparisons are performed. Either the differences in configuration or input information are justified on a case-by-case basis or the necessary changes are issued to the field. The engineering small bore piping design drawings and large bore piping as-built drawings are revised to incorporate as-built information.

The design attributes that are reviewed and the source documents that provide these attributes are provided in Table 3.7A-11 for large bore piping and Table 3.7A-12 for small bore piping. A list of applicable safety-related piping systems is provided in Table 3.2-1. Load combinations and stress criteria are described in Section 3.9A.1.5.

The final documentation of this program occurs at the time of N-5 signoff, when a review is conducted to ensure that all input information is still valid and that any revisions that have taken place do not change the basis for the final analysis.

3.7A.3.8.2 Analytical Techniques

General Criteria

Piping systems are rigidly supported, where possible, to assure a first mode natural frequency above the peak frequency after peak spreading.

Qualification of Small Size Piping

The scope of small size piping is limited to:

1. ASME Safety Class 1 piping of 1-in NPS and smaller, which can be analyzed by Safety Class 2 rules in accordance with Subsubarticle NB-3630.

2. ASME Safety Class 2 and 3 piping of 6-in NPS and smaller.

3. ANSI B31.1 piping (Safety Class 4 piping) of 6-in NPS and smaller.

In general, the analysis of small-size ASME Safety Class 1 piping (1-in NPS and smaller) and ASME Safety Class 2 and 3 piping (6-in NPS and smaller) is performed by means of simplified seismic analysis without computer application.

For ASME Safety Class 1 piping (1-in NPS and smaller) and ASME Safety Class 2 and 3 piping (2-in NPS and smaller), the maximum support spans are determined by limiting the stresses to within the code allowables.

For ASME Safety Class 2 and 3 piping with 2 1/2- to 6-in NPS, the support spacing is selected so that the fundamental frequency (fp) of the piping section will always be beyond the resonant frequency of the structure, as determined from applicable seismic ARS. The peak of the floor response spectra for design of piping supported between two points is used for the simplified seismic analysis. Deadload and thermal responses are also calculated in accordance with Subarticle NC-3600.

The simplified analytical approach is to perform stress calculations for consecutive sections of piping, bounded at support points, without using computer application. This is justifiable because a rigid system with sufficient pipe supports represents many one-dimensional, straight-beam problems wherein the coupling effects of the three-dimensional piping systems are eliminated. Constraints are placed near elbows, tees, and concentrated masses, such as valves, so that coupling effects are negligible. These calculations of maximum combined stresses provide sufficient and conservative data to satisfy the requirement of Subarticle NC-3600.

3.7A.3.8.3 Dynamic Analysis

<u>Model</u>

The modeling procedure, including the selection of mass points and the adequacy of number of degrees of freedom, is described in Section 3.7A.3.3.

<u>Response Spectrum Method</u>

When a piping system is analyzed by means of the response spectrum method, one of the piping analysis computer programs described in Appendix 3A is used to calculate the modal response at each node point in the piping system due to the ARS excitation applied to the system. Generation or selection of the appropriate set of ARS for a subsystem supported at different elevations, and consideration of the effect of seismic differential displacements between restraints, are discussed in Section 3.7A.3.9. The damping values for piping depend on pipe size and are given in Table 3.7A-1. The equations of motion and

their solution are as previously described for structures (Section 3.7.2).

Time History Method

The applicable base motion time history is the structural response at a representative mass point of the structure to the ground motion time history. The equations of motion and their solution are the same as in Section 3.7A.2.5, but the scalar acceleration term in the excitation function is now the amplitude of the acceleration of the base of the subsystem (points of attachment), not of the ground. The effect of parameter variations on the floor response spectra are taken into account (Section 3.7A.2.9).

Dynamic Analysis Formulation

The basic equations of motion and their solutions are the same as for structures (Section 3.7.2). Absolute accelerations at points on the piping system are sometimes needed for qualification of critical, safety-related equipment. With the response spectrum method, the maximum absolute acceleration at a mass point in mode i is obtained from Newton's law by dividing the effective inertia force by the mass at the mass point:

$$\{a\} = [M]^{-1} \{Q\} \qquad (3.7A\text{-}6)$$

Where:

[M] = Diagonal mass matrix of the system

{Q} = Effective inertia forces in mode i

With the time history method, the absolute accelerations are obtained by adding the base acceleration to the relative accelerations of the mass points.

Seismic Differential Displacements

Description of Input The seismic differential displacements are also called seismic anchor movements. This effect is analyzed in a separate static load case for OBE anchor movements. The anchor movements are obtained from the seismic differential displacements of the structural nodes.

The displacements are obtained in the following form, one set for each mass point, N, of the building model:

	Earthquake Direction		
Mass Node	X (East-West)	Y (Vertical)	Z (North-South)
1	D_1	D_1	D_1

	X	Y	Z
2	D_{2_X}	D_{2_Y}	D_{2_Z}
3	D_{3_X}	D_{3_Y}	D_{3_Z}
	.	.	.
	.	.	.
	.	.	.
N	D_{N_X}	D_{N_Y}	D_{N_Z}

Where:

$D_{N_X}(D_{N_Y}, D_{N_Z})$ = SRSS of displacements in X (Y, Z) direction due to earthquake excitation in X, Y, and Z directions at Node N

These are the movements of points on the walls relative to the foundation of the building. Relative displacements between mass points are used to determine the movements of support points. Support displacements are imposed on the system in a conservative manner.

Combination of Anchor Movement Loads The individual X, Y, and Z anchor movement components of OBE are analyzed as three separate static anchor movement load cases. These load cases are then combined by the SRSS method and the resultant load case is used in the code stress evaluation. Anchor movement load cases are analyzed using one of the piping analysis computer programs described in Appendix 3A.

Combined Seismic Response

The system response to the response spectrum excitation (i.e., displacements, internal forces and moments, stresses, and support reactions) is obtained by first combining the modal contributions for each earthquake component. In conformance with RG 1.92, the effect of closely-spaced modes is taken into account by the procedure described in Section 3.7A.2.7. The contributions of each of the three components are then combined by the SRSS methods.

When the response spectrum method is used, response to the differential support motion is considered. In Safety Class 1 piping analysis, this motion is combined with the inertial response; the result is then combined with other load cases. In the analysis of other piping classes, the seismic anchor movement is combined with secondary loads. Seismic load cases are combined with other load cases (thermal, weight, pressure, other occasional loads) in accordance with ASME Section III, 1974. The load combinations are given in Section 3.9A.3.1.

Fatigue Considerations

For ASME Safety Class 1 piping, if Equation (10) of Subparagraph NB-3653.1 is not satisfied, a fatigue analysis is performed in accordance with Subparagraph NB-3653.2 or Subarticle NB-3200. This analysis uses the total number of stress cycles of all OBEs. The number of earthquake cycles is discussed in Section 3.7A.3.2.

Computer Programs Used for Seismic Analysis

All analyses are performed using one of the piping analysis computer programs described in Appendix 3A. These programs handle response spectrum and support motion time-history analyses. Programs used for generating ARS curves for piping analysis are also described in Appendix 3A.

Development of Relative Displacements and Their Application to Piping Analysis

The relative displacement between a point on the reactor building and a point on the primary containment is obtained as follows:

1. The displacement time history at each point is calculated using as input the three-directional synthetic time history of ground motion.

2. The displacement time history of the first point is subtracted from that of the second to get a time history of relative displacement.

Some examples of displacements are included on Figure 3.7A-35.

The effect of relative displacement between the supports is considered in the piping analysis.

One support is selected as a reference point. Dynamic displacements at this reference point are taken either as zero or as displacements relative to a given structural node. At the first (adjacent) support from the reference point, relative displacements between this support and the reference point are taken and added to the dynamic displacement of the reference point (the direction of displacements is determined and applied to the supports in relation to its function). The resultant displacements become the dynamic displacements of the first support. The first support then becomes the new reference point. This process is repeated until dynamic displacements at all supports are obtained.

3.7A.3.9 Multiply Supported Equipment Components with Distinct Inputs

When a subsystem is attached to different parts of a structure, such as separate elevations on one wall or several walls, the response spectra of all structural nodes for which response spectra exist and which lie nearest to the support elevation at the subsystem, both below and above the support elevation, are enveloped, and this envelope spectrum is applied to the

subsystem. In cases where a subsystem runs between two different buildings, a single ARS enveloping the spectra associated with all support points is used.

In conjunction with the response spectrum loading, the loading from differential support displacements is calculated, and the two load cases are combined as described in Section 3.9A.3.1. Components and equipment generally have localized supports and the effect can be ignored. The application to piping is discussed in detail in Section 3.7A.3.8.3.

3.7A.3.10 Use of Constant Vertical Static Factors

Constant vertical static factors are not used.

3.7A.3.11 Torsional Effects of Eccentric Masses

For Category I piping systems, concentrated loads in the piping system, such as valves and valve operators, are modeled as massless members with the mass of the components lumped at the center of gravity. A rigid member is modeled connecting the center of gravity to the piping so that the torsional effects of the eccentric masses are considered. The stress produced at the pipe connection is given in the NUPIPE output.

3.7A.3.12 Buried Category I Piping Systems and Tunnels

There are no buried Category I piping systems at Unit 2. The design of Category I tunnels is performed in one of two ways.

1. Tunnels which are physically connected to other Category I structures are founded on rock and are analyzed and designed for seismic events as parts of these structures.

2. Tunnels which are physically separated from other structures are founded on rock (including buried Category I tunnels) and are analyzed using the methods of Section 3.7 for seismic events or are conservatively designed for acceleration values of 0.5g for SSE and 0.25g for OBE. A typical dynamic analysis was performed to demonstrate and verify that the acceleration values chosen were conservative.

Partially or completely buried tunnels are also designed to consider the static and dynamic effects of the backfill using the pressure distribution shown on Figure 2.5-110.

The design of Category I tunnels is described in Section 3.8.4.4.7.

3.7A.3.13 Interaction of Other Piping with Category I Piping

In order to prevent propagation of failure from the seismically induced effect of nonseismic class piping on Category I piping,

each nonseismic class piping system is generally designed to be isolated from any Category I piping system. If it is not feasible or practical to isolate the Category I piping system from the nonseismic class piping system, then adjacent nonseismic class piping is seismically designed according to the same criteria that are applicable to the Category I piping system. For the nonseismic class piping systems interfacing with Category I piping systems, the seismic analysis encompasses the nonseismic class system to the first anchor point.

Isolation anchors that separate QA Category I from nonsafety-related piping are designed considering seismic loads from both sides of the anchor. If the nonsafety-related side has not been seismically analyzed, the seismic load from that side of the anchor is assumed to be three times the seismic load from the QA Category I side. The Engineer will evaluate the piping and support design of the nonsafety-related piping to ensure that the seismic load will not exceed the assumed loads. In the event that the assumption cannot be justified, the anchor will be designed to sustain the maximum moment which the nonsafety-related piping can impose on the anchor.

The loading conditions and load combinations are in accordance with Table 3.9A-14. Allowable stresses designated in the AISC specification are used for the design of structural members and welds. When the maximum moment is used, the allowable stresses in the structural member will be increased to 0.9Sy.

Allowable stresses for induced local pipe wall stresses are in accordance with ASME III, Subsection NC, 1974. When the maximum moment is used, the allowable local stress will be increased to 2Sy.

3.7A.3.14 Seismic Analyses for Reactor Internals

See Section 3.7B.3.14.

3.7A.3.15 Analysis Procedure for Damping

The percentages of critical damping values assigned to Category I subsystems and components are in accordance with RG 1.61 (Table 3.7A-1). Higher damping values than those delineated in Table 3.7A-1 are used where documented test data are provided to support them, as allowed by RG 1.61, paragraph C.2.

In the dynamic analysis of any particular item of equipment, the same percentage of critical damping is used for all modal responses. In cases where pipe size dictates the use of two sets of damping values for the same analysis, the damping values corresponding to the pipe size of the majority of the system are used for the entire analysis.

3.7A.4 Criteria for Seismic Instrumentation Program

3.7A.4.1 Comparison with Regulatory Guide 1.12

A seismic instrumentation program has been implemented to monitor and record input motion and behavior of the plant in the event of an earthquake. This instrumentation program complies with the requirements of RG 1.12 and ANSI Standard ANSI/ANS-2.2-1978.

3.7A.4.2 Location and Description of Instrumentation

3.7A.4.2.1 Triaxial Time History Accelerograph

Strong motion triaxial time history accelerometers are installed in three locations. Two accelerometer sensor packages are installed in the reactor building outside of the primary containment. One sensor package is located on the reactor mat in the secondary containment adjacent to the exterior reactor building wall at el 175 ft, and the second is located approximately 178 ft above the first on the refueling floor at el 353 ft. The third sensor package is located on top of the control building mat at el 214 ft.

The strong motion triaxial time history accelerograph has the following physical characteristics:

1. Accelerometers are of the force-balance type, with the capability of recording a maximum of 1.0g at full scale.

2. Accelerometers are sensitive to frequencies in the range of 0 to 50 Hz.

3. The seismic instrumentation and recording system is in a quiescent state until activated by seismic triggers set at 0.01g. These seismic triggers (both horizontal and vertical) activate the recording system in less than 100 msec. The recording system will operate continuously during the period in which the earthquake exceeds the 0.01g threshold, plus 10 sec minimum beyond the last seismic trigger signal. The system will be capable of a minimum of 30 min total recording time.

4. Each sensor package contains three orthogonal accelerometers. All strong motion sensor packages are oriented to the same azimuths.

3.7A.4.2.2 Triaxial Peak Accelerograph

Also in accordance with RG 1.12, triaxial peak accelerographs are installed on other selected Category I structures, equipment, and components to verify the seismic response determined analytically by using the traces recorded by the accelerographs. The accelerographs are located as follows:

1. Reactor pedestal.

2. Reactor building high-pressure core spray (CSH) piping.

3. Service water piping in diesel generator building.

These instruments detect and record peak amplitudes of accelerations in a minimum frequency range of 0 to 26 Hz. Triaxial peak accelerographs have the following physical characteristics:

1. Accelerographs are of the short period torsional type, with a sensitivity from 0.01 to 10g full scale.

2. The accelerograph records by scribing excursions of a diamond stylus on a replaceable metal plate.

3. No power is required to operate the instrument.

4. Air damping is used to 60 percent of critical with an accuracy of ±5 percent of critical.

5. Operating temperature range is -40°F to 185°F.

3.7A.4.2.3 Triaxial Response Spectrum Recorder

The triaxial response spectrum recorder senses and permanently records information defining a response spectrum. It is a completely passive device, covering the range from 1 to 32 Hz in 1/3 octave increments. Sixteen reeds of different lengths and weights, 1 for each frequency, have attached to their free end a diamond-tipped stylus that inscribes a permanent record of its deflection on 16 respective record plates. A calibration sheet lists the resonant frequency and g-sensitivity of each reed and allows a plot of acceleration versus frequency to be made. These instruments have the following physical characteristics:

1. Damping for the oscillators is not less than 2 percent nor more than 5 percent of critical.

2. Operating temperature range is -40°F to 185°F.

3. Accuracy: frequency is ±1 percent; acceleration is ± 3 percent of full scale; damping is ±0.15 percent of nominal.

4. The dynamic range for acceleration is 100 to 1 minimum.

Four triaxial response spectrum recorders are installed at the following locations:

1. Reactor building mat (el 175 ft).

2. Control building mat (el 214 ft).

3. Refueling floor (el 353 ft 10 in).

4. Primary containment wall penetration for RHR piping (el 294 ft 6 in).

The basis for selection of these locations is to provide some measure of redundancy to the strong motion accelerographs and also to provide additional data to verify the seismic response determined analytically by using the traces recorded by the strong motion accelerographs.

3.7A.4.2.4 Triaxial Seismic Switch

One triaxial seismic switch is installed on the reactor building mat to provide an immediate signal to the main control room to indicate if specified design accelerations (OBE) have been exceeded.

The seismic switch has the following physical characteristics:

1. The package is composed of three orthogonal acceleration transducers.

2. Setpoint is adjustable from 0.025 to 0.25g.

3. Switch remains closed for 6 to 20 sec (adjustable) after detection of an acceleration over the preset value.

4. Operating temperature range is from 0°F to 130°F.

3.7A.4.3 Main Control Room Operator Notification

Recording equipment, seismic annunciators, and response spectrum annunciators are located in the relay and computer room, el 288. The annunciators have both visual and audible alarms to notify the Control Room Operator when a seismic event causes the OBE design accelerations to be exceeded.

The seismic trigger on the reactor building mat activates the triaxial accelerograph and the magnetic tape recording system when the acceleration exceeds 0.01g.

The response spectrum recorder on the reactor building mat activates the response spectrum annunciator when preset g levels at corresponding frequencies are exceeded.

The triaxial seismic switch on the reactor building mat sets off the seismic annunciator when the acceleration exceeds the OBE limits of 0.075 g horizontal and 0.050 g vertical.

The seismic trigger and, hence, the triaxial accelerograph and the magnetic tape recording system will be activated by accelerations from nonseismic events such as SRV blowdown loads and hydrodynamic loads due to LOCA events. However, since the normal operating SRV acceleration level (i.e., the acceleration value corresponding to zero period in the reactor building mat

design response spectra, as defined in RG 1.12) is less than the OBE acceleration level, the seismic switch will not be activated by the normal operating nonseismic events. Therefore, these events will not interfere with normal plant operations.

3.7A.4.4 Comparison of Measured and Predicted Responses

To determine if a nuclear power plant can continue to operate safely following a seismic event, comparisons are made between the seismic response as measured by the seismic instrumentation and the computed response used as a design basis. Such comparisons are performed only after OBE or more severe seismic conditions occur. To make such comparisons, the following procedure is implemented:

1. Magnetic tape records are digitized and corrected for time signal variations and baseline deviations.

2. Time-history records from triaxial sensors located on the reactor building mat are used to directly calculate ARS at appropriate critical damping values.

3. Time-history records from the reactor building mat sensor are used as input ground motion for the reactor building dynamic model. ARS are then calculated at the locations of the other two sensors in the containment structure for comparison and correlation with the response spectra determined in Item 2. Reasonable correlation between the spectra is accomplished on an iterative basis by varying the physical properties of the models (stiffnesses and damping characteristics) to calibrate the dynamic model. Once the dynamic model has been calibrated, additional verification of its correctness is made using the acceleration readings from the peak recording accelerographs.

The results of the comparison will be used to evaluate the seismic effects on the structures and equipment by forming the basis for remodeling, detailed reanalysis, and physical inspection. The reanalysis and inspection results will be used to determine the appropriate actions that are required as a result of the earthquake.

3.7A.4.5 Inservice Surveillance Requirements

Inservice surveillance will be performed on the seismic instrumentation at the intervals specified in the TRM.

3.7A.5 References

1. Newmark, N. M., Blume, J. A., and Kapur, K. K. Design Response Spectra for Nuclear Power Plants, Journal of Power Division, Proceedings of American Society of Civil Engineers, Vol. 99, No. P02, November 1973.

2. Roesset, J. M., Whitman, R. V., and Dobry, R. Modal Analysis for Structures with Foundation Interaction. Journal of the Structural Division, Proceedings ASCE, pp 399-416, March 1973.

3. Whitman, R. V. Soil Structure Interaction. Seismic Design for Nuclear Plants. MIT Press, Cambridge, MA, pp 241-269, 1970.

4. Gwinn, J. M. and Goldstein, N. A. Equivalent Static Loads from Amplified Response Curves. ASME Paper 74-NE-6, presented at the American Society of Mechanical Engineers Pressure Vessel and Piping Conference, Miami, FL, June 1974.

5. ASCE Structural Division Committee on Nuclear Structures and Materials, Seismic Response of Buried Pipes and Structural Components, Report Prepared by Committee on Seismic Analysis.

TABLE 3.7A-1
(Sheet 1 of 1)

DAMPING FACTORS

Item, Equipment, or Structure	Damping (Percent Critical) OBE	SSE
Equipment and large diameter piping systems, pipe diameter greater than 12 in	≤3*	≤4*
Small diameter piping systems, diameter less than or equal to 12 in	≤3*	≤4*
Welded or bolted steel structures with friction connections	≤3	4
Bolted steel structures with bearing connections	≤5	7
Prestressed concrete structures	≤3	5
Reinforced concrete structures	4	7

* An alternate approach specified in RG 1.61 Revision 1, Section C.2, Piping Damping, may also be used with the associated restrictions.

TABLE 3.7A-2
(Sheet 1 of 1)

FOUNDATION/SUPPORT MEDIA FOR CATEGORY I STRUCTURES

Category I Structure	Foundation Embedment Depth (ft)	Approx. Contact Area of Structural Foundation (ft x ft)	Total Structural Height (ft)	Support Media
Control building	51	(1)	117.5	Rock
Diesel generator building	19	(1)	48	Rock
Electrical ductlines Cl-IE 907 Cl-IE 922	 14(2) 14(2)	 4.3 x 107.5 4.3 x 40	 3.75 3.75	Category I structural backfill
Electrical manhole No. 1	17	12.5 x 12.5	17	Category I structural backfill
Electrical tunnels	50(1)	17 (min) x 730	20 (min)	Rock
Intake structure	-	(1)	10.5	Rock
Intake tunnels	-	(1)	6.8	Rock
Main stack	28.5	(1)	430	Rock
Offgas room	24	(1)	98.8	Rock
Radwaste building	29	(1)	103.5	Rock
Reactor building	97	(1)	255.12	Rock
Screenwell building	39	(1)	112(3)	Rock
Service water tunnel	21 (max)	12 x 670	8.7 (max)	Rock
Standby gas treatment building and railroad access lock	14 (min)	(1)	71.0	Rock
Turbine building(4)	25	(1)	126(3)	Rock

(1) The approximate contact areas of structural foundations are given in Table 3.8-13.
(2) Top surface of structure is below plant grade.
(3) Category I status does not extend to roof.
(4) Turbine building is Category I in the following areas only:
a. Pipe tunnel below the electrical bay area between column lines AK and AM up to el 261' and pipe tunnels below the turbine building between column lines 10 and 12 up to el 248'.
b. Main steam tunnel area and area underneath main steam tunnel between column lines 10 and 12, when providing support to main steam tunnel.

TABLE 3.7A-3
(Sheet 1 of 1)

PRIMARY CONTAINMENT MODE SHAPES (CRACKED)

	Mode Frequency (CPS) Direction of Displacement									
	1	2	3	4	5	6	7	8	9	10
	3.763	3.764	8.829	8.8513	9.908	11.220	11.242	12.327	15.677	16.202
Node	N-S	E-W	E-W	N-S	Torsional	E-W	N-S	Vertical	Torsional	E-W
Reactor										
4	1.0000	1.0000	1.0000	1.0000	0.9977	0.3786	0.3720	0.9898	0.9992	1.0000
2	0.8536	0.8539	0.7236	0.7228	0.9881	0.0392	0.0338	0.9818	0.9753	0.4191
6	0.7110	0.7116	0.4800	0.4787	0.9579	-0.3085	-0.3125	0.9653	0.9000	-0.0765
10	0.5607	0.5617	0.2343	0.2328	0.9121	-0.6110	-0.6131	0.9382	0.7879	-0.4563
Shield Wall										
1	0.8141	0.8143	0.5076	0.5073	1.0000	0.1183	0.1141	0.9253	1.0000	0.0376
3	0.7291	0.7295	0.4153	0.4146	0.9805	-0.1208	-0.1246	0.9143	0.9512	-0.1303
7	0.6417	0.6423	0.3087	0.3079	0.9381	-0.3280	-0.3311	0.8903	0.8468	-0.2561
11	0.5499	0.5507	0.1901	0.1892	0.8725	-0.4865	-0.4886	0.8528	0.6897	-0.3155
Pedestal										
13	0.4578	0.4587	0.0709	0.0700	0.7855	-0.5744	-0.5752	0.8026	0.4905	-0.2927
15	0.3653	0.3664	-0.0369	-0.0378	0.6851	-0.5591	-0.5587	0.7243	0.2738	-0.1440
18	0.1802	0.1812	-0.0802	-0.0804	0.3904	-0.4448	0.4441	0.4373	-0.0060	0.0715
20	0.0703	0.0709	-0.0371	-0.0371	0.1989	-0.2363	0.2354	0.2215	-0.0032	0.0484
Drywell Floor										
17	0.2809	0.2821	-0.1053	-0.1056	0.5455	-0.4874	-0.4865	0.6165	-0.0080	-0.0562
Primary Containment										
8	0.7539	0.7535	-0.1409	-0.1401	0.9004	1.0000	1.0000	1.0000	-0.0528	-0.0696
5	0.6892	0.6893	-0.1359	-0.1353	0.8896	0.7675	0.7660	0.9882	-0.0512	-0.0259
9	0.6258	0.6263	-0.1525	-0.1520	0.8672	0.5599	0.5577	0.9628	-0.0480	0.0093
12	0.5431	0.5439	-0.1581	-0.1578	0.8159	0.2620	0.2601	0.9086	-0.0408	0.0472
14	0.4582	0.4592	-0.1509	-0.1509	0.7418	-0.0345	-0.0357	0.8319	-0.0309	0.0697
16	0.3743	0.3755	-0.1349	-0.1351	0.6580	-0.2901	-0.2903	0.7413	-0.0204	0.0757
19	0.1775	0.1785	-0.0815	-0.0817	0.4004	-0.4666	-0.4652	0.4425	-0.0066	0.0946
21	0.0780	0.0786	-0.0444	-0.0444	0.2080	-0.3022	-0.3010	0.2269	-0.0036	0.0840

TABLE 3.7A-4
(Sheet 1 of 1)

PRIMARY CONTAINMENT MODE SHAPES (UNCRACKED)

	Mode Frequency (CPS) Direction of Displacement									
	1	2	3	4	5	6	7	8	9	10
	5.003	5.004	9.264	9.287	13.659	14.407	14.430	15.691	16.053	17.457
Node	E-W	N-S	E-W	N-S	Torsional	E-W	N-S	Vertical	Torsional	E-W
Reactor										
4	1.0000	1.0000	1.0000	1.0000	0.9975	1.0000	1.0000	1.0000	0.9999	1.0000
2	0.8347	0.8344	0.7153	0.7147	0.9793	0.3858	0.3843	0.9868	0.9748	0.4187
6	0.6788	0.6783	0.4671	0.4662	0.9221	-0.1763	-0.1781	0.9600	0.8958	-0.0543
10	0.5146	0.5139	0.2184	0.2175	0.8363	-0.6322	-0.6331	0.9158	0.7785	-0.3995
Shield Wall										
1	0.7581	0.7582	0.4785	0.4783	1.0000	0.1745	0.1735	0.8873	1.0000	-0.0703
3	0.6771	0.6769	0.3899	0.3895	0.9629	-0.0840	-0.0853	0.8703	0.9488	-0.1972
7	0.5894	0.5891	0.2864	0.2860	0.8832	-0.3028	-0.3041	0.8331	0.8395	-0.2706
11	0.4934	0.4930	0.1706	0.1703	0.7617	-0.4512	-0.4521	0.7755	0.6754	-0.2723
Pedestal										
13	0.3946	0.3940	0.0545	0.0543	0.6049	-0.4992	-0.4995	0.6992	0.4681	-0.1999
15	0.2946	0.2940	-0.0488	-0.0490	0.4302	-0.3893	-0.3887	0.5789	0.2436	-0.0245
18	0.1358	0.1352	-0.0796	-0.0794	0.1444	-0.2035	-0.2026	0.2991	-0.0350	0.2045
20	0.0542	0.0538	-0.0358	-0.0355	0.0748	-0.1204	-0.1196	0.1527	-0.0184	0.1392
Drywell Floor										
17	0.2080	0.2074	-0.1098	-0.1095	0.1963	-0.2010	-0.2003	0.4164	-0.0465	0.1746
Primary Containment										
8	0.5409	0.5421	-0.2311	-0.2308	0.3149	0.4738	0.4727	0.6083	-0.0934	-0.2361
5	0.4958	0.4967	-0.2138	-0.2135	0.3113	0.3752	0.3738	0.6025	-0.0919	-0.1614
9	0.4502	0.4507	-0.2090	-0.2086	0.3039	0.2808	0.2794	0.5899	-0.0889	-0.0916
12	0.3915	0.3917	-0.1937	-0.1933	0.2868	0.1484	0.1472	0.5631	-0.0820	-0.0005
14	0.3318	0.3316	-0.1712	-0.1707	0.2621	0.0156	0.0148	0.5249	-0.0721	0.0809
16	0.2730	0.2727	-0.1446	-0.1443	0.2341	-0.1020	-0.1020	0.4795	-0.0610	0.1429
19	0.1320	0.1315	-0.0762	-0.0760	0.1437	-0.1845	-0.1837	0.2964	-0.0349	0.1786
21	0.0584	0.0580	-0.0373	-0.0371	0.0745	-0.1158	-0.1152	0.1513	-0.0184	0.1214

TABLE 3.7A-5
(Sheet 1 of 2)

SECONDARY CONTAINMENT MODE SHAPES

Node	Mode Frequency (cps) Direction								
	1st			2nd			3rd		
	3.502			3.710			4.355		
	E-W	Vert	N-S	E-W	Vert	N-S	E-W	Vert	N-S
20	-0.1028	-0.0266	1.0000	1.0000	-0.0184	0.1210	0.0004	1.0000	0.0022
21	-0.0780	-0.0097	0.7356	0.7268	-0.0052	0.0851	0.0007	0.0173	0.0017
22	-0.0708	-0.0110	0.6373	0.6402	-0.0100	0.0712	0.0011	0.0136	0.0019
23	-0.0666	-0.0124	0.5598	0.5808	-0.0183	0.0595	0.0014	0.0129	0.0020
24	-0.0381	-0.0057	0.3345	0.3624	0.0035	0.0389	0.0012	0.0074	0.0017
25	-0.0165	0.0098	0.1897	0.1825	0.0019	0.0247	0.0006	0.0036	0.0012
26	-0.0118	0.0126	0.1345	0.1315	0.0009	0.0175	0.0004	0.0026	0.0008
27	-0.0063	0.0069	0.0728	0.0709	0.0005	0.0095	0.0002	0.0014	0.0004
28	-0.0039	0.0035	0.0431	0.0438	0.0002	0.0056	0.0001	0.0009	0.0002
	4th			5th			6th		
	4.643			5.880			7.588		
Node	E-W	Vert	N-S	E-W	Vert	N-S	E-W	Vert	N-S
20	-0.1940	0.0756	1.0000	-0.0521	0.0146	1.0000	1.0000	0.0136	0.2847
21	-0.1170	-0.0088	0.5837	-0.0238	-0.0116	0.3878	0.0581	-0.0270	0.0163
22	-0.0622	-0.0101	0.3482	0.0209	-0.0133	0.1274	-0.0406	-0.0303	-0.0115
23	-0.0091	-0.0108	0.0338	0.0708	-0.0152	-0.2133	-0.0807	-0.0353	-0.0246
24	-0.1517	-0.0060	0.4191	-0.1703	-0.0061	0.3782	-0.1165	-0.0114	-0.0353
25	-0.1482	0.0104	0.2196	-0.2047	0.0081	0.1972	-0.0810	-0.0058	-0.0274
26	-0.1348	0.0136	0.1366	-0.1937	0.0110	0.1138	-0.0654	-0.0045	-0.0225
27	-0.0686	0.0076	0.0722	-0.1015	0.0062	0.0596	-0.0387	-0.0026	-0.0140
28	-0.0422	0.0039	0.0439	-0.0633	0.0031	0.0370	-0.0253	-0.0017	-0.0091
	7th			8th			9th		
	7.635			9.199			10.250		
Node	E-W	Vert	N-S	E-W	Vert	N-S	E-W	Vert	N-S
20	-0.2974	0.0059	1.0000	0.9278	-0.2954	0.7576	1.0000	0.0263	-0.2876
21	-0.0157	-0.0120	0.0521	-0.1857	1.0000	-0.1589	-0.4509	-0.1169	0.1316
22	0.0118	-0.0119	-0.0409	-0.1210	0.9930	-0.1129	-0.2970	-0.1018	0.0871
23	0.0220	-0.0114	-0.0834	-0.0356	0.9869	-0.0458	-0.1353	-0.0765	0.0316
24	0.0334	-0.0106	-0.1121	0.0818	0.6150	0.0917	0.3143	-0.1020	-0.0849
25	0.0231	-0.0023	-0.0840	0.0778	0.3123	0.1094	0.3642	-0.0430	-0.1006
26	0.0190	-0.0019	-0.0691	0.0675	0.2348	0.0952	0.3279	-0.0309	-0.0909
27	0.0111	-0.0015	-0.0432	0.0419	0.1343	0.0609	0.2094	-0.0184	-0.0613
28	0.0072	-0.0010	-0.0283	0.0280	0.0876	0.0395	0.1408	-0.0123	-0.0416

TABLE 3.7A-5
(Sheet 2 of 2)

SECONDARY CONTAINMENT MODE SHAPES

Node	10th			
	10.448			
	E-W	Vert	N-S	
20	0.1982	0.0371	1.0000	
21	-0.1034	-0.1731	-0.5115	
22	-0.0731	-0.1621	-0.3499	
23	-0.0394	-0.1475	-0.1509	
24	0.0667	-0.1092	0.3617	
25	0.0977	-0.0796	0.4269	
26	0.0921	-0.0620	0.3987	
27	0.0595	-0.0323	0.2739	
28	0.0400	-0.0200	0.1869	

TABLE 3.7A-6
(Sheet 1 of 1)

CONTROL AND DIESEL GENERATOR BUILDING MODE SHAPES

Node	Mode Frequency (CPS) Direction											
	1			2			3			4		
	7.417			10.898			11.603			19.078		
	E-W	Vert	N-S	E-W	Vert	N-S	E-W	Vert	N-S	E-W	Vert	N-S
6	1.000	0.005	0.028	-0.059	0.083	1.000	0.723	0.034	0.189	1.000	0.065	0.053
5	0.797	0.006	0.019	-0.044	0.069	0.810	0.537	0.034	0.179	-0.026	0.063	0.004
4	0.382	0.001	0.018	-0.048	0.001	0.568	1.000	0.004	0.097	-0.786	0.020	-0.045
3	0.182	-0.000	0.014	-0.006	0.019	0.265	0.143	0.005	0.031	-0.610	0.016	-0.055
2	0.103	0.001	-0.005	-0.007	0.028	0.110	-0.186	0.011	0.031	-0.368	0.016	0.002

Node	5			6			7			8		
	20.127			22.694			24.552			26.169		
	E-W	Vert	N-S	E-W	Vert	N-S	E-W	Vert	N-S	E-W	Vert	N-S
6	-0.091	1.000	0.100	-0.416	-0.305	0.709	0.020	-0.383	1.000	0.676	0.115	0.052
5	-0.010	0.846	-0.100	-0.276	-0.270	0.041	0.036	-0.351	0.117	1.000	0.045	0.054
4	0.063	0.399	-0.214	1.000	-0.186	-0.442	0.087	-0.332	-0.628	-0.290	0.110	-0.029
3	0.036	0.266	-0.180	-0.132	-0.103	-0.407	0.048	-0.152	-0.595	-0.480	0.059	-0.047
2	0.029	0.194	-0.113	-0.647	-0.052	-0.174	0.140	-0.047	-0.350	-0.399	-0.001	-0.012

TABLE 3.7A-7
(Sheet 1 of 1)

PRIMARY CONTAINMENT MODAL RESPONSES (CRACKED)

Horizontal SSE Acceleration (North-South)
(ft/sec^2)

Mode	Frequency (cps)	Node 1 Shield Wall El 315.08'	Node 13 Pedestal El 265.5'	Node 17 Drywell Floor El 238'	Node 8 Primary Containment El 330.08'
1	3.76	-19.8013	-11.1353	-6.8322	-18.3368
4	8.85	7.9392	1.0956	-1.6532	-2.1931
7	11.24	0.9643	-4.8618	-4.1124	8.4525
11	16.22	-0.2755	2.1074	-0.4068	0.5002
13	21.45	-2.3919	0.0430	0.1094	-1.3993
16	21.90	1.7953	0.0572	0.0596	-0.8474
19	24.44	-0.2266	0.1609	0.1863	0.0692
22	25.56	-0.3213	0.2430	0.0722	-0.7095
24	29.36	-0.6003	0.3903	0.0315	0.2673
26	31.23	1.0818	-0.5080	0.4893	0.9233
30	38.17	0.0622	0.0118	0.0182	-0.3727
33	41.09	-0.1806	-0.1220	-0.0339	0.0370
35	43.85	-0.0093	-0.0084	0.0007	0.0083
37	45.10	-0.0223	-0.0342	0.0200	-0.0309
40	51.74	-0.0014	0.0017	-0.0028	0.2497
43	53.27	-0.0230	0.0673	-0.0611	-0.1200
44	53.40	0.0166	-0.0027	-0.0147	-0.0257
49	57.64	0.0206	-0.0007	-0.0092	-0.0053
50	61.54	0.0068	0.0065	-0.0082	0.0006
54	64.30	0.0003	-0.0010	0.0018	-0.0229
58	69.49	0.0115	-0.0004	0.0003	-0.0042
60	70.55	-0.0115	0.006	-0.0005	-0.0169

TABLE 3.7A-8
(Sheet 1 of 1)

PRIMARY CONTAINMENT MODAL RESPONSES (CRACKED)

Vertical SSE Acceleration
(ft/sec^2)

Mode	Frequency (cps)	Node 1 Shield Wall El 315.08'	Node 13 Pedestal El 265.5'	Node 17 Drywell Floor El 230'	Node 8 Primary Containment El 330.08'
8	12.33	-10.7415	-9.3176	-7.1570	-11.6089
14	21.76	-0.0638	-0.0386	-0.0004	0.0451
27	33.47	-1.8872	-0.2795	-0.3137	0.4762
28	36.44	3.3469	0.0678	-1.2377	1.6788
47	57.24	-0.1455	0.1143	0.1186	-1.3309
55	66.30	-0.4614	0.4514	0.1176	0.2832

TABLE 3.7A-9
(Sheet 1 of 1)

SSE ACCELERATION COMPARISON

(g)

Location	Response Spectrum			Time History		
	E-W	Vert	N-S	E-W	Vert	N-S
Secondary Containment						
PT 20 El 416.83	1.222	0.528	1.217	1.166	0.552	1.084
PT 22 El 353.83	0.518	0.394	0.506	0.624	0.431	0.483
PT 24 El 289.00	0.331	0.265	0.333	0.367	0.295	0.379
Primary Containment						
PT 1 Shield Wall						
315.08 CR	0.678	0.371	0.678	0.729	0.328	0.696
UCR	0.743	0.380	0.742	0.781	0.420	0.820
PT 13 Pedestal						
266.5 CR	0.386	0.290	0.386	0.388	0.295	0.400
UCR	0.397	0.296	0.396	0.464	0.336	0.453
PT 17 Drywell Floor						
238.0 CR	0.255	0.227	0.254	0.325	0.251	0.282
UCR	0.216	0.179	0.216	0.272	0.252	0.302
PT 8 Primary Containment						
330.08 CR	0.636	0.369	0.637	0.669	0.345	0.642
UCR	0.534	0.270	0.535	0.564	0.299	0.670
Control Building						
PT 6	0.473	0.254	0.470	0.498	0.242	0.431
PT 5	0.365	0.211	0.369	0.393	0.230	0.370

TABLE 3.7A-10
(Sheet 1 of 1)

TABULATION OF STRUCTURAL GAP AND WORST-CONDITION GAP FOR SSE CONDITION

Structural Gap Provided Between Building A and Building B		Structural Gap (in)	Maximum Deflection at Roof Elev. of Shorter Bldg.		Cumulative Deflection (in)	Worst Computed Gap (in)
			Bldg. A (in)	Bldg. B (in)		
Reactor	Auxiliary Service	3	0.285	0.088	0.373	2.627
Reactor	Standby Gas Treatment	3	0.466	0.162	0.628	2.372
Reactor	Radwaste Tunnel	3	0.155	0.037	0.192	2.808
Reactor	Main Steam Tunnel	3	0.466	0.070	0.536	2.464
Control	Normal Switchgear	3	0.031	0.055	0.086	2.914
Control	Turbine	3	0.037	0.081	0.118	2.882
Control	Auxiliary Service	3	0.021	0.075	0.096	2.904
South Auxiliary Bay	Control	3	0.119	0.015	0.134	2.866
South Auxiliary Bay	Auxiliary Service	3	0.119	0.024	0.143	2.857
North Auxiliary Bay	Standby Gas Treatment	3	0.155	0.003	0.158	2.842
North Auxiliary Bay	Radwaste Tunnel	3	0.119	0.024	0.143	2.857
Radwaste	Turbine	3	0.050	0.081	0.131	2.869
Radwaste	Screenwell	12	0.148	0.059	0.207	11.793

TABLE 3.7A-11
(Sheet 1 of 1)

DESIGN ATTRIBUTES TO BE REVIEWED FOR VERIFICATION OF LARGE BORE SAFETY-RELATED PIPING AS REQUIRED BY IE BULLETIN 79-14

Design Attribute	Source Document
Pipe run configuration and geometry	L/B as-built drawing
Pipe support location	L/B as-built drawing
Valve location	L/B as-built drawing
Support design and Function	Engineering design drawing
Embedment plate, base plate, and structural steel	Engineering design drawing and pipe support installation Specification No. NMP2-P301J
Pipe clearance at Supports	Engineering design drawing
Other pipe clearances	Construction Site Instructions CSI 2.11
Attachments to pipe	L/B as-built drawing and engineering design drawing
Attachments to supports	Engineering design drawing
Valve weights	Vendor drawing
Amplified response spectra	Piping Design Specification No. NMP2-P301A
Support seismic anchor motion	Piping Design Specification No. NMP2-P301A
Other design attributes	Piping Design Specification No. NMP2-P301A

TABLE 3.7A-12
(Sheet 1 of 1)

DESIGN ATTRIBUTES TO BE REVIEWED FOR VERIFICATION OF SMALL BORE SAFETY-RELATED PIPING AS REQUIRED BY IE BULLETIN 79-14

Design Attribute	Source Document
Pipe run configuration and geometry	Engineering design drawing
Pipe support location	Engineering design drawing
Valve location	Engineering design drawing
Support design and Function	Engineering design drawing
Embedment plate	Engineering design drawing and S/B pipe support installation Specification No. NMP2-P301F
Pipe clearance at Supports	Engineering design drawing
Other pipe clearances	Construction Site Instructions CSI 2.11
Attachments to pipe	Engineering design drawing
Attachments to supports	Engineering design drawing
Valve weights	Vendor drawing
Amplified response spectra	Piping Design Specification No. NMP2-P301A
Support seismic anchor motion	Piping Design Specification No. NMP2-P301A
Other design attributes	Piping Design Specification No. NMP2-P301A

TABLE 3.7A-13
(Sheet 1 of 1)

COMPARISON OF RESULTS: STATIC VERSUS DYNAMIC ANALYSIS FOR INSTRUMENT PANEL 2DFM-PNL102

Description of Response		Static Analysis Using 1.3 Static Coefficient (Vendor Rpt. #18609-83N-2; Spec. C062G) (in)	Response - Spectrum Dynamic Modal Analysis (Run #4 of SWEC Calc. No. 12177-NM(C)-MS-1841 (in)	Ratio = Static Analysis Response / Dynamic Analysis Response
Maximum Deflections				
Node	Direction			
19	X1	0.0021	0.00004	52.00
	X2	0.1733	0.14001	1.24
	X3	0.0007	0.00005	14.00
66	X1	0.0012	0.00002	60.00
	X2	0.0017	0.00015	11.00
	X3	0.0085	0.00034	25.00
4	X1	0.0021	0.00004	52.00
	X2	0.0034	0.00023	14.70
	X3	0.0007	0.00005	14.00
Max. Principal Stresses		(psi)	(psi)	
Element				
Triplate 140		18,492	1,402	13.00
15		5,070	4,189	1.21
Quadplate 23		5,027	4,186	1.21
Beam 42		22,495	12,257	1.83
Max. Mounting Bolt Stresses		(psi)	(psi)	
Element 103				
Shear Resultant		9,055	413	21.90
Axial Stress (P/A)		5,725	552	10.40
Max. Machine Screw Stresses		(psi)	(psi)	
Element 95				
Shear Resultant		6,775	167	40.00
Axial (P/A)		1,775	235	7.50
Element 98				
Shear Resultant		3,897	488	8.00
Axial (P/A)		1,382	156	8.80

3.7B SEISMIC DESIGN (GE SCOPE OF SUPPLY)

As discussed at the beginning of Section 3.7, the following input is applicable to the design of systems, components and equipment within the NSSS scope of supply by GE.

All systems, components and equipment of the NSSS are defined as either Category I or non-Category I. The requirements for Category I identification are given in Section 3.2 along with a list of systems, components, and equipment that are so identified.

All systems, components and equipment important to plant safety are designed to withstand potential earthquakes defined as follows. The SSE is an earthquake based upon an evaluation of the maximum earthquake potential considering the regional and local geology, seismology, and specific characteristics of local subsurface material. This earthquake produces the maximum vibratory ground motion for which Category I systems and components are designed to remain functional. These systems and components are those necessary to ensure:

1. The integrity of the RCPB.
2. The capability to shut down the reactor and maintain it in a safe shutdown condition.
3. The capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guidelines exposures of 10CFR100.

The OBE is the earthquake which, considering the regional and local geology, seismology, and specific characteristics of local subsurface material, could reasonably be expected to affect the plant site during the operating life of the plant. This earthquake produces the vibratory ground motion for which those features of the nuclear power plant necessary for continued operation without undue risk to the health and safety of the public are designed to remain functional.

3.7B.1 Seismic Input

3.7B.1.1 Design Response Spectra

See Section 3.7A.1.1.

3.7B.1.2 Design Time History

See Section 3.7A.1.2.

3.7B.1.3 Critical Damping Values

The damping factors indicated in Table 3.7B-1 are used in the response analysis of various systems, components, and equipment,

and in preparation of floor response spectra used as forcing inputs for piping and equipment analysis or testing. These factors are in compliance with RG 1.61.

3.7B.1.4 Supporting Media for Category I Structures

See Section 3.7A.1.4.

3.7B.2 Seismic System Analysis

3.7B.2.1 Seismic Analysis Methods

Analysis of Category I GE-supplied systems and components is accomplished, where applicable, using the response spectrum or time history approach. Both utilize the natural period, mode shapes, and appropriate damping factors of the particular system. Certain pieces of equipment having very high natural frequencies may be analyzed statically if the fundamental frequency of the component is greater than the ZPA frequency of the excitation. In some cases, dynamic testing of equipment may be used for seismic qualification.

The time history analyses involve the solution of the equations of dynamic equilibrium (Section 3.7B.2.1.1) by means of the methods discussed in Section 3.7B.2.1.2. In this case, the duration of motion is of sufficient length to ensure that the maximum values of response have been obtained.

A response spectrum analysis involves the solution of the equations of motion (Section 3.7B.2.1.1) by the method discussed in Section 3.7B.2.1.3. The total seismic structural response will be predicted by combining the response calculated from the two horizontal and the one vertical analyses. When the response spectrum is used, the methods for combining the loads from the three analyses will be based on the method described in Section 3.7B.3.6.

3.7B.2.1.1 The Equations of Dynamic Equilibrium

Assuming velocity-proportional damping, the dynamic equilibrium equations for a lumped mass, distributed stiffness system are expressed in matrix form as:

$$[M][\ddot{u}(t)] + [C][\dot{u}(t)] + [K][u(t)] = 0 \qquad (3.7B\text{-}1)$$

$$u_t(t) = u(t) + u_s(t) \qquad (3.7B\text{-}2)$$

$$[M][\ddot{u}(t)] + [C][\dot{u}(t)] + [K][u(t)] = [P(t)] \qquad (3.7B\text{-}3)$$

Where:

[M] = Lumped mass matrix (nxn)

$[\ddot{u}(t)]$ = Time-dependent acceleration vector (1xn) of nonsupport points relative to the base support

[C] = Damping matrix (nxn)

$[\dot{u}(t)]$ = Time-dependent velocity vector (1xn) of nonsupport points relative to the base support

[K] = Stiffness matrix (nxn)

[u(t)] = Time-dependent displacement vector (1xn) of nonsupport points relative to the base support
$[u_t(t)] = [u(t)+u_s(t)]$ (3.7B-2)

[P(t)] = Time-dependent inertial force vector $(-[M]\ [\ddot{u}_s(t)])$ acting at nonsupport points (1xn)

The manner in which a distributed mass, distributed stiffness system is idealized into a lumped mass distributed stiffness system representation of the NSSS component is shown on Figure 3.7B-1, along with a schematic representation of relative acceleration $[\ddot{u}(t)]$, support acceleration $[\ddot{u}_s(t)]$, and total acceleration $[\ddot{u}_t(t)]$.

3.7B.2.1.2 Solution of the Equations of Motion by Mode Superposition

The technique used for the solution of the equations of motion is the method of mode superposition in which the equations of motion are decoupled by the eigen transform.

The set of homogeneous equations represented by the undamped free vibration of the system is:

$$[M][\ddot{u}(t)] + [K][u(t)] = [0] \qquad (3.7B\text{-}4)$$

Since the free oscillations are assumed to be harmonic, the displacement vector [u(t)] can be written:

$$[u(t)] = [\phi]\ e^{i\omega t} \qquad (3.7B\text{-}5)$$

Where:

$[\phi]$ = Column matrix of the amplitude of displacements [u]

ω = Circular frequency of oscillation

t = Time

i = $\sqrt{-1}$

Substituting Equation 3.7B-5 and its derivatives in Equation 3.7B-4 and noting that $e^{i\omega t}$ is unequal to zero for all values of ωt yields:

$$[-\omega^2 [M] + [K]] [\phi] = [0] \qquad (3.7B-6)$$

Equation 3.7B-6 is the characteristic equation for the classical eigenvalue problem in which the eigenvalues are the frequencies of vibrations ω_i, and the eigenvectors are the mode shapes, [ϕ_i], (i = 1,2,....,n).

For each frequency ω_i, there is a corresponding solution vector [ϕ_i]. It can be shown that the eigenvectors are orthogonal with respect to the weighted stiffness matrix [K] in the n-dimensional vector space. The eigenvectors are also orthogonal with respect to the weighted mass matrix [M].

The orthogonality of the eigenvectors is used to effect a coordinate transformation to the generalized coordinate system in which the governing equations of motion are decoupled. Thus, the problem becomes one of solving n independent differential equations rather than n simultaneous differential equations, and since the system is linear, the principle of superposition holds and the total response of the system oscillating simultaneously in n modes is determined by direct addition of the responses in the individual modes.

3.7B.2.1.3 Analysis by Response Spectrum Method

The response spectrum method is based on the fact that the modal responses can be expressed in terms of a set of convolution integrals that satisfy the governing differential equations. The advantage of this form of solution is that for a given ground motion the only variables under the integral are damping factor and frequency. Thus, for a specified damping factor, it is possible to construct a curve that gives a maximum value of the integral as a function of frequency. This curve is called a response spectrum for the particular input motion and the specified damping factor. The integral has units of velocity; consequently, the maximum of the integral is called the spectral velocity.

Using the calculated natural frequencies of vibration of the system, the maximum values of the modal responses are determined directly from the appropriate response spectrum. The modal maxima are then combined as discussed in Section 3.7B.3.7.

When the equipment is supported at more than two points located at different elevations in the building, response spectrum analysis is performed using the envelope response spectrum of all attachment points. In some cases, the worst single floor response spectrum selected from a set of floor response spectra obtained at various floors may be applied identically to all floors provided there is no significant shift in frequencies of the spectra peaks.

Alternatively, multiple support excitation analysis methods may be used where individual acceleration time-histories or response spectra are applied at all the equipment attachment points.

3.7B.2.1.4 Multisupport Excitation Analysis of Systems, Components, and Equipment

Analytical procedures for obtaining force and displacement responses engendered by time-dependent, base support excitation are discussed in preceding sections. In a multisupported system, the relative motion among the individual multisupport points gives rise to time-varying displacements at the nonsupport points.

The governing equations of motion of a multisupported system, component, or equipment undergoing individual multisupport excitations may be expressed in the following matrix form:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{F\} \qquad (3.7B\text{-}7)$$

Where:

$\{u\} = \{u(t)\}$ = The corresponding dynamic model nodal displacement vector of absolute displacements

The general case is considered in which k of the total n degrees-of-freedom corresponds to the individual multisupport points which undergo known time-history motions. The nodal displacement vector of absolute displacements can be partitioned and written as:

$$\{u\} = \left\{\frac{u_a}{\bar{u}_s}\right\} = \left\{\frac{u_a^d + u_a^s}{\bar{u}_s}\right\} \qquad (3.7B\text{-}8)$$

Where:

$\{u_a\}$ = Absolute displacement vector of the active (unsupported) degrees-of-freedom

$\{\bar{u}_s\}$ = Known absolute displacement vector corresponding to the multisupported degrees-of-freedom

The vector $\{u_a\}$ in Equation 3.7B-8 has been further separated into a dynamic part and a pseudo-static part where:

$\{u_a^d\}$ = Dynamic part of $\{u_a\}$

$\{u_a^s\}$ = Pseudo-static part of $\{u_a\}$

Multisupport excitation may require the utilization of all modes which span the $\{u_a\}$ space of active (unsupported)

degrees-of-freedom in the modal superposition in order to obtain reliable solutions of Equation 3.7B-7. Substitution Equation 3.7B-8 enables the circumventing of that very costly requirement. Only the dynamic part $\{u^d_a\}$ is obtained by modal superposition which does not require all modes. The pseudo-static part $\{u^s_a\}$ is obtained from the known multisupport excitation.

The partitioned equations of motion are obtained by substituting Equation 3.7B-8 into Equation 3.7B-7 to yield:

$$\left[\begin{array}{c|c} M_a & o \\ \hline o & M_s \end{array}\right] \left\{\begin{array}{c} \ddot{u}^d_a + \ddot{u}^s_a \\ \hline \ddot{u}_s \end{array}\right\} + \left[\begin{array}{c|c} C_{aa} & C_{as} \\ \hline C_{aa} & C_{ss} \end{array}\right] \left\{\begin{array}{c} \dot{u}^d_a + \dot{u}^s_a \\ \hline \dot{u}_s \end{array}\right\} + \left[\begin{array}{c|c} K_{aa} & K_{as} \\ \hline K_{sa} & K_{ss} \end{array}\right] \left\{\begin{array}{c} u^d_a + u^s_a \\ \hline \overline{u}_s \end{array}\right\} = \left\{\begin{array}{c} F_a \\ \hline F_s \end{array}\right\} \tag{3.7B-9}$$

Where:

$\{u^d_a\}$ = Dynamic part (as defined by Equation 3.7B-8) of the absolute displacement vector of the active (unsupported) degrees-of-freedom

$\{u^s_a\}$ = Pseudo-static part (as defined by Equation 3.7B-8) of the absolute displacement vector of the active (unsupported) degrees-of-freedom

$[M_a]$ and $[M_s]$ = Lumped diagonal mass matrices associated with the active degrees-of-freedom and the multisupport points, respectively

$[C_{aa}]$ and $[K_{aa}]$ = Damping matrix and elastic stiffness matrix, respectively, relating the forces developed in the active degrees-of-freedom to the motion of the active degrees-of-freedom

$[C_{ss}]$ and $[K_{ss}]$ = Support forces due to unit velocities and displacements, respectively, of the multisupport points

$[C_{as}]$ and $[K_{as}]$ = Damping and stiffness matrices denoting the coupling forces developed in the active degrees-of-freedom due to the motion of the supports, or vice versa

$\{F_a\}$ = Prescribed time-dependent applied load vector corresponding to the active degrees-of-freedom

$\{F_s\}$ = Reaction force vector corresponding to the system multisupport points

$(\cdot)$ = Total differentiation with respect to time when appearing over a time variable

The procedure utilized to construct the damping matrix is discussed in Section 3.7B.2.15. The mass matrix and elastic stiffness matrix are formulated by standard procedures.

Since the components of $\{\ddot{\bar{u}}_s\}$, hence of $\{\dot{\bar{u}}\}$ and $\{\bar{u}_s\}$, are known functions of time, only the first partitioned portion of Equation 3.7B-9 is of interest.

$$[M_a]\{\ddot{u}_a^d\} + [M_a][\ddot{u}_a^s\} + [C_{aa}]\{\dot{u}_a^d\} + [C_{aa}]\{\dot{u}_a^s\} + [C_{as}]\{\dot{\bar{u}}_s\} + [K_{aa}]\{u_a^d\} + [K_{ag}]\{u_a^s\} + [K_{as}]\{\bar{u}_s\} = \{F_a\} \tag{3.7B-10}$$

The pseudo-static displacement vector is written in terms of the multisupport displacement vector by taking:

$$[K_{ag}]\{u^s_a\} + [K_{as}]\{\bar{u}_s\} = \{0\} \tag{3.7B-11}$$

Therefore:

$$\{u^s_a\} = -[K_{aa}]^{-1}[K_{as}]\{\bar{u}_s\} \tag{3.7B-12}$$

It follows from Equation 3.7B-11 that:

$$[C_{aa}]\{\dot{u}^s_a\} + [C_{as}]\{\dot{\bar{u}}_s\} = \{0\} \tag{3.7B-13}$$

The partitioned portion of the equation is reduced to its final form by substituting Equations 3.7B-11, 3.7B-12, and 3.7B-13 into Equation 3.7B-10 to yield:

$$[M_a]\{\ddot{u}^d_a\} + [C_{aa}]\{\ddot{u}^d_a\} + [K_{ag}]\{u^d_a\} = \{F_a\} + [M_a][K_{aa}]^{-1}[K_{as}]\{\ddot{\bar{u}}_s\} \tag{3.7B-14}$$

The solution in time of Equation 3.7B-14 for $\{u^d_a\}$ is readily obtained by the standard normal mode solution methodology. Once $\{u^d_a\}$ is obtained, the total solution for the absolute displacement vector $\{u_a\}$, corresponding to the active degrees-of-freedom, is given by substituting $\{u^d_a\}$ from Equation 3.7B-14 and $\{u^s_a)$ from Equation 3.7B-12 into Equation 3.7B-8.

After obtaining the absolute displacement vector response of the active degrees-of-freedom, $\{u_a\}$, the second partitioned portion of Equation 3.7B-9 can be used to calculate the reaction force

vector $\{F_s\}$ corresponding to the multisupport degrees-of-freedom; i.e.:

$$\{\bar{F}_s\} = [M_s]\{\ddot{u}_s\} + [C_{sa}]\{\dot{u}_a\} + [C_{ss}]\{\dot{\bar{u}}_s\} + [K_{sa}]\{u_a\} + [K_{ss}]\{\bar{u}_s\} \quad (3.7B\text{-}15)$$

Note that $\{\bar{F}_s\}$ is the total external force vector applied to the multisupport degrees-of-freedom required to produce the given multisupport excitation $\{\ddot{u}_s\}$. The interaction force vector $\{F_s\}$ corresponding to the reaction of the active degrees-of-freedom portion of the dynamic model on the multisupport points is given by:

$$\{F_s\} = [C_{sa}]\{\dot{u}_a\} + [K_{sa}]\{u_a\} \quad (3.7B\text{-}16)$$

The interaction force vector $\{F_s\}$ can also be expressed in terms of the multisupport excitation input motion $\{\bar{u}_s\}$ by substituting Equations 3.7B-12 and 3.7B-8 into Equation 3.7B-16 to yield:

$$\{F_s\} = [C_{sa}]\{\dot{u}^d_a\} + [K_{sa}]\{u^d_a\} - [C_{sa}][K_{aa}]^{-1}[K_{as}]\{\dot{\bar{u}}_s\} - [K_{sa}][K_{aa}]^{-1}[K_{as}]\{\bar{u}_s\} \quad (3.7B\text{-}17)$$

3.7B.2.1.5 Dynamic Analysis of Category I Systems, Components, and Equipment

Time-history and response spectrum techniques are used as applicable for the dynamic analysis of Category I systems, components, and equipment that are sensitive to dynamic seismic events.

Dynamic Analysis of Piping Systems

Each pipe line is idealized as a mathematical model consisting of lumped masses connected by elastic members. The stiffness matrix for the piping system is determined using the elastic properties of the pipe. This includes the effects of torsion, bending, shear, and axial deformations as well as change in stiffness due to curved members. Next the mode shapes and the undamped natural frequencies are obtained. When the piping system is anchored and supported at points with different excitations, the response spectrum analysis is performed using the envelope response spectrum of all attachment points. Alternately, the multiple excitation analyses may be used where acceleration time-histories or response spectra are applied to piping system attachment points.

The relative displacement between anchors is determined from the dynamic analysis of the structures. The results of the relative anchor point displacement are used for a static analysis to determine the additional stresses due to relative anchor point displacements.

Dynamic Analysis of Equipment

Each component of equipment is idealized as a mathematical model consisting of lumped masses connected by elastic members or springs. When the equipment is supported at more than two points located at different elevations in the building, the response spectrum analysis is performed using the envelope response spectrum of all attachment points. Alternatively, the multiple excitation analysis methods may be used where individual acceleration time-histories or response spectra are applied at each of the equipment attachment points. The relative displacement between supports is determined from the dynamic analysis of the structure. The relative support point displacements are used for a static analysis to determine the secondary stresses due to support displacements. Further details are given below.

Differential Seismic Movement of Interconnected Components

The procedure for considering differential displacements for equipment anchored and supported at points with different input motion is as follows: The relative displacements between the supporting point induces additional stresses in the equipment supported at these points. These stresses can be evaluated by performing a static analysis where each of the supporting points is displaced a prescribed amount. The time-history of displacement at each supporting point is readily obtained from the corresponding multisupport acceleration time-history which is provided as input for the dynamic analysis of the total component. These displacements are used to calculate stresses by determining the peak nodal responses.

In the static calculation of the stresses due to relative displacements in the response spectrum method, the maximum value of the nodal displacement is used. Therefore, the mathematical model of the equipment is subjected to a maximum displacement vector of its supporting points obtained from the nodal displacements. This procedure is repeated for the significant modes (modes contributing most to the total displacement response at the supporting point) of the component. The total stresses due to relative displacements are obtained by combining the modal results using the SRSS method. Since the maximum displacements for different modes do not occur at the same time, the SRSS method is a reasonable method. When a component is covered by the ASME Boiler and Pressure Vessel Code, the stresses due to relative displacement as obtained above are treated as secondary stresses.

3.7B.2.1.6 Seismic Qualification by Testing

For certain Category I equipment and components where dynamic testing is necessary to ensure functional integrity, test performance data and results reflect the following:

1. Performance data of equipment which, under the specified conditions, has been subjected to dynamic loads equal to or greater than those to be experienced under the specified seismic in-service conditions.

2. Test data from previously tested comparable equipment which, under similar conditions, has been subjected to dynamic loads equal to or greater than those specified.

3. Actual testing of equipment in accordance with one of the methods described in Sections 3.9B.2 and 3.10B.

3.7B.2.2 Natural Frequencies and Response Loads

See Section 3.7A.2.2.

3.7B.2.3 Procedure Used for Modeling

3.7B.2.3.1 Modeling Techniques for Category I Systems, Components, and Equipment

An important step in the seismic analysis of Category I systems, components, or equipment is the procedure used for modeling. The techniques currently being used for modeling are represented by lumped masses and a set of spring dashpots idealizing both the inertial and stiffness properties of the system. The details of the mathematical models are determined by the complexity of the actual system and the information required from the analysis.

3.7B.2.3.2 Modeling of Reactor Pressure Vessel and Internals

The seismic loads on the RPV and reactor internals are based on a dynamic analysis of the reactor building with the appropriate forcing function supplied at ground level. The mathematical model of the RPV and internals is shown on Figure 3.7B-2.

The RPV and internals mathematical model consists of lumped masses connected by linear elastic beam element members. Using the elastic properties of the structural components, the stiffness properties of the model are determined and the effects of bending shear, torsion, and axial loading are included.

Mass points are located at all points of critical interest such as anchors, supports, and points of discontinuity. In addition, mass points are chosen so that the mass distribution in various zones is as uniform as practicable and the full range of frequency of response of interest is adequately represented. Further, in order to facilitate hydrodynamic mass calculations, several mass points (fuel, shroud, vessel) are selected at the same elevation. The various lengths of CRD housings are grouped into the two representative lengths shown on Figure 3.7B-2. These lengths represent the longest and shortest housings in order to adequately represent the full range of frequency response of the housings.

The high fundamental frequencies of the CRD housings result in very small seismic loads. Furthermore, the small frequency differences between the various housings due to the length differences result in negligible differences in dynamic response. Hence, the modeling of intermediate length members becomes unnecessary. Not included in the mathematical model are the stiffness of light components, such as jet pumps, in-core guide tubes and housings, spargers, and their supply headers. This is done to reduce the complexity of the dynamic model and is justified because dynamic interaction is not significant. For the seismic responses of these components, floor response spectra generated from the system analysis are used.

The presence of the fluid and other structural components (e.g., fuel within the RPV) introduces a dynamic coupling effect. Dynamic effects of water enclosed by the RPV are accounted for by introduction of a hydrodynamic mass matrix that will serve to link the acceleration terms of the equations of motion of points at the same elevation in concentric cylinders with a fluid entrapped in the annulus. The details of the hydrodynamic mass derivation are given in Seismic Analysis of the Boiling Water Reactor[1]. The seismic model of the RPV and internals has 6 degrees-of-freedom for each mass point considered in the analysis.

The shroud support plate in its own plane is extremely stiff and therefore is modeled as a rigid link in the translational direction. The shroud support legs and local flexibilities of the vessel and shroud contribute to the rotational flexibilities and are modeled as an equivalent torsional spring.

3.7B.2.4 Soil-Structure Interaction

See Section 3.7A.2.4.

3.7B.2.5 Development of Floor Response Spectra

See Section 3.7A.2.5.

3.7B.2.6 Three Components of Earthquake Motion (NSSS)

Details are the same as those given in Section 3.7B.3.6.

3.7B.2.7 Combination of Modal Responses (NSSS)

All the modal responses are combined as described in Section 3.7B.3.7.

3.7B.2.8 Interaction of Non-Category I Structures with Category I Structures

See Section 3.7A.2.8.

3.7B.2.9 Effects of Parameter Variations on Floor Response Spectra

See Section 3.7A.2.9.

3.7B.2.10 Use of Constant Vertical Static Factors (NSSS)

Constant vertical static factors are not used for systems such as the RPV, internals, and large piping. See Section 3.7B.3.10 for subsystems and components.

3.7B.2.11 Method Used to Account for Torsional Effects (NSSS)

The RPV is an axisymmetric model with no built-in eccentricity.

3.7B.2.12 Comparison of Responses (NSSS)

Either the time-history method or the response spectrum approach may be used for the seismic analysis of NSSS components. Generally, the responses computed by both methods are comparable in magnitude, with the loads determined by the response spectrum method being somewhat more conservative. As both of these approaches are acceptable, additional comparison of results is unnecessary.

3.7B.2.13 Methods for Seismic Analysis of Dams

See Section 3.7A.2.13.

3.7B.2.14 Determination of Category I Structure Overturning Moments

See Section 3.7A.2.14.

3.7B.2.15 Analysis Procedure for Damping

In a linear dynamic analysis the procedure utilized to properly account for damping in different elements of a coupled system model is as follows:

1. The structural percent critical damping of the various structural elements of the model are first specified. Each value is referred to as the damping ratio (C_j) of a particular element that contributes to the complete stiffness of the system.

2. An eigenvalue analysis of the linear system model is performed. This results in the eigenvector matrices (ϕ_i) that are normalized and satisfy the orthogonality conditions:

$$[\,'\omega_i^2] = [\phi^T][K][\phi]$$
$$[\,'I] = [\phi^T][M][\phi]$$

(3.7B-18)

Where:

[K] = Stiffness matrix

['ω_i] = Diagonal matrix of circular natural frequency of mode i

[ϕ^T] = Transpose of ϕ, which is a column vector of ϕ corresponding to the mode shape of mode i

The matrix ϕ contains all translational and rotational coordinates.

3. Using the strain energy of the individual components as a weighting function, the following equation is used to obtain a suitable damping ratio (β_i) for mode i:

$$\beta_i = \frac{l}{\omega_i} \sum_{j=i}^{N} \left[C_j \left(\phi_i^\tau \ K \ \phi_i \right) \right]$$

(3.7B-19)

Where:

N = Total number of structural elements

ϕ_i = Components of mode i eigenvector corresponding to the beam element j

β_i = Modal damping coefficient for mode i

K_j = Stiffness contribution of element j

ω_i = Circular natural frequency of mode i

ϕ_i^τ = Transpose of ϕ defined above

C_j = Percent critical damping associated with element

3.7B.3 Seismic Subsystem Analysis

3.7B.3.1 Seismic Analysis Methods

The seismic system analysis methods described in Section 3.7B.2.1 are applicable to the Category I subsystems, components, and equipment. A description is given of the following methods by which Category I subsystems and components are qualified to

ensure the functional integrity of the specific operating requirements that characterize their Category I designations.

In general, one of the following five methods of seismically qualifying the equipment is chosen based on the characteristics and complexities of the subsystem:

1. Dynamic analysis.

2. Testing procedures.

3. Equivalent static load method of analysis.

4. Combination of 1 and 2, or

5. Combination of 2 and 3.

The equivalent static load method of subsystem analysis is described in Section 3.7B.3.5.

Appropriate design response spectra (OBE and SSE) are furnished to the manufacturer of the equipment for seismic qualification purposes. Additional information such as input time-history is also supplied only when necessary.

When analysis is used to qualify Category I subsystems and components, the analytical techniques must conservatively account for the dynamic nature of the subsystems or components. Both the SSE and OBE, with their different damping values, are considered when the dynamic analysis is performed.

The general approach employed in the dynamic analysis of Category I equipment and component design is based on the response spectrum technique. The time-history technique described in Section 3.7B.2.1.1 generates time-histories at various support elevations for use in the analysis of subsystems and equipment. The structural response spectrum curves are subsequently generated from the time-history accelerations.

At each level of the structure where vital components are located, three orthogonal components of floor response spectra (two horizontal and one vertical) are developed. The response spectra are peak broadened ±15 percent.

For vibrating systems and their supports, multidegree-of-freedom models are used in accordance with the lumped-parameter modeling techniques and normal mode theory described in Section 3.7B.2.1.1 and the references listed in Section 3.7B.5. Piping analysis is described in Sections 3.7B.3.3.1 and 3.7B.2.1.5.

When testing is used to qualify Category I subsystems and components, all the loads normally acting on the equipment are simulated during the test. The actual mounting of the equipment is also simulated or duplicated. Tests are performed by

supplying input accelerations to the shake table to such an extent that generated TRS envelop the required response spectra.

Section 3.7B.2.1.6 discusses qualification requirements for certain Category I equipment and components where dynamic testing is necessary to ensure functional integrity.

The methodology used to account for the effects of differential support motion of multisupported equipment is described in Subsection 3.7B.2.1.5. The secondary stresses due to the support motions are combined with the corresponding inertia stresses by the SRSS method.

Also see Section 3.7A.3.1.

3.7B.3.2 Determination of Number of Earthquake Cycles

3.7B.3.2.1 Piping Systems

A total of 50 peak OBE stress cycles are postulated for fatigue evaluation.

3.7B.3.2.2 Other Equipment and Components

To evaluate the number of cycles caused by a given earthquake, a typical BWR building-reactor dynamic model was excited by three different recorded time-histories (May 18, 1940, El Centro NS component, 29.4 sec; 1952, Taft N 69° W component, 30 sec; and March 1957, Golden Gate S 80° E component, 13.2 sec). The modal response was truncated so that the response of three different frequency bandwidths could be studied: 0 to 10 Hz, 10 to 20 Hz, and 20 to 50 Hz. This was done to give a good approximation to the cyclic behavior expected from structures with different frequency content. Enveloping the results from the three earthquakes and averaging the results from several different points of the dynamic model, the cyclic behavior as given in Table 3.7B-2 was formed.

Independent of earthquake or component frequency, 99.5 percent of the stress reversals occur below 75 percent of the maximum stress level, and 95 percent of the reversals lie below 50 percent of the maximum stress level.

In summary, the cyclic behavior number of fatigue cycles of a component during an earthquake is found in the following manner:

1. The fundamental frequency and peak seismic loads are found by a standard seismic analysis (i.e., from eigenvalue extraction and a forced response analysis).

2. The number of cycles that the component experiences are found from Table 3.7B-2 according to the frequency range within which the fundamental frequency lies.

3. For fatigue evaluation, 0.5 percent of these cycles are conservatively assumed to be at the peak load, 4.5 percent at or above three-quarter peak. The remainder of the cycles have negligible contribution to fatigue usage.

The SSE has the highest level of response. However, the encounter probability of the SSE is so small that it is not necessary to postulate the possibility of more than one SSE during the 40-yr life of a plant. Fatigue evaluation due to the SSE is not necessary since it is a faulted condition and thus not required by ASME Boiler and Pressure Vessel Code, Section III.

The OBE is an upset condition and, therefore, must be included in fatigue evaluations according to ASME Boiler and Pressure Vessel Code, Section III. Investigation of seismic histories for many plants shows that during a 40-yr life, it is probable that five earthquakes with intensities 10 percent of the SSE intensity, and one earthquake with approximately 20 percent of the proposed SSE intensity, will occur. To cover the combined effects of these earthquakes and the cumulative effects of even lesser earthquakes, 10 peak OBE stress cycles are postulated for fatigue evaluation.

3.7B.3.3 Procedure Used for Modeling

Also see Section 3.7A.3.3.

3.7B.3.3.1 Modeling of Piping Systems

Summary

To predict the dynamic response of a piping system to the specified forcing function, the dynamic model must adequately account for all significant modes. Careful selection must be made of the proper response spectrum curves and proper location of anchors in order to separate Category I from non-Category I piping systems.

Selection of Mass Points

When performing a dynamic analysis, a piping system is idealized as a mathematical model consisting of lumped masses connected by weightless elastic members. The elastic members are given the properties of the piping system being analyzed. The mass points are carefully located to adequately represent the dynamic properties of the piping system. A mass point is located at the beginning and end of every elbow or valve, at the extended valve operator, and at the intersection of every tee. On straight runs, mass points are located at spacings no greater than the span length corresponding to 33 Hz. A mass point is located at every extended mass to account for torsional effects on the piping system. In addition, the increased stiffness and mass of valves are considered in the modeling of a piping system.

Selection of Spectrum Curves

In selecting the spectrum curves to be used for dynamic analysis of a particular piping system, curves are chosen that most closely describe the accelerations existing at the end points and restraints of the system. The procedure employed for decoupling the NSSS recirculation piping systems when establishing the analytical models to perform seismic analysis are as follows:

1. The small branch lines (6-in diameter and less) are decoupled from the recirculation piping systems and analyzed separately, except for the bypass lines around 2RHS*V39A,B, which are not decoupled from the recirculation piping analysis.

2. The stiffness and mass of all the anchors and their supporting steel are large enough to effectively decouple the piping on either side of the anchor for analytic and code jurisdictional boundary purposes. The RPV is very stiff and massive compared to the piping system and, thus, during normal operating conditions, the RPV is also assumed to act as an anchor. Penetration assemblies (flued head fittings) are also very stiff compared to the piping system and are assumed to act as anchors. The stiffness matrix at the attachment location of the process pipe (i.e., RCIC, RHR supply, or RHR return) head fitting is sufficiently high to decouple the penetration assembly from the process pipe. GE analysis indicates that a satisfactory minimum stiffness for this attachment point is equal to the stiffness in bending and torsion of a cantilevered pipe section of the same size as the process pipe and equal in length to three times the process pipe outer diameter.

For a piping system supported at more than two points located at different elevations in the building, the response spectrum analysis is performed using the envelope response spectrum of all attachment points. The worst single floor response spectrum selected from a set of floor response spectra obtained at various floors may be applied identically to all floors provided it envelops the other floor response spectra in the set. Alternatively, the multiple support excitation analysis methods may be used where acceleration time-histories or response spectra are applied to all the piping attachment points.

3.7B.3.3.2 Modeling Equipment

For dynamic analysis, Category I equipment is represented by lumped mass systems that consist of discrete masses connected by weightless springs. The criteria used to lump masses are:

1. The number of modes of a dynamic system is controlled by the number of masses used. Therefore, the number of masses is chosen so that all significant modes are

included. The modes are considered significant if the corresponding natural frequencies are less than the ZPA frequency of the excitation, and the stresses calculated from these modes are greater than 10 percent of the total stresses obtained from lower modes. The number of degrees-of-freedom are taken more than twice the number of modes with frequencies less than 33 Hz.

2. Mass is lumped at any point where a significant concentrated weight is located. Examples are the motor in the analysis of the pump motor stand and the impeller in the analysis of the pump shaft.

3. If the equipment has a free-end overhang span with flexibility significant compared to the center span, a mass is lumped at the overhang span.

4. When a mass is lumped between two supports, it is located at a point where the maximum displacement is expected to occur. This tends to lower the natural frequencies of the equipment, which results in a conservative analysis because the equipment frequencies are in the higher spectral range of the response spectra. Similarly, in the case of live loads (mobile) and a variable support stiffness, the location of the load and the magnitude of support stiffness are chosen so as to yield the lowest frequency content for the system. This ensures conservative dynamic loads since the equipment frequencies are always higher than the frequencies at which the spectral peaks occur. If not, the model is adjusted to give more conservative results.

3.7B.3.3.3 Field Location of Supports and Restraints

The final location of seismic supports and restraints for Category I piping, piping system components, and equipment, including the placement of snubbers, is checked against the drawings and instructions issued by the Engineer. An additional examination of these supports and restraining devices is made to assure that their location and characteristics are consistent with the dynamic and static analyses of the system. The final analyses of the as-built systems are performed as necessary, and the final certified as-built design reports are issued.

3.7B.3.4 Basis of Selection of Frequencies

All frequencies in the range of 0.25 to 33 Hz are considered in the analysis and testing of systems, components, and equipment. These frequencies are excited under the seismic excitation.

If the fundamental frequency of a component is greater than or equal to 33 Hz, it is treated as seismically rigid and analyzed accordingly. Frequencies less than 0.25 Hz are not considered as they represent very flexible structures and are not encountered

in this plant. The frequency range between 0.25 and 33 Hz covers the range of the broad band response spectrum used in the design.

The number of modes used in the dynamic analysis of piping depends upon the number of mass points, dynamic degrees-of-freedom, and cutoff frequency.

The cutoff frequencies used for different dynamic loads are as follows:

1. For SRV and LOCA loads, the cutoff frequency is determined by evaluating the loading and structural characteristics so that the dynamic responses of interest are not significantly affected by the omission of modes with frequencies higher than the cutoff value. To ensure that considered modes include at least 90 percent of the response (in compliance with NUREG-0800, Criterion 3.7.2.II.1.a(5)), the following rules are used:

    a. For systems with fundamental frequency in the direction of excitation less than 20 Hz, the cutoff frequency is 60 Hz.

    b. For systems with fundamental frequency in the direction of excitation greater than 20 Hz, the cutoff frequency is 100 Hz.

2. If hydraulic transients are postulated, the system response will be obtained by direct integration of the equation of motion, rather than modal integration.

3.7B.3.5 Use of Equivalent Static Load Method of Analysis

When the natural frequencies of a system, component, or equipment are unknown, it may be analyzed by applying an equivalent static coefficient analysis. This procedure allows a simpler technique for added conservatism. The static acceleration of a component is conservatively assumed to be the peak spectral acceleration of the RRS which envelops the multisupport input spectra. The oscillation damping associated with the enveloping RRS must be representative of the actual component damping.

The equivalent static acceleration is then obtained by multiplying the static acceleration by a static coefficient, C_s, which takes into account the effects of both multifrequency excitation and multimode response. For verifying the structural integrity of frame-type components physically similar to beams and columns, the static coefficient, C_s, is taken as 1.5. For equipment having other than a frame-type configuration, justification is provided for the static coefficient used.

The equivalent static forces on each subcomponent of the equipment are obtained by multiplying the subcomponent masses by the equivalent static acceleration. The resulting static load

vector is distributed over the equipment in a manner proportional to its mass distribution. The static stress analysis is then performed in a normal manner.

3.7B.3.6 Three Components of Earthquake Motion

3.7B.3.6.1 Response Spectrum Method

The total seismic response is predicted by combining the response calculated from the two horizontal and the vertical excitations. When the response spectrum method is used, the methods for combining the responses due to the three orthogonal components of seismic excitation are as follows:

$$R_i = \left[\sum_{K=1}^{3} R_{ij}^2 \right] 1/2$$

(3.7B-20)

Where:

R_{ij} = Maximum, coaxial seismic response of interest (e.g., displacement, moment, shear, stress, strain) in directions i (due to earthquake excitation) and j (j = 1,2,3)

R_i = Seismic response of interest in i direction for design (e.g., displacement, moment, shear, stress, strain) obtained by the SRSS rule to account for the nonsimultaneous occurrence of the R_{ij}'s

3.7B.3.6.2 Time-History Method

When the time-history method of analysis is used, the time-history responses from each of the three components of the earthquake motion are combined algebraically at each time step. The maximum response is obtained from this combined time solution. The earthquake motions specified in the three different directions are verified to be statistically independent.

3.7B.3.7 Combination of Modal Responses

The requirements of RG 1.92 are satisfied as follows. In a response spectrum modal dynamic analysis, if the modes are not closely spaced (i.e., if the frequencies differ from each other by more than 10 percent of the lower frequency), the modal responses are combined by the SRSS method as described in Section 3.7B.3.7.1 and RG 1.92. If some or all of the modes are closely spaced, a double sum method (Section 3.7.3.7.2) is used to evaluate the combined response. The use of the time-history analysis method precludes the need to consider closely-spaced modes.

3.7B.3.7.1 Square Root of the Sum of the Squares Method

Mathematically, the SRSS method is expressed as follows:

$$R = \left(\sum_{i=1}^{n} (R_i)^2 \right)^{1/2}$$

(3.7B-21)

Where:

R = Combined response

R_i = Response due to mode i

n = Number of modes considered in the analysis

3.7B.3.7.2 Double Sum Method

This method, as defined in RG 1.92, is expressed mathematically:

$$R = \left(\sum_{k=1}^{N} \sum_{s=1}^{N} \left| R_k \, R_s \right| \varepsilon_{ks} \right)^{0.5}$$

(3.7B-22)

Where:

R = Representative maximum value of a particular response of a given element to a given component of excitation

R_k = Peak value of the response of the element due to mode k

n = Number of significant modes considered in the modal response combination

R_s = Peak value of the response of the element attributed to mode s

Where:

$$\varepsilon_{ks} = \left[1 + \left\{ \frac{\left(\omega_k^{'} - \omega_s^{'} \right)}{\left(\beta_k^{'} \omega_k + \beta_s^{'} \omega_s \right)} \right\}^2 \right]^{-1} \qquad (3.7B\text{-}23)$$

In which:

$$\omega_k^{'} = \omega_k \left[1 = \beta_k^2\right]^{0.5}$$

$$\beta_k^{'} = \beta_k + \frac{2}{t_d \omega_k} \qquad (3.7B\text{-}24)$$

Where:

ω_k = Modal frequency in mode k

β_k = Damping ratio in mode k

t_d = Duration of the earthquake

3.7B.3.8 Analytical Procedure for Piping

The analytical procedures for piping analysis are described in Sections 3.7B.2.1.5 and 3.7B.3.3.1.

3.7B.3.9 Multiply Supported Equipment Components with Distinct Inputs

The procedure and criteria for analysis are described in Sections 3.7B.2.1.5 and 3.7B.3.3.2.

3.7B.3.10 Use of Constant Vertical Static Factors

Constant vertical static factors in analysis for subsystems and components are used as described in Section 3.7B.3.5.

3.7B.3.11 Torsional Effects of Eccentric Masses

Torsional effects of eccentric masses are included for Category I subsystems (Section 3.7B.3.3.1).

3.7B.3.12 Buried Category I Piping Systems and Tunnels

See Section 3.7A.3.12.

3.7B.3.13 Interaction of Other Piping with Category I Piping

When other (non-Category I) piping is attached to Category I piping, the other piping is analytically coupled sufficiently so as not to significantly degrade the accuracy of the analysis of the Category I piping. Furthermore, the other piping is designed to withstand the SSE sufficiently to prevent failure of the Category I piping.

3.7B.3.14 Seismic Analysis for Reactor Internals

The modeling of the RPV and internals is discussed in Section 3.7B.2.3.2. The damping values are given in Table 3.7B-1. The seismic model is shown on Figure 3.7B-2, and a summary of loading

conditions (including seismic and hydrodynamic), evaluation criteria, calculated maximum stresses in the selected locations, and the allowable stresses is given in Table 3.9B-2.

3.7B.3.15 Analysis Procedures for Damping

Analysis procedures for damping are discussed in Section 3.7B.2.15.

3.7B.4 Seismic Instrumentation

See Section 3.7A.4.

3.7B.5 Reference

1. Liu, L. K. Seismic Analysis of the Boiling Water Reactor, Symposium on Seismic Analysis of Pressure Vessel and Piping Components, First National Congress on Pressure Vessel and Piping, San Francisco, CA, May 1971.

TABLE 3.7B-1
(Sheet 1 of 1)

CRITICAL DAMPING RATIOS FOR DIFFERENT MATERIALS

Item	Percent Critical Damping OBE Condition	SSE Condition
Welded structural assemblies with friction connections	≤3	4.0
Bolted steel frame structures with friction connections	≤3	4.0
Equipment	2.0	3.0
Bolted or riveted structural assemblies with bearing connections	≤5	7.0
Vital piping systems		
Diameter greater than 12 in	≤3*	≤4*
Diameter less than or equal to 12 in	≤3*	≤4*
Reactor pressure vessel, support skirt, shroud, shroud head/separator	2.0	4.0
CRD housings and guide tubes	1.0	2.0
Fuel assembly	6.0	6.0
CRD support springs, shroud support spring, and stabilizer	2.0	4.0
Primary containment	4.0	7.0
Shield wall	4.0	7.0
Pedestal	4.0	7.0

* An alternate approach specified in RG 1.61 Revision 1, Section C.2, Piping Damping, may also be used with the associated restrictions.

TABLE 3.7B-2
(Sheet 1 of 1)

NUMBER OF DYNAMIC RESPONSE CYCLES EXPECTED
DURING A SEISMIC EVENT

Frequency Band (Hz)	0-10	10-20	20-50
Total number of seismic cycles	168	359	643
No. seismic cycles - 0.5% cycles between 75 and 100% of peak loads	0.8	1.8	3.2
No. seismic cycles - 4.5% cycles between 50 and 75% of peak loads	7.5	16.2	28.9

3.8 DESIGN OF SEISMIC CATEGORY I STRUCTURES

3.8.1 Concrete Containment

The concrete containment structure is designed to house the reactor vessel and RCPB and is part of the containment system whose functional requirement is to control the release of radioactivity. The major components of this pressure suppression-type containment system are the primary containment steel liner and the primary containment concrete structure. This section describes the structural design considerations for the primary containment.

Since the original design of this containment preceded the issuance of ASME Section III, Division 2, the reinforced concrete primary containment is designed and constructed to the requirements of the American Concrete Institute, Building Code Requirements for Reinforced Concrete ACI 318-71. Except for regions around penetrations that are designed to meet the requirements of ASME Section III, Division 2, the primary containment steel liner is designed following the requirements of ASME Section III, Division 1.

3.8.1.1 Description of the Containment

The primary containment is a reinforced concrete structure that consists of a drywell chamber located above a suppression chamber, and a drywell floor which separates the drywell chamber from the suppression chamber. The primary containment structure is supported on a 10-ft thick reinforced concrete mat which also supports the reactor building. A series of 24-in diameter downcomer vent pipes penetrates the drywell floor.

The drywell is a steel-lined reinforced concrete vessel in the shape of a frustum of two cones. It is enclosed at the top by a drywell head dome. The steel liner is attached to the inside face of the wall and functions primarily as a leak-tight membrane. The inside diameter of the drywell is 91 ft at the drywell floor level (el 240 ft 7 in) and 34 ft at the top of the primary containment (el 326 ft 10 in).

The suppression chamber is a stainless clad steel-lined cylindrical shell with an inside diameter of 91 ft. It is located directly below the drywell and is supported on a 10-ft thick reinforced concrete mat at el 175 ft. The suppression chamber contains a large reservoir of water called the suppression pool, which serves as a heat sink to absorb energy released into the suppression pool as a result of SRV blowdown or a LOCA.

The suppression pool is composed of an inner and outer pool, the inner pool being located inside the cylindrical reactor pedestal and the outer pool being located between the pedestal outside diameter and the primary containment wall. The inner and outer

pools are connected by six vent openings located in the reactor pedestal wall.

The drywell floor is a 4-ft thick annular reinforced concrete slab separating the drywell from the suppression chamber. It is anchored at the reactor support pedestal and the containment wall. Its primary function is to separate the drywell from the suppression pool. It also provides the primary support for the downcomer vent lines and also supports other penetrations and embedments (Section 3.8.3).

The transfer of loads from the drywell floor to the primary containment concrete wall (Figure 3.8-1) is made through thickened liner sections. The force in the reinforcing steel is transmitted to the liner by Cadweld sleeves which are attached, in line, to each side of the thickened liner plate (Figure 3.8-1). Continuity is thus provided to the reinforcing steel without perforating the liner boundary.

The primary containment wall contains penetrations for process piping, instrument piping, and electrical conductors. It supports floor beam seats, and supports embedments for pipe restraints, pipe supports, conduit, and duct supports. Five access hatches and one airlock penetrate the containment wall and provide for personnel and equipment egress (Section 3.8.1.1.2). One of the larger hatches contains an additional airlock as described in Section 3.8.1.1.2.

The containment wall is designed to withstand anticipated loads without participation of the liner as a structural component.

3.8.1.1.1 Reinforcing Steel Arrangement

The main reinforcing steel in the primary containment wall consists of inside and outside layers of hoop and meridional reinforcement and diagonal reinforcement. The diagonal reinforcement is placed in two orthogonal directions near the outside wall face to form a helix with an angle of 45 deg from the vertical axis of the shell. Hoop and longitudinal tension forces along with the tangential shears will be resisted by the hoop, meridional and diagonal reinforcing steel.

To resist the large radial shear near the base of the wall, flat steel bars inclined at 45 deg to the horizontal, and welded to the vertical reinforcing steel, are installed (Figure 3.8-3). The welded flat bars are terminated at a level above the mat, and single-leg radial shear reinforcing steel is placed as required by the ACI-318 code.

Supplementary reinforcing steel, normal to the face of the wall, is provided in the lower portion of the containment structure wall to resist splitting of the concrete in the vertical plane. Minimum concrete cover for all principal reinforcing steel of the

primary containment structure either equals or exceeds the requirements of ACI-318-71.

Figures 3.8-3 and 3.8-3a show typical details of reinforcing steel in the containment wall. Section 3.8.1.4 describes the reinforcing steel arrangement at hatch openings, as shown on Figure 3.8-4.

3.8.1.1.2 Steel Liner and Penetrations

Steel Liner

Except at various penetrations and access openings through the walls, the primary containment liner is a continuous steel membrane, backed by reinforced concrete. The function of the primary containment liner is to act as a leak-tight membrane to provide a barrier to the release of fission products.

Generally, the liner is 3/8-in thick except at discontinuities such as corners, penetrations, and beam seat and pipe restraint attachment regions, where it is adequately thickened. In the drywell, the steel liner plate material is carbon steel. In the suppression chamber it is carbon steel clad with stainless steel, except at embedments or penetrations which are stainless steel throughout. The portion of the primary containment liner that functions as the suppression pool floor is 1/4 in thick except in regions where load transfers through the floor require that it be thicker. The 1/4-in thick floor liner is welded to the wall liner through a corner junction embedment. It is also welded to the reactor pedestal embedment plates, the SRV T-quencher embedments, and the bridging bar seam embedments that anchor the membrane to the reinforced concrete reactor building floor mat. Approximately 12 in of insulation concrete is installed over the floor liner to protect it from the thermal effects of the DBA and to minimize the corresponding loads on the concrete containment wall. Also, an additional liner is provided as described below.

Other Liners Inside the Primary Containment

An additional liner is provided over the insulation concrete to serve as a waterproof membrane for that concrete. This floor insulation liner is stainless steel as is all other suppression pool steel in contact with its atmosphere. This liner is 1/8 in thick and is anchored to the insulation concrete through a rectangular grid of channel and headed concrete anchors, and is anchored to the 18 T-quencher supports, the primary containment liner, and the reactor pedestal liner. It is installed after the concrete is poured.

The reactor pedestal (Section 3.8.3) is also lined with stainless steel. This liner serves as a concrete form during construction and functions as a waterproof membrane for the pedestal concrete during plant operation. It is anchored to the concrete by headed concrete anchors.

The bottom surface of the drywell floor is formed with corrugated stainless steel and the top is covered with carbon steel liner. The stainless corrugated steel on the bottom surface functions as a concrete form. The steel liner on the top surface functions as a positive gas-tight membrane between the drywell and the suppression chamber to ensure that steam can enter the suppression chamber only through the SRV vent lines or the downcomers. This liner is anchored and seal-welded to the pedestal wall, the primary containment liner knuckle, and all of the drywell floor penetrations and embedments.

Attachments to the Primary Containment Wall

The loads from the structural equipment supports inside the containment are transferred directly to the concrete wall through thickened and stiffened liner insert embedment plates. Loads are distributed on these insert plates through gussets and shear bars which are welded on the back side in an orthogonal grid pattern to provide stiffness to the plate and to provide a means to transfer shear loads to the concrete. The liner insert plates are attached to the concrete wall by either standard headed concrete anchors or larger fabricated anchors, depending upon the load to be transferred. The fabricated anchors are made from 1 1/2-in plate stock and are generally 3 in wide and 18 to 24 in long with a head or flat plate welded at the end to develop the concrete shear cone necessary for load transfer directly to the concrete. This ensures that the loads are not transferred to the steel liner.

Equipment and floor loads are transferred to the wall by floor beam seats, pipe restraint seat/embedments, pipe support embedments, and overlay plates.

Floor beam seat embedments are provided to support the structural steel framing inside the containment. A typical design is shown on Figure 3.8-5.

Pipe restraint embedments are provided to transfer piping restraint loads from postulated accidents. They can either be of a design similar to the floor beam seats or of a design that accepts the welding of a restraint beam to the embedment. A typical pipe whip restraint embedment is shown on Figure 3.8-5.

The reactor stabilizer embedments are provided to transfer the load from the reactor stabilizer structure (the star truss) to the concrete wall.

Pipe support embedments are provided to transfer moderate pipe and duct support loads to the wall.

When the loads from the equipment supports are comparatively small, overlay pads are attached to the liner to distribute and transfer loads through the liner and the liner anchorage into the concrete wall.

Anchorage

The primary containment liner is in intimate contact with the concrete wall through a series of anchor studs welded to the back side of the liner and embedded in concrete on a controlled nodal point pattern (Section 3.8.1.4.2 and Figure 3.8-1). The size of these anchor studs is 5/8 in diameter by 6 1/2 in long. The anchor studs have been designed to accommodate the loads due to both operating and postulated accident conditions.

The primary containment suppression pool floor liner at el 175 ft is anchored to the mat through a pattern of 1/2- by 7-in bridging bars which are embedded on edge and located at each floor liner plate seam (Figure 3.8-2). The bridging bars employ fabricated anchors to anchor the insulation concrete as well as the floor liner to the mat.

Equipment Hatch

A 13 ft 3-in diameter equipment hatch (Figure 3.8-6) is provided in the drywell at el 266 ft 4 in and azimuth 315 deg to service the 261 ft floor and nearby areas during reactor shutdown. The hatch barrel has a floor at el 261 ft and has a bolted-flange type closure mounted outside the containment which contains a double O-ring seal with a leak test tap between the O-rings to accommodate the periodic Type B leak rate test requirements. The hatch barrel is 5/8-in thick and the thickness increases to 3 in in the region of the closure. The hatch cover is a 1 1/2-in thick spherical cap and is bolted to the hatch barrel with 1 1/4-in diameter swing bolts.

Combination Equipment Hatch and Personnel Airlock

A combination equipment hatch and personnel airlock (Figure 3.8-6) is provided in the drywell at el 266 ft 4 in and azimuth 135 deg to service the 261 ft floor and nearby areas located on the opposite side of the drywell from the 315-deg equipment hatch. It provides access for large equipment during reactor shutdown and access for personnel when required. The equipment hatch geometry is the same as the hatch at azimuth 315 deg except that the hatch cover that contains the personnel airlock is conical.

The personnel airlock is located in and welded to the hatch cover. It is 9 ft 7 1/2-in outside diameter, 15 ft 3 in long, and contains a 3 ft 6-in by 6 ft 8-in door at each end. The doors are located in reinforced bulkheads and each is sealed with a double-gasket compression seal with a leak test tap for leak rate testing between the seals. Both doors can be leak rate tested from outside the containment. The mechanical and electrical bulkhead penetrations also have provisions for leak rate testing. Both doors swing inward so as to seat under the positive accident pressure. To ensure containment integrity, they are mechanically interlocked so that if one door is open,

the other cannot be activated. Each door can be closed automatically from outside the opposite door to facilitate entry and egress from the containment when the opposite door has been left open by the last person to enter or exit. The doors are equipped with valves for equalizing air pressure prior to opening. Limit switches are provided on the doors to enable their operation to be monitored from outside each door and in the control room.

Escape Airlock

A 5 ft 6-in diameter emergency airlock (Figure 3.8-6) is provided in the drywell at el 264 ft 9 in and azimuth 236 deg. This airlock serves as a personnel alternate escape route to the main airlock should the need arise. It also provides additional service accessibility during shutdown. It is similar to the personnel airlock in that it also has interlocked doors, equalizing valves, and double-gasket compression seals have provisions for leak testing from outside the containment. Door operation is manual. Limit switches are provided on the doors to enable their operation to be monitored from outside each door and in the control room.

Suppression Chamber Access Hatches

Two 3 ft 9-in diameter hatches provide access to the suppression chamber (Figure 3.8-7). The hatches are located at el 225 ft and at azimuths 130 deg 30 min and 310 deg 30 min. Each access opening has a bolted steel flanged closure mounted outside the containment structure. Each hatch has a double O-ring seal with a leak test tap provided between the seals for periodic testing.

Control Rod Drive Removal Hatch

A 2 ft 1-in diameter CRD removal hatch provides access for the CRD assemblies in the drywell at a centerline location of el 263 ft and azimuth 221 deg (Figure 3.8-7). The hatch has a bolted steel closure mounted outside the containment structure. It has a double O-ring seal with a leak test tap provided between the seals for periodic testing.

Drywell Head

The drywell has a removable 34-ft diameter top cover called a drywell head (Figure 3.8-8), which provides accessibility to the top of the reactor during refueling. It is a freestanding 1 1/8-in thick torispherical head that utilizes a finger pin quick-disconnect closure joint designed as shown on Figure 3.8-8. The finger pin connection represents an improvement over earlier bolted flanges in that stress discontinuities are reduced, and it results in saving time required to remove and install the head. The head is welded to a cylindrical skirt that is, in turn, welded to a thickened joint closure. The upper portion of the joint closure fits inside the U-shaped bottom closure.

Forty-eight 3-in diameter horizontal pins engage the top and bottom to join the mating parts as shown on Figure 3.8-8. The flange design incorporates a flat-faced, double O-ring seal with a leak test tap between the O-rings to accommodate the Type B leak rate test requirements described in Section 6.2.6.

Penetrations

Services and communications between the inside and outside of the primary containment are effected through penetrations. All penetrations consist of a basic wall insert pipe or sleeve that is welded to a liner reinforcing plate and any additional items such as pressure piping, insulation, electrical, or mechanical components required for the individual service.

There are three basic types of penetrations: electrical, instrument, and piping. Electrical penetration sleeve assemblies (Figure 3.8-9) consist of a seamless pipe wall insert sleeve that is welded to a liner reinforcing plate. The sleeve design is similar to the thermally cold piping penetration discussed below. A weld neck flange is welded to the outside end of the sleeve to accommodate the electrical penetration bolted flange. Sealing is effected by a double O-ring seal and a leak test tap is provided between the O-rings to accommodate the Type B leak rate tests.

The basic instrument-type penetration (Figure 3.8-9) is similar to the thermally hot piping penetrations described below in that the process pipe portion is located inside the wall insert sleeve. The process pipe is socket-welded to a transition forging that is in turn welded to the outboard end of the wall insert sleeve. The instrument-type penetration includes: instrument penetrations, piping penetrations for the CRD system, and mechanical penetrations for the TIP drive. Also there is a series of temperature-monitoring instrument penetrations (Figure 3.8-9) located symmetrically around the outer wall of the suppression pool. The wall insert is a heavy wall stainless steel pipe that is welded to the liner reinforcing plate. A special stainless thermowell tip is welded to the end of the sleeve to accommodate the temperature sensor.

Piping penetrations fall into two categories: thermally hot and thermally cold (Figure 3.8-10). Thermally hot penetrations are provided when the design temperature exceeds 200°F. For thermally hot penetrations the wall insert pipe acts as a sleeve. For thermally cold penetrations the wall insert serves as a section of the process piping. For thermally hot penetrations the annular space between the wall insert pipe and the process pipe provides thermal protection to the wall. Each thermally hot penetration is designed with sufficient space between the insert pipe and the process pipe for the required pipe insulation to ensure that the temperature of the concrete in contact with the sleeve remains within 200°F for normal conditions. However, a short term maximum of 350°F is allowed for accident conditions. The process pipe portion of the penetration assembly is a

continuous forging. It is attached to the sleeve through a flued-head transition that is an integral part of the forging (Figure 3.8-10).

Provisions for a heat exchanger were originally designed into each hot penetration sleeve but these were found to be unnecessary considering the geometry, insulation, and current temperature limits.

All penetrations are designed to be capable of accommodating the stresses imposed on them by the design conditions. The design is discussed in detail in Section 3.8.1.4.2.

3.8.1.2 Applicable Codes, Standards, and Specifications

The design and construction of the primary containment, steel liner, equipment and personnel access hatches, and penetrations meet or exceed the requirements of the codes, standards and regulations listed in Section 3.8.4.2 for the design of Category I structures.

3.8.1.3 Loads and Load Combinations

3.8.1.3.1 Primary Containment Structure

The reinforced concrete structure of the primary containment is designed to withstand the loadings and stresses anticipated during the 40-yr operating life of the unit which arise from normal operation and other postulated loads such as earthquake, LOCA, jet impingement, and suppression pool hydrodynamic loads as defined in Table 3.8-1. The steel liner which is attached to and supported by the concrete also transmits loads to the concrete. The load combinations for which the structure is designed are defined in Table 3.8-1.

Design load criteria conform to current containment design practice. The criteria contain varying load factors for combining dead, pressure, temperature, and earthquake forces. The total loading resulting from the summation of any one of the combinations may cause a maximum stress condition depending on the type of stress and member under consideration.

The primary containment is designed to withstand the applicable loads and reaction forces due to postulated pipe rupture (Section 3.6A) in combination with other loads. The primary containment is also designed for the post-accident environments.

3.8.1.3.2 Steel Liner and Penetrations

Table 3.8-2 lists the load and symbol definitions for the following portions of the primary containment liner.

<u>Wall Liner, Floor Liner, and Embedments</u>

The wall liner, floor liner plate, and corner transition section are designed for the loads and load combinations described in Table 3.8-2 so that either the resulting stress levels do not exceed the allowable limits given in ASME Section III, Division 1, Subsection NE, 1971 Edition through 1973 Summer Addenda, or that the resulting strain levels do not exceed the allowable strain levels given in ASME Section III, Division 2, Subsection CC-3700, 1975 Edition. The welded attachments to the primary containment liner are designed to meet the requirements of Subsection NE, 1974 Edition, with Winter 1976 Addenda and Code Case N-224-1. The liner is shielded from the application of internally-generated concentrated missile loads (Section 3.5). Jet impingement effects from postulated pipe rupture inside primary containment are applied to the liner. Structural attachment positions of the liner embedments that are beyond the scope of ASME III, Division 1, are designed using the load combinations for structural steel design (Section 3.8.4.3).

Access Locks and Hatches

The equipment hatch, equipment hatch/personnel airlock, escape airlock, suppression pool hatches, and CRD removal hatch are subjected to the loads and load combinations described in Table 3.8-3.

Drywell Head

The drywell head is subjected to the loads and load combinations described in Table 3.8-4.

Penetrations

Load combinations and stress allowables for the penetrations are summarized in the following tables:

ASME III Safety Class 1 piping	Table 3.8-5
ASME III Safety Class 2 piping	Table 3.8-6
ASME III Class MC components	Table 3.8-7

3.8.1.4 Design and Analysis Procedure

3.8.1.4.1 Concrete Primary Containment Structure

The reinforced concrete primary containment structure is analyzed for the various loading conditions using the SHELL 1 computer code (Appendix 3A) for thin shells of revolution that are symmetrically as well as asymmetrically loaded. The effects of dead and live loads, internal pressure, temperature, earthquake, pipe rupture, and hydrostatic loads are considered. For the purpose of analysis and design, the concrete is assumed to be fully cracked in the hoop direction over the entire height. In the vertical direction, the concrete is also assumed to be fully

cracked everywhere except for a small region adjacent to the foundation mat. The stiffness properties of the wall are calculated accordingly.

Additional reinforcing steel is provided around penetrations and hatches in order to minimize the effects from localized load concentrations and to provide continuity of the load-carrying capability of the wall. These local effects, which do not affect the overall analysis, are considered on an individual basis for design.

After the internal loads in the shell are determined, the reinforcing steel is checked for adequacy. Special attention is given to the effects of shear forces in the shell, with additional shear reinforcing being provided as required. As a result, diagonal shear bars are used to resist radial shear at the top of the upper cone, the junction of the two cones, the cone-to-cylinder junction, and at the base of the wall adjacent to the foundation mat. Typical arrangement of reinforcing is shown on Figure 3.8-3.

Certain areas of the wall require special analysis due to geometric conditions. One region requiring special analysis is the top of the concrete containment. The design of this area is based on a supplementary analysis using a finite element approach, which addresses the geometric and nonlinear material effects, i.e., concrete cracking. The particular model used for this analysis is extended well into the membrane zone of the shell so that membrane conditions are assumed at the boundaries. Figure 3.8-3 shows the reinforcing detail in this area.

Another area that requires special analysis using the finite element approach is in the vicinity of the hatches. Typical reinforcing in the area of the equipment hatch is shown on Figure 3.8-4.

These regions in the wall are analyzed by means of the three-dimensional, finite element capability of the computer program STRUDL II (Appendix 3A). Because of geometrical symmetries in the area of the hatches, the finite element models selected for these analyses are semicircular. In order to eliminate the effects of the hatch opening and thickened ring beam at the grid's outer boundary, the semicircular finite element grid is assumed to have a maximum radius approximately equal to 2.5 times the radius to the outer edge of the thickened ring beam. A geometry of quadrilateral shapes emanating radially from the center of the opening is used to provide a fine grid in the meridional and circumferential shell directions. Three elements are taken through the typical wall section, with additional elements added on each side of the typical wall section to account for the thickened ring beam section.

The STRUDL II program allows the modeling of the structural characteristics of the wall (cracked versus uncracked) and the

inclusion of the liner for the various loadings including temperature. During an accident the liner will normally be in a state of compression due to the sharp temperature rise within the containment structure, and therefore will add load to the ring beam. The liner, however, is assumed not to contribute to the structural capacity of the ring beam or wall for any loading condition other than the primary containment structural acceptance test.

To obtain a more realistic assessment of the strains, displacements, and stresses in the area of the hatches, the ring beam and cylindrical wall are assumed to be fully cracked and to have only the stiffness of the reinforcing bars.

The STRUDL II program yields both the discontinuity effects between the cylinder wall and the thickened ring beam and the pattern of membrane forces in the region of the hatch openings. Additional reinforcement (circumferential, meridional, and diagonal) is provided in regions where a significant increase over the typical membrane forces is observed. The principal circumferential and meridional reinforcing bars are extended to the inner face of the ring beam and are either bent at right angles or Cadwelded to each other.

Seismic analysis of the primary containment structure provides the acceleration to which the primary containment would be subjected during an earthquake. These accelerations, when multiplied by the associated masses, are applied as static loads to the structure. Tangential shears caused by asymmetric loads are resisted by the concrete and the steel reinforcement in the wall. Strains, compatible with the effects of internal pressure-induced stresses, are assumed in calculating the reinforcing steel requirements for earthquake loads. The steel liner plate is conservatively assumed to provide no shear capacity to the wall section.

3.8.1.4.2 Steel Liner and Penetrations

<u>Steel Liner</u>

The liner was analyzed using the computer program KALNINS. The orthotropic capability is used to model the reinforcing bar array as an equivalent orthotropic shell. Temperature variations within the containment are considered by applying an equivalent pressure that satisfies the equilibrium equations of the liner and concrete vessel. The primary containment is conservatively assumed to be completely cracked to obtain the maximum liner deformation. The boundary condition at the wall-to-mat junction is assumed to be completely fixed to obtain the maximum liner stress intensity range.

The liner structural integrity against buckling is obtained from the results of an analysis performed using the ANSYS computer

program. Results show that the anchor studs have a safety factor of at least 2.0 against progressive failure.

Floor Liner Plate and Corner Transition Section

The outer edge of the suppression pool floor liner plate and the corner transition section are analyzed using the computer code SHELL 1 (Appendix 3A) for thin shells of revolution. The liner model extends to the first row of wall liner plate anchor studs. At this anchor stud location, the forces obtained from the containment liner analysis are imposed on the corner junction.

Temperature distributions following a DBA are calculated using the finite difference computer code TAC-2D (thermal analysis code - two-dimensional) (Appendix 3A). The temperatures determined by TAC-2D are used as input for SHELL 1 to determine thermal stresses.

Hot Piping Penetrations (Sleeved)

The hot (sleeved) piping penetration assemblies are analyzed for stress using the finite element computer code ASAAS (asymmetric stress analysis of axisymmetric solids) (Appendix 3A). This program is capable of evaluating the effects of symmetric and asymmetric loads as well as pressure and temperature loads.

Temperature distributions in the penetrations are evaluated using TAC-2D. The temperatures as determined by TAC-2D are used as input for ASAAS to determine thermal stresses. For the ASME Code Class 1 penetrations, pressure and temperature transients are analyzed for all modes of operation. For the ASME Code Class 2 penetrations, only steady-state temperatures are considered. The stresses are limited to the allowables in Table 3.8-5 for Code Class 1 piping penetrations and Table 3.8-6 for Code Class 2 piping penetrations. For the portions of the penetrations that form part of the primary containment boundary and are classified MC, the stresses are limited to the allowables in Table 3.8-7.

Cold Piping Penetrations (Unsleeved)

The cold (unsleeved) piping penetrations through the primary containment are analyzed using the program ASAAS. Stress concentration factors for the junction of the pipe and the containment wall are developed using this program. The allowable stresses for these Safety Class 2 penetrations are given in Table 3.8-6.

Beam Seat and Pipe Restraint Embedments

The embedments for beam seats and pipe restraints are analyzed for the loads imposed by the design and operating conditions. The anchor-concrete interface pattern is assumed analogous to a reinforced concrete beam, and anchor loads and stresses in reinforcing steel are determined. The front face of the

embedment is designed by conventional steel design methods using AISC Specification for Design, Fabrication and Erection of Structural Steel for Buildings. The anchor concrete interface is designed to the guidelines outlined in ACI-318-71.

Access Openings

The equipment hatches, suppression pool access hatches, and CRD removal hatch are analyzed using classical equations and the ANSYS finite element computer program (Appendix 3A) for axisymmetric shells. Where required, natural frequencies are determined and seismic loadings are applied accordingly. The suppression pool and CRD hatches are assumed to be rigidly supported by the concrete wall. The equipment hatch barrels are supported by the concrete around a 90-deg arc at the bottom and rigidly restrained at the liner reinforcing plate-to-barrel junction.

Analysis of the escape lock and the personnel lock is performed using hand calculations and by the NASTRAN (Appendix 3A) computer program. The escape lock is assumed to be rigidly restrained by the concrete wall. The personnel lock is assumed to be supported by the equipment hatch cover welded to its barrel. The NASTRAN finite element models use bar and plate elements for analysis of the bulkheads.

Drywell Head

The drywell head is analyzed as a finite element shell of revolution using axisymmetric conical shell elements. The structural analysis program used is ANSYS (Appendix 3A). The analysis is performed for pressure, deadweight, seismic, thermal, and jet impingement loadings.

The finger pin closure is analyzed using the same ANSYS program used for the drywell head. It is also modeled as an axisymmetric shell and includes a portion of the shell above and below the closure. The finger pins which join the upper and lower closure fingers are modeled as two equivalent rings in the two 1/8-in gaps between the fingers. Loads obtained from the head analyses are applied at the upper boundary of the closure analysis. The concrete is conservatively assumed to provide a fixed support at the lower boundary of the straight portion of the shell at the top of the containment.

3.8.1.5 Structural Acceptance Criteria

3.8.1.5.1 Primary Containment Structure

The design of the primary containment structure follows ACI-318-71. The basic criterion for concrete strength design is expressed as:

Required strength ≤ calculated strength

All members of the primary containment structure are proportioned to meet this criterion. The required strength is expressed in terms of design loads or their related internal moments and forces. Design loads are defined as loads that are multiplied by their appropriate load factors (Section 3.8.1.3). Calculated strength is computed by provisions of the ACI-318-71 Code, including the appropriate capacity reduction factors. The modulus of elasticity of reinforcing steel is 29,000,000 psi. A Poisson ratio of 0.167 is used for reinforced concrete. Tangential shear (v_u) resulting from asymmetric loadings will be resisted by either the concrete or the concrete and steel reinforcing bars.

The maximum allowable stress for tangential shear (v_u) in the concrete alone is 40 psi. The actual and allowable stresses for the containment structure are given in Table 3.8-8.

3.8.1.5.2 Steel Liner and Penetrations

The allowable stresses and strains for the primary containment liner are limited by either the stress criteria given in ASME Section III, Division 1, 1971 Edition through the Summer 1973 Addenda, or the strain criteria given in ASME Section III, Division 2, 1975 Edition. The allowable stresses and strains for the primary containment penetrations are limited by the criteria given in ASME Section III, Division 1, 1971 Edition through the Summer 1973 Addenda. In particular:

1. Wall liner, floor liner, corner junction transition and liner insert plate embedments (Table 3.8-2 and Section 3.8.1.3.2).

2. Access locks and hatches (Table 3.8-3 and Section 3.8.1.3.2).

3. Drywell Head (Table 3.8-4 and Section 3.8.1.3.2).

4. Penetrations (Tables 3.8-5 through 3.8-7 for the ASME Safety Class 1, 2, and MC portions, respectively, and Section 3.8.1.3.2).

5. The minimum allowable stresses for ASTM A500 Grade B tube steel (for welded attachments to the liner) are as specified in Code Case N-224-1.

3.8.1.6 Materials, Quality Control, and Construction Techniques

3.8.1.6.1 Concrete Primary Containment Structure

The materials used in the construction of the primary containment are the same as those used for other Category I structures

(Section 3.8.4.6). The quality control program for the primarycontainment structure meets or exceeds the requirements outlined in Sections 3.8.4.2 and 3.8.4.6 for procurement, fabrication, testing, and construction of Category I structures. There are no special techniques used in the construction of the concrete primary containment structure.

3.8.1.6.2 Steel Liner and Penetrations

Steel Liner

All materials used in the fabrication of the primary containment liner and liner components are in accordance with approved materials listed in Section III Division 1 Appendixes of the ASME Boiler and Pressure Vessel Code for Nuclear Power Plant Components, 1971 Edition through Summer 1973 Addenda. The welded attachments to the liner meet the material requirements of Subsection NE, 1974 Edition, with Winter 1976 Addenda, and Code Case N-224-1. All liner materials are tested and certified in accordance with ASME Boiler and Pressure Code Section II, 1971 Edition through Summer 1973 Addenda. The steel liner is fabricated (but not stamped) in accordance with ASME Section III, Subsection NE. Exceptions are discussed in Section 3.8.1.6.3.

Material selected for construction of the liner is ferritic steel with appropriate ductility. All ferritic materials forming part of the primary containment boundary are Charpy V-notch tested at -10°F (except bolting at +40°F) and conform to ASME Section III, Subarticle NE-2300. The welded attachments to the liner are impact-tested in accordance with ASME Section III, 1974 Edition, with Winter 1976 Addenda, Subarticle NE-2300. All ferritic materials that form part of the pressure boundary and are more than 5/8 in thick, except as noted below under access locks, are also dropweight tested in accordance with Section 15 of ASTM E208 at a set temperature of 10°F or less to verify that the NDTT is equal to or less than 0°F.

All welding procedures and tests required by ASME Section IX are adhered to for the selection of weld filler material, heat treatment, the performance of welding machines, and the qualifying of welding procedures and Welding Operators who construct the primary containment liner, except as noted in Table 1.8-1, RG 1.19, Item 4. The welding qualification includes 180-deg bend tests of weld material. These procedures ensure that the ductility of welded seams will be comparable to the ductility of the containment liner plate material.

Liner surfaces exposed to the suppression chamber atmosphere are made from the following materials: stainless steel plate for embedments, carbon steel plate clad with stainless steel for the liner, and carbon steel plate overlaid with stainless steel sheet for the knuckle. All carbon steel weld seams in the suppression pool wall liner are weld overlaid with stainless steel. Stainless steel plate is furnished by the supplier to the

requirements of SA-240 Type 304L in the solution-annealed condition.

Surfaces of the liner exposed to the drywell atmosphere inside the containment are coated in accordance with RG 1.54. The backside of the liner in contact with the concrete has adequate corrosion protection and is not painted.

The primary containment wall liner plate and suppression pool floor liner plate material is SA-537 Class 2, quenched and tempered with a specified minimum tensile strength of 80,000 psi, a minimum guaranteed yield strength of 60,000 psi, and a guaranteed minimum elongation of 22 percent in a standard 2-in specimen. The wall liner has a 3/8-in nominal base thickness, and the floor liner is 1/4 in thick. The wall liner in the suppression pool is SA-537 Class 2 that has been roll-bonded under heat and pressure with stainless steel clad plate, or has been wallpapered with stainless steel to ensure adequate protection from corrosion. All cladding is SA-240 Type 304L stainless steel made to SA-264, and is 20 percent of nominal base metal thickness with a 0.0375-in minimum thickness and a bond shear strength of 30,000 psi. All wallpapered cladding is 1/8-in thick SA-240 Type 304 stainless steel. Thickened embedment plates inserted into the liner which transfer structural equipment loads to the reinforced concrete in the drywell are 1 1/2-in thick SA-537 Class 2 Lukens Lectrifine to enhance the cross-transverse (Z-axis) properties. Insert plates in the suppression pool are 1 1/2-in thick SA-240 Type 304L stainless steel. Liner cone transitions are thickened plates of 1 1/4-in thick SA-537 Class 2 material. The primary containment liner plates were ordered to conform to mill practice with regard to thickness and tolerance in accordance with ASTM-A20.

The construction tolerances specified for the liner are:

1. The maximum difference in cross-sectional diameters of the steel liner shall be in accordance with ASME Section III, Subparagraph NE-4221.1.

2. The out-of-round tolerance measured at the top of the knuckle or starter plate shall not exceed ±0.25 percent of the nominal inside diameter. The top of the knuckle plate shall be level within ±3/8 in.

3. The maximum deviation from a straight line or from circular form measured anywhere in the liner shall not exceed ±1/4 in in a 14-in span.

4. The cylindrical portion of the liner shall be plumb within 2 3/4 in at any height of the liner measured from an established vertical line extending up from the base of the mat liner. For conical sections, the liner

shall be within 2 3/4 in of basic as a function of elevation.

5. The maximum misalignment between liner plates shall be in accordance with ASME Section III, Paragraph NE-4232.

Nondestructive testing for the steel liner was performed as required by RG 1.19 (or acceptable alternative as described in Table 1.8-1) or the applicable code for noncode-stamped components. Procedures for the steel liner were in accordance with the requirements of ASME Section V, 1971 Edition through Summer 1973 Addenda, except that radiographic examination was also performed to the requirements of the 1977 Edition through the Summer 1978 Addenda. Acceptance standards for radiographic examination are in accordance with ASME Section III, 1971 Edition through Summer 1973 Addenda, or 1977 Edition through Summer 1978 Addenda. For magnetic particle and liquid penetrant examination, the evaluation of indications and the acceptance standards are in accordance with ASME Section III, 1971 Edition through Summer 1973 Addenda. For these examinations conducted after November 1, 1978, the evaluation of indications and acceptance standards is in accordance with Paragraphs NE-5341, NE-5342, NE-5351, and NE-5352 of ASME Section III, 1977 Edition, and Code Case N-339. Acceptance requirements for all other nondestructive testing procedures are in accordance with ASME Section III, 1971 Edition through Summer 1973 Addenda, Subarticle NE-5300.

Leak-tightness testing of all wall and floor liner seam welds was completed during construction. When leakage was detected, repair welding was performed and the weld was retested by the same methods applied to the original weld.

The Cadweld sleeves welded to the liner to accommodate the drywell floor and pedestal interfaces (Section 3.8.1.1.2) were qualified by sister splices which were tensile-tested to failure. Cadweld sister splices were tested to substantiate the structural integrity of the Cadweld sleeve-to-plate joint by demonstrating that failure occurred each time in the rebar. A sister splice plate is composed of material, thickness, and configuration identical to the production piece.

Anchor Studs

Anchor stud material is ASTM A108 Grades 1010-1020, with a minimum tensile strength of 60,000 psi. The anchor studs are welded as an ASME Section IX P-1, Group 1 material. Welding of anchor studs is not performed when the base metal temperature is below 50°F. The areas to which the studs are to be welded are brushed or ground free of scale or rust.

The normal tolerance for locating the centerline of a concrete anchor stud on the 3/8-in liner is 1 1/2 in horizontally and meridionally. Additional anchor studs are added to the liner if interferences exist with reinforcing steel, and are documented to

assure that they do not exceed the minimum spacing requirements. An anchor stud may be bent up to 10 deg to avoid reinforcing steel. However, an anchor stud may be bent up to 20 deg in order to avoid interference with reinforcing steel if no other anchor stud within an 18-in radius is bent more than 10 deg. Any anchor stud that is not acceptable by the preceding criteria will be replaced by locating an additional anchor stud within a 3 1/2-in radius.

For each shift, stud welders were qualified by successfully welding two studs to a sister plate. After the weld cooled, each stud was bent 30 deg by striking the stud with a hammer. If failure occurred in the weld of either stud, the procedure was corrected and two successive studs were successfully welded and tested before work proceeded. Sister plates were of material similar to that of the production piece and had the same thickness as the production piece, except that a 1-in sister plate was acceptable for production items larger than 1 in. All studs were visually inspected. Studs on which a full 360-deg weld flash was not obtained were replaced or repaired as a minimum for compliance to AWS D1.1.

Access Hatches and Locks

The materials used in the construction of the access locks and hatches are in accordance with ASME Section III, Division 1, 1971 Edition through Summer 1973 Addenda, and are tested and certified in accordance with Section II of the above code.

The access hatches, personnel airlock, and escape lock are fabricated but not stamped in accordance with ASME Section III, Subsection NE.

The major components (panel, cover, and flanges) for the suppression chamber hatches are built with SA-240 Type 304L stainless steel with a minimum tensile strength of 70,000 psi and a minimum yield strength of 25,000 psi. The major components (panel, cover, and flanges) of the equipment hatch, equipment hatch/personnel airlock, escape airlock, and CRD hatch are built with SA-516 Grade 70 with a minimum tensile strength of 70,000 psi, and a minimum yield strength of 38,000 psi. SA-516 Grade 70 material 2 1/2 in and larger is used in the hatch closer joints in the quench and tempered condition without dropweight tests. Bolting material is SA-193 Grade B7 and the O-ring is material ethylene-propylene-diene-monomer (EPDM). The plates conform to standard mill practice with regard to thickness and tolerance in accordance with ASTM A-20.

All welding, welding procedure qualification, and welder qualifications are in accordance with ASME Section III. The methods of nondestructive examination of access opening welds are in accordance with ASME Section III, Subarticle NE-5200. Nondestructive examinations are in accordance with RG 1.19 (except as noted in Table 1.8-1, RG 1.19, Item 5) and are

qualified in accordance with the methods and techniques described in ASME Section III, Division 1, Appendix X, 1971 Edition through Summer 1973 Addenda with acceptance requirements in accordance with ASME Section III, Subarticle NE-5300.

<u>Drywell Head</u>

The drywell head is fabricated to the requirements of ASME Section III, Division 1, 1971 Edition through Summer 1973 Addenda. The torispherical head is built with SA-516 Grade 70 with a minimum tensile strength of 70,000 psi, minimum yield strength of 38,000 psi, and a guaranteed minimum elongation of 21 percent in a standard 2-in specimen. The closure flange is SA-240 Type 304. The closure pin material is SA-564 Grade 630 with Code Case 1388-2. The O-ring material is EPDM.

All welding procedures for the fabrication of the drywell head are in accordance with ASME Section III, Division 1, Class MC requirements. Nondestructive testing procedures are qualified in accordance with the methods and techniques as described in ASME Section III, Division 1, Appendix X, 1971 Edition through Summer 1973 Addenda with acceptance criteria according to ASME Section III, Subarticle NE-5300.

The elevation tolerance of the drywell head flange is +2 to 0 in and the levelness tolerance is 1/8 in. Out-of-roundness does not exceed 0.75 in. Variation in closure circumference does not exceed $\pm$0.375 in. The centerline of the installed closure joint with respect to the containment centerline is concentric within a 2-in diameter circle. The maximum gap between the sealing surfaces without the O-rings in place does not exceed 0.031 in after shop fabrication and 0.051 in after field installation.

<u>Penetrations</u>

The penetration pipe sleeves (Class MC boundary) are fabricated (but not stamped) in accordance with ASME Section III, Subsection NE, except for Paragraphs NE-4121 and NE-4125 and Subparagraph NE-4621.1. The process pipes are fabricated and stamped in accordance with ASME Section III, Subsection NB or NC as applicable.

The materials used for the penetration assemblies are discussed below.

<u>Hot Penetrations (Figure 3.8-10)</u> All carbon steel integral flued head forgings forming a portion of a piping system are either SA-105 Grade 2, or SA-508 Class 1, including ASME Code Case 1332-6, except that the integral forgings for the feedwater system are SA-508 Class 2. Stainless steel flued head forgings for applicable piping systems are SA-182 Type F304L. The carbon steel sleeves in the drywell assemblies are SA-333 Grade 6 and SA-106 Grade B, except for the main steam and feedwater sleeves

which are SA-155 Grade CMSH80. Sleeves for the stainless steel forgings are SA-312 Type 304L and SA-240 Type 304L for seamless pipe up to 10 in in diameter, and are SA-312 Type 304 for welded pipes over 10 in in diameter.

The ASME Section III Safety Class 1 and 2 penetration pipe ferritic forging with process pipe regions thicker than 5/8 in will be Charpy V-notch toughness tested for acceptance at +40°F and dropweight tested at 10°F in accordance with ASME Section III Subarticle NB-2300.

All welding, welding procedure qualifications, and welder qualifications for ASME Section III piping Safety Class 1 or 2 components are in accordance with the requirements of ASME Section III. For fabrications of the Class MC portions, the welding requirements are the same as those for the steel plate liner. The methods of nondestructive examination of Safety Class 1 and 2 piping penetration welds are in accordance with ASME Section III Subarticles NB-5200 and NC-5200, respectively. Nondestructive testing procedures are qualified in accordance with ASME Section V with acceptance requirements to Subarticle NE-5300 for the Class MC items.

<u>Thermally Cold Penetrations (Figure 3.8-10)</u> The carbon steel service pipe forming a portion of the piping system is SA-333 Grade 6 or SA-106 Grade B, fine-grained and normalized. Stainless steel service piping for applicable piping systems is SA-312 Type 304L or 316L for pipe sizes up to 10 in diameter and SA-312 Type 304 for pipe sizes over 10 in diameter.

Carbon steel process pipes with thickness greater than 5/8 in are impact tested and dropweight tested as described above under thermally hot penetrations.

<u>CRD and Instrument Penetrations (Figure 3.8-9)</u> The inner pipe forming a portion of the piping system pressure boundary is SA-312 Type 304L. Except in 2CMS*Z61B, piping material is SA-312, TP 304. The forged adapter which also forms part of the piping system boundary, in addition to attaching the inner pipe to the containment sleeve, is SA-182 Type 304L. The containment sleeves in the suppression chamber are SA-312 Type 304L. The containment sleeves in the drywell are SA-333 Grade 6. The requirements of welding and testing (examination) of these items are the same as those described under Thermally Hot Penetrations.

<u>Electrical Penetration Sleeves (Figure 3.8-9)</u> The material for the electrical penetrations in the drywell is SA-333 Grade 6, fine-grained and normalized. The stainless steel electrical penetrations in the suppression chamber are SA-312 Type 304L. The weld neck flange that accommodates the bolted electrical insert is SA-105 in the drywell and SA-182 Type 304L in the suppression chamber.

Primary containment liner reinforcement plates in the drywell are SA-537 Class 2 material. Liner reinforcement plates in the suppression chamber are SA-240 Type 304L.
The meridional slope and the circumferential slope for the penetrations are specified as ±1/2 in at the end of the penetration farthest from the plane of the reinforcement plate-to-penetration weld. The specified slope tolerances for the main steam and feedwater penetrations are ±1 in. Tolerances for service pipe and sleeve wall thicknesses for forged penetrations are in accordance with ASTM A530 except that the maximum thickness is more tightly controlled to accommodate machining operations and to limit eccentricity between the outside and inside diameters.

3.8.1.6.3 Exceptions and/or Clarifications to ASME Code

The exceptions and/or clarifications to the ASME Boiler and Pressure Vessel Code, Division 1, 1971 Edition through Summer 1973 Addenda, are as follows for noncode-stamped items on the containment:

NA-1120	Definition of Nuclear Power System Components and Containment Vessels
NA-3500	Responsibilities of Inspection Agencies Engineering Specialists and Inspectors
NA-4000	Quality Assurance
NA-5000	Inspection
NA-8000	Name Plates, Stamping, and Reports
NE-1110	Aspects of Construction Covered by These Rules
NE-1120	Rules for Class MC Containment Vessels
NE-4121	Means of Certification - NE-4121 has a requirement for the application of the code symbol to the containment structure, but if the containment liner is backed by concrete, it does not qualify as a MC vessel.
NE-4125	Testing of Welding Materials - NE-4125 has a requirement that defines a "lot" of weld material based on heats of metal or dry batch. Code Cases 1567 and 1568 were invoked to define a "lot" of weld material to be based on time and quantity of production runs.
NE-4311	Studwelding Restrictions
NE-4322	Maintenance and Certification of Records

NE-4421 Backing Rings

NE-4429 Welding of Clad Parts - This section requires that any weld-deposited cladding be examined by a liquid penetrant method. In lieu of 100-percent liquid penetrant examination, some categories of weld-deposited cladding on the suppression chamber lower knuckle were visually examined. A small portion of the cladding in one of these categories was inaccessible for visual examination and is accepted based on the satisfactory visual examination results for the accessible cladding.

NE-4621.1 Materials Exempted from Post-Weld Heat Treatment - NE-4621.1 contains a requirement in Subparagraph NE-4621.1(b) to postweld heat treat all subassemblies prior to insertion into the shell. Subparagraph NE-4622.1, which exempts weldments up to 1 1/2-in thick (embedments) from postweld heat treatment, is used in place of Subparagraph NE-4621.1(b) since the liner insert portion of the embedment is tapered down to the liner thickness.

NE-5211 Category A and B Welds

NE-5212 Examination of Embedded Welds of Vessels

NE-5231 Butt Welds - This section requires that for butt welds in nonpressure-retaining structural parts attached to pressure-retaining parts, the portion of the butt weld within 16t, where t is the thickness of the structural part, shall be examined by a radiographic method in accordance with Appendix X. In lieu of the radiographic examination in butt welds within this 16t, the welds will be examined by an ultrasonic method, a magnetic particle method, or a liquid penetrant method all in accordance with ASME V.

NE-5232 Nonbutt Welds in Non-Category Joints - This section requires that for nonbutt welds in nonpressure-retaining structural parts attached to pressure-retaining parts, the nonbutt welds within the limit of 16t, where t is the thickness of the structural part, shall be examined by an ultrasonic method, a magnetic particle method, or a liquid penetrant method, all in accordance with Appendix X. The nonbutt weld of the structural part to the pressure-retaining part will be examined in accordance with the requirements of NE-5232 using the methods of ASME V in lieu of Appendix X. The nonbutt welds in nonpressure-retaining structural parts within the limit of 16t will be examined in accordance with

the governing structural fabrication code requirements.

NE-5522 Verification by Inspector of Personnel Qualification

NE-5610 Vessel Materials - Examination

NE-6000 Testing

NE-8000 Nameplates, Stamping, and Reports

The exceptions and/or clarifications to the ASME Boiler and Pressure Vessel Code, Division 1, 1974 Edition through Winter 1976 Addenda, are as follows for noncode-stamped attachments to the primary containment liner:

NA-1120 Definition of Nuclear Power System Items

NA-3500 Responsibilities of Inspection Agencies, Inspection Specialists, and Inspectors

NA-4000 Quality Assurance

NA-5000 Inspection

NA-8000 Certificates of Authorization, Nameplates, Stamping, and Reports

NE-1110 Aspects of Construction Covered by These Rules

NE-1120 Rules of Class MC Containment Vessels

NE-4121 Means of Certification - NE-4121 has a requirement for the application of the code symbol to the containment structure, but the containment liner is backed by concrete. It does not qualify as a MC vessel.

NE-4125 Testing of Welding Materials - NE-4125 refers to NE-2400 which has a requirement that defines a "lot" of weld material based on heats of metal or dry batch. Code Cases 1567 and 1568 were invoked to define a "lot" of weld material to be based on time and quantity of production runs.

NE-4311.1 Studwelding Restrictions

NE-4322 Maintenance and Certification of Records

NE-4421 Backing Rings

NE-5210 Category A Welds

NE-5220 Category B Welds

NE-5261 Butt Welds - This section requires that for butt welds in nonpressure-retaining structural parts attached to pressure-retaining parts, the portion of the butt weld within 16t, where t is the thickness of the structural part, shall be examined by a radiographic method. In lieu of the radiographic examination in butt welds within this 16t, the welds will be examined by an ultrasonic method, a magnetic particle method, or a liquid penetrant method, all in accordance with ASME V.

NE-5262 Nonbutt Welds in Non-Category Joints - This section requires that for nonbutt welds in nonpressure-retaining structural parts attached to pressure-retaining parts, the nonbutt weld within the limit of 16t, where t is the thickness of the structural part, shall be examined by an ultrasonic method, a magnetic particle method, or a liquid penetrant method, all in accordance with ASME V. The nonbutt weld of the structural part to the pressure-retaining part will be examined in accordance with the requirements of NE-5262. The nonbutt welds in nonpressure-retaining structural parts within the limit of 16t will be examined in accordance with the governing structural fabrication code requirements.

NE-5522 Verification by Inspector

NE-5610 Vessel Materials

NE-6000 Testing

NE-8000 Nameplates, Stamping, and Reports

The above exceptions/clarifications, for the most part, reflect the fact that the containment liner, being backed up by concrete, does not qualify as an ASME Section III Class MC vessel and the examination requirements of RG 1.19 apply. Therefore, it could not be stamped MC and thus the portions of the code that address definitions, stamping, third party inspection, and data reports do not apply.

The exceptions and/or clarifications to the ASME Boiler and Pressure Vessel Code Section III, Division 1, 1974 Edition through Winter 1976 Addenda, for the installation of code-stamped electrical penetrations mechanically attached to a noncode-stamped item on the containment liner are as follows:

NA-1280 - Installation

NA-3300 - Responsibilities of an N Certificate Holder

NA-3400 - Responsibilities of an NPT Certificate Holder
NA-3500 - Responsibilities of an NA Certificate Holder

NA-4000 - Quality Assurance

NA-5000 - Inspection

NA-8000 - Certificates of Authorization Nameplates, Stamping, and Reports

NE-8000 - Nameplates, Stamping, and Reports

The electrical penetrations have been designed, manufactured, and shop-tested in accordance with the applicable portions of the ASME Boiler and Pressure Vessel Code Section III, Division 1, 1974 Edition through Winter 1976 Addenda.

The electrical penetrations are installed, inspected, and field-tested in accordance with SWEC Quality Standards (QA and EA procedures relating to Category I installation of safety-related equipment) which conform to 10CFR50 Appendix B.

Acceptability of the installation of the electrical penetrations will be based on a field-performed leak rate test as described in Section 8.3 of IEEE-317-1976.

3.8.1.7 Testing and In-service Inspection

3.8.1.7.1 Concrete Containment

Primary Containment Test

The primary containment is subjected to a structural acceptance test in which it is pressurized internally to 1.15 times the 45 psi design pressure. It is pressurized in increments of 10 psi from atmospheric pressure to its peak value of 51.75 psi and then depressurized in a similar manner to atmospheric pressure. At the end of each 10 psi step change, the pressure is held for at least 1 hr. Strain and deflection measurements are recorded and compared to predicted values.

The structural acceptance test for primary containment equals or exceeds the requirements of RG 1.18 for nonprototype containments. The strain measurements are made by using strain gauges that have been installed at two azimuths on the inside and outside meridional and hoop rebars at various elevations. The radial displacements of the containment are measured using direct current differential transducers (DCDTs) at several points along six meridians spaced around the containment. In addition to this, additional strain gauges and DCDTs are mounted around the personnel hatch to record the strains and displacements, respectively. These measurements are compared to the predicted values which are shown in Table 3.8-15. The predicted response of the structure is based on the analysis procedures described in

Section 3.8.1.4. The measured maximum deflections at points of maximum predicted deflection are acceptable if they do not exceed the predicted values by more than 30 percent. This requirement may be waived if the deflection recovery within 24 hr after complete depressurization is greater than 80 percent. These tolerances are based on the tolerances allowed in ASME III, Division 2. Crack patterns exceeding 0.01 in in width before, during, or after the test are mapped.

<u>Drywell Floor Test</u>

The drywell floor is subjected to the design differential pressure of 25 psi. The structural adequacy of the floor will be determined by a visual inspection for signs of permanent damage to the concrete after return to the atmospheric pressure. The response of the containment is measured during the test using procedures similar to the ones used for the primary containment test.

3.8.1.7.2 Steel Liner and Penetrations

<u>Steel Liner</u>

The primary containment liner is required to withstand the pressure of 115 percent of the design pressure by the acceptance test described in Section 3.8.1.7.1. The primary containment leakage rate test (Section 6.2.6) is conducted after the structural acceptance test in compliance with the requirements of Appendix J of 10CFR50. Test channels are provided for all seams covered by concrete or otherwise made inaccessible after completion of construction and all basic liner seam welds in the suppression chamber. This will enable leak-tightness testing of these local areas to be performed at any time during the life of the plant should the need arise.

<u>Access Hatches and Drywell Head</u>

The personnel airlocks and the escape airlocks were subject to a shop acceptance test in accordance with Article NE-6000 of ASME Section III, Division 1. They were also subject to a shop pressure decay test. The seals for the airlocks were subject to a halogen leak test, and the seals for the drywell head were subject to a pressure decay test. The seals for the drywell hatches were subject to a halogen leak test, and the stainless steel suppression pool hatch seals were pressure decay tested. Test channels are provided for the convenience of leak testing during construction for all containment hatch-to-liner reinforcement plate welds. This also will enable leak-tightness testing of these weld joints to be performed at any time during the life of the plant should the need arise.

<u>Penetrations</u>

All penetrations were subject to a shop hydrostatic acceptance test in accordance with NB-6000 or NC-6000, as applicable, of ASME III, Division 1, 1971 Edition through Summer 1973 Addenda.

Additionally, penetrations Z-32 (neutron monitoring - GSN), Z-46A and Z-47 (closed loop cooling - CCP), Z-46C (fire protection - FPW), Z-99A through D and Z100A through D (reactor core cooling hydraulic system - RCS) were hydrostatically tested after installation in accordance with Paragraph NC-6129 of ASME III, 1977 Edition, Winter 1978 Addenda.

Test channels are provided only for all containment penetration insert-to-liner reinforcement plate welds which are welded from one side with a backing strip to enable leak-tightness testing of these weld joints to be performed at any time during the life of the plant should the need arise. The integral flued head forging design of the thermally hot piping penetration permits ISI of the process pipe to transition region. The ends of the piping forging are dimensioned to facilitate ISI of the process pipe welds.

3.8.2 Not Applicable

3.8.3 Concrete and Steel Internal Structures of the Primary Containment

3.8.3.1 Description of Internal Structures

The internal structures support the RPV, provide shielding, and form the pressure suppression system. The internal structures include the following:

1. Drywell floor.
2. Reactor vessel pedestal.
3. Biological shield wall.
4. Star truss.
5. Floors.

Steel linear supports for RCS systems are described in Section 3.9A.3.4.

The containment internal structures are Category I and are shown on Figure 3.8-11. The BSW is a composite structure of pressure vessel quality steel and heavy-density fill material. The drywell floor and pedestal are heavily reinforced concrete structures. The concrete structures are designed for DBA conditions and provide radiation shielding. The DBA radiation source terms have been evaluated and do not adversely affect these structures (Section 12.3). The star truss is constructed

and designed to withstand DBA conditions, including temperature effects, along with seismic and other applicable loads.

3.8.3.1.1 Drywell Floor

The drywell floor (Figures 3.8-12 and 3.8-13) is a reinforced concrete annular slab, 4 ft thick, having an inner diameter of 30 ft supported by the pedestal, and an outer diameter of 91 ft supported by the primary containment wall.

The drywell floor serves both as a pressure barrier between the drywell and suppression chamber and as the lateral support structure for the reactor pedestal and anchor support for the downcomers.

The drywell floor is rigidly connected to the containment wall thereby preventing relative motion between the two structures. This alternative approach is used as a substitute for the requirement of a drywell floor seal. A full moment and shear connection is provided by cadwelding the reinforcing bars to the reinforced liner plate (Figure 3.8-13). Thermal expansion is considered in the containment design, and the resulting forces and moments on the floor are accommodated within the allowable stress limits.

The drywell floor at el 240 ft is penetrated by 118 24-in diameter stainless steel downcomer pipes, and four 24-in diameter truncated downcomers for vacuum breakers. A description of the downcomer vents is given in Section 6.2.1. Figure 3.8-12 shows the arrangement of the downcomers. The drywell floor is also penetrated by 18 12-in diameter SRV lines, 18 2 1/2-in diameter SRV vent lines, and 20 4-in diameter floor drains. SRV line penetrations 12 in in diameter are flued head-type penetrations as shown on Figure 3.8-13. SRV vent line penetrations 2 1/2 in in diameter are collar-type penetrations similar to instrument penetrations as shown on Figure 3.8-9.

The seal for the drywell slab is provided by a 3/16-in thick liner that is placed on top of the floor slab (Figure 3.8-13). A minimum of 6-in thick reinforced concrete insulation slab, sloped for drainage, is placed on the liner.

The cavity floor slab, located at el 232 ft, is 3 ft thick and is within the pedestal shell walls. This slab is an extension of the drywell floor but is 8 ft lower than the drywell floor to avoid interference with the CRD assemblies. The slab is penetrated by eight downcomer pipes and one floor drain. A full moment and shear connection is provided between the slab and pedestal. Similar to the drywell floor, this slab provides a pressure barrier between the drywell and suppression chamber.

3.8.3.1.2 Reactor Vessel Pedestal

The reactor vessel pedestal is a 91-ft 6 1/2-in high, heavily reinforced concrete right vertical cylindrical shell with a 20-ft 3-in inside diameter and a thickness that varies from 4 ft at the base to 5 ft 1 1/2 in above the drywell floor. The wall thickness increases to 7 ft 9 1/2 in at the top portion which supports the RPV and the BSW. At lower elevations the pedestal also supports the drywell floor, the cavity floor slab, various pipe rupture restraints, radial beams, and pipe supports. The pedestal is located concentric with the RPV centerline and is supported on the reactor building foundation mat (Figure 3.8-14). A full moment and shear connection at the junction of the pedestal and mat is provided by reinforcement extending into the mat.

The top of the pedestal contains embedded anchor bolts for anchorage of the RPV and BSW. The reactor vessel is anchored by two 3-in diameter bolts at every 6 deg of pedestal circumference; the BSW is anchored by two 2 1/4-in diameter bolts at approximately every 5 deg of pedestal circumference. All pedestal concrete surfaces in the suppression chamber are lined with a 1/4-in stainless steel liner plate. The lined surface includes the bottom of the cavity floor on the inside of the pedestal and extends to the drywell floor on the outside of the pedestal. The pedestal liner plate is anchored to the concrete by headed concrete anchor studs.

3.8.3.1.3 Biological Shield Wall

The BSW consists of two concentric steel cylinders connected by internal horizontal and vertical stiffeners (Figure 3.8-15). The shield wall is 48 ft 4 in high and has an inner radius of 14 ft 3/4 in and an outer radius of 15 ft 9 1/4 in. The shield wall is supported by the reactor pedestal and is attached to the pedestal by embedded anchor bolts (Figure 3.8-15).

The inner and outer walls and the stiffeners are 1 1/2 in thick, ASTM A537 Class 1 steel plates connected by full penetration welds. The space between the walls is filled with nonstructural heavy-density fill material (a combination of cement and iron ore) for radiation shielding purposes. The shield wall is penetrated by air duct openings, inspection and access openings, instrumentation lines, and piping penetrations for various systems. Attached to the BSW are pipe restraints, star truss and stabilizer supports, insulation support brackets, and miscellaneous supports for structural steel floor beams. The BSW protects the RPV from pipe whip, jet impingement, and missile loads, and protects drywell structures from the effects of recirculation and feedwater line breaks in the annular region.

3.8.3.1.4 Star Truss

The star truss (Figure 3.8-16) is a tubular structure spanning between the BSW and the primary containment wall (Figure 3.8-16), whose function is to provide lateral support to the BSW, which in

turn provides lateral support to the RPV. It is welded to the ring girder extension on top of the BSW at six equally spaced, circumferential locations. This welded connection provides a fixed support for the star truss extending to the primary containment wall. The star truss connection at the primary containment wall allows unrestrained radial and vertical translation but prevents tangential translation.

3.8.3.1.5 Floors

Floors are located within the containment to provide support for and access to equipment. The floors generally are constructed of steel framing with steel grating or checkered plate decks. The steel grating or checkered plate flooring is supported by steel beams which, in turn, are supported by radial or nonradial steel girders spanning from the reactor pedestal or the BSW to the primary containment wall. This floor framing system is braced against lateral movement. The steel girders are simply supported at the primary containment wall by beam seat arrangements and are pin connected to the embedment plates at the reactor pedestal end.

3.8.3.2 Applicable Codes, Standards, and Specifications

The design codes, standards, specifications, and regulations used in the design, procurement, fabrication, and construction of the steel and concrete containment internal structures, meet or exceed the requirements outlined in Section 3.8.4.2 for the design of Category I structures.

The procurement specification requirements for concrete, reinforcing steel, and other materials used in the drywell floor and the RPV pedestal are described in Section 3.8.4.2. The summary of the procurement and erection specifications for the BSW and the star truss are given in Sections 3.8.3.2.1 and 3.8.3.2.2, respectively.

3.8.3.2.1 Biological Shield Wall

Design and construction criteria for the BSW are:

1. <u>Design Stresses</u> AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, 1969.

2. <u>Quality Control</u> Testing, inspection, and documentation, ANSI N45.2.5 and 10CFR50 Appendix B.

3. <u>Welder Qualification</u> ASME Section IX or AWS D1.1.

4. <u>Welding</u> AWS D1.1 with the exception to Section 3.3 that an offset of 1/2 in maximum is permitted.

5. NDE Examination for Welding Ultrasonic inspection or progressive magnetic particle method of inspection.

6. Packaging, Shipping, Storage, and Handling of Materials ANSI N45.2.2 and RG 1.38.

7. Traceability Full traceability for inner and outer wall plates, stiffeners, base plate, sole plate, doors, and bolts over 1 in diameter.

8. Painting RG 1.54.

9. Erection AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, 1969, and AWS D1.1.

3.8.3.2.2 Star Truss

Design and construction criteria for the star truss are:

1. Design Stresses ASME Section III, Subsection NF, 1977, unstamped.

2. Quality Control Testing, inspection, and documentation - 10CFR50 Appendix B, ASME Section III, Subsection NF.

3. Fabrication and Welding ASME Section III, Subsection NF, unstamped, AWS D1.1.

4. NDE Examination for Welding ASME Section III, Subsection NF, ASME Section V, progressive magnetic particle inspection.

5. Packaging, Shipping, Storage, and Handling of Materials ANSI N45.2.2 and RG 1.38.

6. Painting RG 1.54.

7. Erection AWS D1.1 and ASME Section III, Subsection NF, unstamped.

3.8.3.3 Loads and Loading Combinations

3.8.3.3.1 Drywell Floor and Reactor Vessel Pedestal

The loads imposed on these concrete structures, the notations used, and the load combinations are the same as those listed in Section 3.8.1.3 for concrete structures.

3.8.3.3.2 Biological Shield Wall

For normal operating loading combinations, the design stresses for the BSW are in accordance with the AISC Specification for the

Design, Fabrication, and Erection of Structural Steel for Buildings; however, for abnormal and/or extreme environmental design load combinations, the allowable elastic stresses are increased in accordance with factored (e.g., 1.6, 1.8, 2.0) sectional strengths as shown in load combination equations 3 through 6 of Table 3.8-9.

The BSW is analyzed and designed for a variety of individual pipe rupture forces and associated pressures. Each section of the BSW is designed to satisfy the maximum pipe loading condition for that area.

The load combination equations used for the design of the shield wall are presented in Table 3.8-9. The BSW is also designed for hydrodynamic load combinations listed in Table 3.8-10, Part II.

3.8.3.3.3 Star Truss

The star truss loads, notations, and load combinations are the same as those listed in Section 3.8.4.3 for steel structures. Additionally, the star truss is checked for the effect of SRV loading resulting from suppression pool hydrodynamics, in addition to other concurrent loading as listed in Table 3.8-10, Part II.

3.8.3.4 Design and Analysis Procedures

3.8.3.4.1 Drywell Floor

The drywell floor is analyzed and designed for the load combinations outlined in Section 3.8.4.3. The analysis procedure used is outlined in Section 3.8.3.4.2. The following effects of a potential LOCA occurring within the drywell are considered in the design:

1. Jet impingement forces on the drywell floor.

2. Impact loads transmitted to the drywell floor by any attached pipe whip restraints.

3. Differential pressure across the drywell floor of:

    a. 25 psi (drywell pressure greater than suppression chamber).

    b. 10 psi (suppression chamber pressure greater than drywell).

Additionally, the drywell floor is designed for dead load, seismic, thermal, and hydrodynamic loading (Appendix 6A).

All loads appropriately combined will not compromise the intended function of the drywell floor to provide a barrier between the drywell and suppression chamber. Stress resultants are

calculated using elastic methods in accordance with ACI-318 and the AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, as applicable. Reinforcement arrangements in the drywell floor are shown on Figure 3.8-13.

3.8.3.4.2 Reactor Pedestal

The pedestal is analyzed and designed for the load combinations outlined in Section 3.8.4.3. Inertia loadings from earthquakes are obtained from the dynamic analyses of the reactor building, as outlined in Section 3.7A.2.

The reactor pedestal and drywell floor are analyzed using SHELL 1, a finite element computer program (Appendix 3A). The structures are modeled using axisymmetric shell elements. The asymmetric loadings (e.g., seismic loads) are represented by a series of Fourier coefficients.

The boundary conditions for the pedestal are considered to be fixed at the junction with the foundation mat. For the boundary conditions at the top of the pedestal, forces at the base of the BSW and RPV resulting from the loads defined in Sections 3.8.3.3.2 and 3.8.4.3, respectively, were applied to the SHELL 1 model. The boundary conditions for the drywell floor at the junction with the primary containment reflect the stiffness of the primary containment at the junction.

Design of the pedestal conforms to the requirements of ACI-318-71. Arrangements of reinforcement at the mat-pedestal and pedestal-BSW interface and in the pedestal wall are shown on Figure 3.8-14.

The top portion of the pedestal is designed to resist all seismic and pipe rupture forces transmitted from the reactor vessel skirt and the base of the BSW. Pipe rupture forces and discontinuity forces at the base of the BSW resulting from pressurization of the annulus between the reactor vessel and BSW during a recirculation or feedwater line break are used to analyze and design the pedestal in combination with the seismic forces determined from the dynamic analysis of the reactor building. In addition, the pedestal is analyzed and designed for jet impingement and pipe rupture loads.

To ensure that the pedestal cracking problem that occurred in the James A. FitzPatrick Nuclear Power Plant does not occur in the Unit 2 pedestal, the construction technique and the anchorage detail for the BSW are modified. Because of the BSW construction technique described in Section 3.8.3.6.2, the weld shrinkage and distortion usually encountered during fabrication and assembly have stabilized prior to the BSW's attachment to the reactor pedestal. In addition, the BSW is designed to allow radial movement due to thermal effects with limited effect on the reactor pedestal. Therefore, the reactor pedestal is adequately

designed and will maintain its structural integrity for the effects of BSW loading.

3.8.3.4.3 Biological Shield Wall

The BSW is analyzed and designed for the load combinations described in Section 3.8.3.3.2. Analysis of the wall is performed using the two-dimensional finite element capability of the STRUDL computer code. Both plane stress and plate bending elements were used. As the structure is sufficiently symmetrical, only one-half of the structure (i.e., 180 deg) need be modeled. The boundary condition at the top of the structure approximates the effect of the star truss structure. The boundary condition at the bottom of the structure is considered an equivalent of springs connected to the pedestal in the circumferential and vertical directions and unrestrained in radial direction. The use of a 180-deg model allows for the analysis of asymmetric loadings by applying half of the load to the model with symmetric boundary conditions and half with asymmetric boundary conditions. The results of the two analyses are then superimposed for the net results. In addition, classical beam theory and plate and shell theory are used for the analysis and design of local areas. The following loads are considered in the analysis and design for LOCA effects:

1. Jet impingement forces on the BSW.

2. Impact loads transmitted to the BSW by any attached pipe rupture restraints.

3. Pressurization of the annulus between the BSW and reactor vessel.

4. Thermal effects.

These loads are combined in accordance with Section 3.8.3.3.2, taking account of the postulated failure locations and types. It has been concluded, because of the dynamic characteristics of the BSW, that the peak restraint impact loads are local impulsive loads on the BSW. These impact loads occur in the first milliseconds after rupture. The shield wall is allowed to yield locally at regions of impact loads provided that:

1. Overall capability of the shield wall to resist elastically the other forces listed is not affected.

2. Local yielding does not produce effects that jeopardize the safety of other components.

3.8.3.4.4 Star Truss

The star truss is a cantilevered plane truss that transmits horizontal force between the top of the BSW and the primary containment wall (Section 3.8.3.1.4). The truss joints at the

BSW end are considered fixed. The support at the primary containment wall end is constrained in the circumferential direction so that only tangential horizontal force can be transmitted to the primary containment wall.

The star truss is analyzed using the STARDYNE computer program (Appendix 3A) and is designed for the load combinations listed in Section 3.8.3.3.

The complete structure is modeled using a combination of beam and plate elements for the finite element model. The beam part of the model represents the star truss part of the structure, while the plate elements represent the remaining parts of the structure.

The structure is designed using the STARDYNE computer program for each individual loading condition, and the stresses are calculated by superimposing the applicable loads with appropriate load factors for each load combination described in Section 3.8.3.3.

3.8.3.4.5 Floors

The structural steel framing system for floors within the primary containment are analyzed using the STRUDL computer program, as necessary, for the loads and load combinations outlined in Section 3.8.4.3. The floor framings are supported vertically at the containment end as well as at the reactor pedestal end. The connection at the pedestal end is treated as a pinned connection whereas the connection at the containment end provides sliding support to the framing members. The STRUDL computer program (Appendix 3A) is used to analyze the floors and platforms. The effects of dynamic loading, such as SRV, seismic, and LOCA, are considered while analyzing and designing these members. The design parameters, allowable stresses, and material properties, as described in Section 3.8.4, are selected in accordance with the requirements of the AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings.

3.8.3.5 Structural Acceptance Criteria

For concrete structures, the allowable stresses, load factors, and capacity reduction factors are in accordance with strength design methods of ACI-318-77, with the following exception: design for horizontal shear forces is in accordance with the requirements of ACI-318-71 which incorporates the combined effects of shear and tensile stresses into the nominal allowable shear stress, v_c, to be carried by the concrete.

Tangential shear stress (v_u) resulting from earthquake loading will be resisted by the concrete or by the concrete and steel reinforcing bars. The tangential shear stress (v_c) carried by the concrete is taken equal to one of the following values:

v_c = 12,000 p for p <0.01, or

v_c = 93 + 2,700 p for 0.01 ≤p ≤0.025

Where:

p = Lesser of the steel-to-concrete ratios in the vertical or circumferential direction

v_c = Tangential shear stress, psi

For steel structures, the allowable stresses and safety factors are in accordance with the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, with the following exceptions:

1. No credit is taken for the 33 1/3-percent increase in allowable stresses, S, permitted in the AISC Code when earthquake and wind loads are present in the load combinations.

2. The structural members of the star truss are designed (unstamped) in accordance with ASME Boiler and Pressure Vessel Code, Section III, Division 1, Subsection NF, 1977 Edition including Summer 1978 addenda. The allowable stress limits set forth in Appendix XVII of this code are used in designing the structures.

3.8.3.6 Materials, Quality Control, and Special Construction Techniques

The construction materials and the quality control requirements used for the containment internal structures are the same as those used for other Category I structures, as described in Section 3.8.4.6. When necessary, the special materials and the special construction techniques are used as described in Sections 3.8.3.6.1 through 3.8.3.6.3.

3.8.3.6.1 Reactor Vessel Pedestal and Drywell Floor

Precast concrete beams and stainless steel beams are used as stay-in-place forms to support the drywell floor during construction. The support system for the drywell floor between the reactor vessel pedestal and primary containment at el 240 ft consists of 20 precast beams spanning radially. Stainless steel beams span between the precast beams. Inside the reactor vessel pedestal, the support system for the slab at el 232 ft consists of two precast concrete beams supporting stainless steel beams.

The liner on both sides of the reactor vessel pedestal in the suppression chamber serves as the form for pouring concrete. A regular form is used for the upper portion of the pedestal.

3.8.3.6.2 Biological Shield Wall

The BSW inner and outer walls, stiffeners, and base plate are manufactured in accordance with the ASTM A537 Class 1 specification. In addition, the stiffeners and base plate contain a maximum 0.01 percent sulfur to increase the through-thickness direction properties. The ASTM A537 Class 1 steel plates are 100-percent ultrasonically tested to the ASTM A578 Level I specification.

The BSW anchor bolts are manufactured to ASME SA-193 Grade B7 specification and are nondestructively examined by either magnetic particle or liquid penetrant methods in accordance with ASME Section III, Subparagraph NF-2581.1.

The BSW is filled with a heavy-density fill material that is installed using a pressure grouting technique. The technique consists of pumping heavy-density fill material at the base of the shield wall and allowing it to flow upward through the compartments to a predetermined lift height forcing any trapped air ahead of the fill material. This process continues circumferentially and vertically around the BSW until the entire structure is filled.

The BSW is shop fabricated in three rings, each approximately 16 ft high. Each ring is fabricated in three 120-deg sections. The sections are assembled at the site to form three 360-deg rings which are then stacked and welded together to form the BSW.

3.8.3.6.3 Star Truss

The material used in the star truss assembly is SA-537, Class 1, Lukens Lectrefine, except that the diagonal members of the truss system are tubular in cross section, conforming to SA-333 Grade 6. The ring girder at the top of the BSW is made of ASTM A537, Class 1, Lukens Lectrefine material. The truss connections are welded connections with full penetration welds.

3.8.3.7 Testing and In-service Surveillance Requirements

Drywell pressurization tests are performed in accordance with the requirements of Technical Specifications and Appendix J to 10CFR50. Testing of the construction materials referenced in this section is described in Section 3.8.4.6. A drywell floor structural test is performed and is described in Section 3.8.1.7.1.

3.8.4 Other Seismic Category I Structures

3.8.4.1 Description of the Structures

As shown in Table 3.2-1, other seismic Category I structures (e.g., diesel generator building, control building) that contain or support safety-related systems and/or equipment are designed

to withstand the SSE. These structures, except for the main stack, are also designed to withstand tornado loads including tornado-generated missiles. Seismic loads are not considered to act simultaneously with tornado loads. Table 3.2-1 identifies seismic Category I equipment and structures that are tornado protected.

In general, seismic Category I structures, herein called Category I structures, are completely independent of adjacent structures. Adequate space is provided between structures to retain their independent functional characteristics, and to allow for rotation, translation, and deformation under seismic loading. The spaces have flexible seals (Figure 3.8-17). Structures having both Category I and nonseismic category elements are designed using the Category I criteria for the Category I portions of the structures. The Category I structures or portions of the structures are also investigated to determine the effect of failure of nonseismic category structures or portions of the structures. If it is determined that their failure (under Category I loading conditions) could endanger the integrity of the Category I portions of the structures, then the nonseismic structures or portions are also designed to Category I criteria. In these instances, where Category I structures are integrally connected to the other structures, the Category I structures are analyzed and designed considering the effect of the interconnection and modeled with the connecting structure(s) as a unit. Those portions of the Category I structures located below the DBFL are provided with waterstops (Figure 3.8-17). Vermiculite, vermiculite concrete, or compressible materials are used between the exterior face of substructure concrete walls and the excavated face of rock surfaces, as described in Section 2.5.4.10. No unique materials or features are used in the design or construction of the structures described in this section.

The relative locations of the Category I structures are shown on Figures 1.2-1 and 1.2-2. The general arrangement of operating personnel access between the buildings is shown on Figures 1.2-3 through 1.2-5. The general arrangements of the Category I buildings are illustrated in the following figures:

Reactor building including auxiliary bays, Figures 1.2-6 through 1.2-12

Auxiliary service building, Figures 1.2-7 and 1.2-8

Radwaste building, Figures 1.2-13 and 1.2-14

Standby gas treatment building, Figures 1.2-35 and 1.2-36

Control room building, Figures 1.2-15 and 1.2-16

Screenwell building including service water pump room, Figures 1.2-26 through 1.2-28

Intake tunnels, Figures 1.2-29 and 1.2-30

Main stack, Figure 1.2-31

Diesel generator building, Figures 1.2-17 and 1.2-18

Offgas rooms, Figures 1.2-19 through 1.2-25

Intake structure, Figure 1.2-29 and 1.2-30

The general arrangements of the major buildings that are not Category I are illustrated in the following figures:

Turbine building, Figures 1.2-19 through 1.2-25

Natural-draft cooling tower, Figures 1.2-38 and 1.2-39

Service building, Figure 1.2-19 and 1.2-22

Regeneration and condensate demineralizer regenerative rooms, Figures 1.2-19 through 1.2-25

Auxiliary boiler building, Figure 1.2-34

Normal switchgear building, Figures 1.2-32 and 1.2-33

Hydrogen storage area, Figure 1.2-40

CST building, Figure 1.2-37

The Category I structures other than the primary containment and its internal structural components are described in the following sections. For the primary containment and its internal structural components, refer to Sections 3.8.1 and 3.8.3, respectively.

3.8.4.1.1 Reactor Building

The reactor building completely encloses the reactor and the primary containment. The reactor building houses the refueling and reactor servicing equipment, new and spent fuel storage facilities, and other reactor auxiliary or service equipment, including the RCIC system, reactor water cleanup (RWCU) system, SLC system, CRD system equipment, core standby cooling systems, RHR systems, and electrical equipment components. The primary purposes for the secondary containment are to minimize ground level release of airborne radioactive materials and to provide means for a controlled elevated release of the building atmosphere if an accident should occur.

The reactor building wall is a 166-ft ID reinforced concrete cylinder with varying wall thickness, extending from the top of the mat at el 175 ft to the polar crane level at el 386 ft 10 in. The wall from the crane rail elevation to the roof at el 430 ft

1 in (approximately 40 ft) is steel framing with insulated metal siding. The metal siding panels have sealed joints to minimize air leakage.

Roof construction consists of steel trusses, which support metal decking, covered with insulation, and asphalt and gravel built-up roofing. The structural steel frame of the reactor building upper superstructure is designed to withstand the tornado wind with the metal siding remaining in place during a tornadic event. The reactor building roof decking is designed for normal wind loading. When the design velocity is appreciably exceeded, the decking may blow off. The reactor building metal siding is designed to withstand wind loads generated during a tornadic event.

Various floor levels form diaphragms within the secondary containment to provide support for and access to equipment. The floor and wall thicknesses meet both shielding and structural requirements. The reactor building superstructure floors and walls are entirely separated from the primary containment structure by a space that prevents the reactor building from restraining the primary containment under any condition.

The refueling floor level contains the fuel pool and is supported by the reactor building wall and the fuel pool girders.

The arrangement details of the reactor building wall and reinforced concrete floor levels are shown on Figures 1.2-6 through 1.2-12.

A portion of the main steam tunnel is an integral part of the reactor building between the primary containment and the reactor building wall.

The reactor building, including the auxiliary bays, is founded on a rock-bearing, reinforced concrete mat and is designed for the load combinations in Section 3.8.4.3. The mat acts to support the reactor building, auxiliary bays, and the primary containment. The auxiliary bays are rigidly attached to the reactor building and considered part of the secondary containment structure.

The drainage system around the reactor building is described in Section 2.5.4.6.

3.8.4.1.2 Control Room Building

The control room building is a Category I structure. It is a five-story concrete and steel structure 96 by 134 ft in plan and is designed for earthquake and tornado loads. The exterior walls and roof are constructed of a minimum of 2-ft thick reinforced concrete and are designed to provide tornado missile protection. The interior floors are concrete decking supported by steel

framing. The building is founded on bedrock and is supported by a reinforced concrete mat at el 209 ft 6 in.

The control room building houses the control room, safety-related switchgear, batteries, and associated equipment. The upper four floors are reinforced concrete slabs on steel deck supported by structural steel. Two steel-framed grating stairways are provided for access to the upper floors. In addition, access to the turbine building is provided at each floor.

Underground concrete tunnels connect the control room building to three points of entry into the reactor building. These tunnels are designed to resist both tornado and earthquake loads. The diesel generator building is located south of and adjacent to the control room building.

3.8.4.1.3 Diesel Generator Building

The diesel generator building is a single-story, Category I structure 87 by 98 ft in plan, enclosing the three diesel generators and their associated equipment with a ground floor slab at el 261 ft. The diesel generators are supported on reinforced concrete pedestals. The building is divided into three rooms separated by fire walls, each housing one diesel generator. Three fuel oil storage tanks are located below the building, with their fuel oil pumps housed in the individual diesel generator rooms.

The diesel generator building is a reinforced concrete structure founded on bedrock and supported by wall footings. The exterior walls and roof are a minimum of 2 ft thick and are designed to provide tornado missile protection. All ventilation intakes are arranged to preclude penetration from tornado-generated missiles. The diesel generator exhaust silencer is not protected by a missile hood; however, an exhaust relief valve is provided to maintain the diesel generator function, and this exhaust relief valve is protected against missiles (see Sections 9.5.8.1 and 9.5.8.2). The building is located south of the control room building. Personnel access to the building is provided from the control room building.

3.8.4.1.4 Screenwell Building

The screenwell building consists of a concrete substructure and a steel frame superstructure. The substructure, including the service water pump room, is designated Category I, whereas the steel frame superstructure including the circulating water pump and water treatment area is designed as a non-Category I area. The screenwell building includes the service water pump rooms, the diesel and electric fire pump rooms, the water treatment area, the circulating water pumps, and other associated equipment.

Stop logs, traveling screens, trash rakes, etc., are set in the concrete walls, as required to divert the flow of water, and for maintenance purposes, respectively. These components are built-up structures of steel and concrete guided and supported by the reinforced concrete walls and floors.

The screenwell building concrete substructure consists of walls and slabs arranged to direct the flow of lake water and to support equipment including circulation water pumps, trash racks, and screens. The screenwell building superstructure is a steel building 140 by 220 ft with insulated metal wall panels. The steel roof deck is covered with insulation and four-ply, built-up roofing.

The safety-related service water pumps are enclosed in a tornado-resistant, concrete structure within the screenwell building. The diesel and electric fire pumps are enclosed in separate concrete rooms in the screenwell building.

North of the screenwell building, there are concrete chambers that house the reverse flow gates. These gates are located adjacent to the intake and discharge shafts. The shafts extend vertically down into bedrock to el 120 ft and terminate at the intake and discharge tunnels.

3.8.4.1.5 Intake Structures

The two Category I intake structures are hexagonal-shaped reinforced concrete structures connected to the intake and discharge tunnels. The structures rest on a tremie slab founded on bedrock at the lake bottom and are anchored to the concrete-encased steel tiedowns embedded into the rock. Each hexagonal-shaped intake structure has a face-to-face dimension of approximately 29 ft and a height of 10 ft 6 in. The continuity of waterflow into the intake tunnel is assured by means of electrically heated bar racks, one at each face of the hexagon.

3.8.4.1.6 Intake (and Discharge) Tunnels

The general arrangement and details of intake (discharge) tunnels are shown on Figure 1.2-29. As noted on Sheet 2 of Figure 1.2-29, the tunnels are lined with 3 1/2 in shotcrete.

The two tunnels (13 ft 5 in x 13 ft 5 in) extend from the screenwell shaft about 1,400 and 1,300 ft, respectively, eastward and northward under Lake Ontario to the intake structures. The nonseismic category discharge tunnel extends an additional 500 ft beyond the intake structure to the discharge diffusers. The tunnels are lined with 3 1/2 in shotcrete.

Within Tunnel No. 1, the intake water flows through a 4 ft 6 in ID formed opening in an 8 ft 4 in wide by 6 ft 9 in high Category I concrete encasement. The discharge water flows around the concrete encasement within the tunnel and is eventually

discharged into the lake via a discharge diffuser. The tunnels are normally flooded.

3.8.4.1.7 Electrical Tunnels and Piping Tunnels

Category I electrical tunnels and piping tunnels (Figure 1.2-2) contain Category I systems and are constructed of reinforced concrete. The tunnel walls and roof are of sufficient thickness to resist penetration by tornadic missiles, or the tunnels are buried underground as required for missile protection.

Tunnels are isolated from adjoining structures by a space except that they are integrally connected to the adjacent structures when required to prevent sliding overturning and/or flotation.

Category I electrical and piping tunnels are protected from external flooding by:

1. Sealing the space between the tunnels and the adjoining structures using waterstops and flexible seals (Figure 3.8-17).

2. Providing all penetrations below grade with air and water seals, as applicable.

3.8.4.1.8 Main Stack

The Category I unlined concrete main stack located on the northeast side of the power station is designed and constructed to provide elevated release of offgas, standby gas treatment, turbine building ventilation, and other systems.

The operating conditions for the main stack are described in Sections 6.5.1, 9.4.4, and 11.3 as part of the system description. The base of the mat is at el 242 ft. The ground-level slab is at el 261 ft and the top slab is at el 276 ft. The inside base diameter is approximately 26 ft and the inside top diameter is approximately 6 ft. The height of the stack is approximately 430 ft.

3.8.4.1.9 Standby Gas Treatment Building and Railroad Access Lock Area

The standby gas treatment (SGT) building and railroad access lock area are classified Category I structures up to el 286 ft. The portion of the building above el 286 ft is classified as nonseismic.

The SGT building is a two-story, reinforced concrete and steel-framed structure approximately 40 x 90 ft. The structure shares a common wall with the railroad access lock adjacent to the reactor building. The reinforced concrete floor slab is provided at the grade level of el 261 ft. The roof is steel deck with insulation and four-ply, built-up roofing.

A railroad access lock approximately 25 x 90 ft is provided adjacent to the reactor building. This building is a reinforced concrete and steel-framed structure and shares a common wall with the SGT building. The reinforced concrete floor slab is provided at the grade level of el 261 ft. The roof is steel deck with insulation and four-ply, built-up roofing. Interlocking swinging doors are provided at each end of the structure. The exterior door is designed to withstand tornado-generated missiles. Doors are gasketed to minimize air leakage. The interlocks between the sets of doors safely prevent the doors at one end from being opened until the doors at the other end have been closed and sealed. This building is designed to accommodate a 66-ft long railroad car, plus 8 ft for a track-mobile engine.

3.8.4.1.10 Auxiliary Service Building

The auxiliary service building is a two-story, reinforced concrete and steel-framed structure approximately 55 x 76 ft in plan. The auxiliary service building below el 261 ft is classified as a Category I structure. The building is surrounded by the reactor building, turbine building, and control building. The basement floor is a concrete slab poured over electrical tunnels. The floor at el 261 ft is a concrete slab on steel deck supported by structural steel. The roof is steel deck with insulation and four-ply, built-up roofing.

3.8.4.1.11 Radwaste Building

The radwaste building houses the radioactive waste system and is analyzed to seismic conditions. It is a five-story, concrete and steel building, approximately 110 x 150 ft. The exterior walls are reinforced concrete. A rolling steel door is provided in the north wall for truck access into the building. The radwaste building is classified as shown in Table 3.2-1.

Where required for shielding purposes, interior concrete walls are provided. The basement floor is a concrete mat on rock. The upper four floors are concrete supported by steel deck and beams. The roof consists of steel framing with steel deck, insulation, and four-ply, built-up roofing. Two steel-framed grating stairways are provided for access to the upper floors.

The decontamination area is located south of the radwaste building. It is an extension of the turbine building and the radwaste building. The structure is a four-story building of concrete and steel approximately 110 x 105 ft in plan. The exterior walls are of reinforced concrete. The superstructure is covered with insulated steel roof deck with four-ply, built-up roofing.

3.8.4.1.12 Turbine Building

The turbine building complex includes the turbine building, heater bays, main steam tunnel, and condensate demineralizer regenerative and offgas area. The complex houses the turbine generator, condenser, moisture separator, etc., in the turbine building areas, heaters and related pumps and accessories in heater bay areas, and offgas system equipment and tanks in offgas areas. A portion of turbine building, main steam tunnel area, and offgas area are analyzed to seismic conditions, whereas the remaining portions are designed as nonseismic.

The turbine building houses the turbine generator and associated auxiliary systems. The heater bay which houses the feedwater heaters is adjacent to and north of the turbine building. East of the turbine building, the main steam tunnel connects the turbine building with the reactor building. North of the turbine building and west of the heater bay, the regeneration and condensate demineralizer area, offgas room, building service equipment area, and turbine building railroad passage are located.

The turbine building is a concrete and steel, three-story building, approximately 130 x 333 ft. A slab is provided at el 250 ft. All four walls are of reinforced concrete up to el 261 ft. Above this elevation, the structure is steel framed. Where exterior concrete walls are required for shielding purposes, the structural steel is encased in the concrete walls. Insulated metal wall panels are used for exterior walls. The steel deck roof is covered with rigid insulation and four-ply, built-up roofing, along with a single ply of tri-polymer alloy membrane. Smoke vents are installed on the roof.

The turbine building operating floor is concrete supported by steel deck and beams. Except where concrete floors are required for shielding, the mezzanine floor is steel-supported galvanized steel grating. Access to the upper two floors of the turbine building is provided by three sets of steel-framed galvanized grating stairs and three electric elevators.

The turbine pedestal support is of reinforced concrete and located near the center of the turbine building. The support is isolated from the turbine building by vibration joints at the mezzanine floor el 277 ft 6 in and at the operating floor el 306 ft.

The three-story heater bay is located north of the turbine building and divided into three areas by concrete walls. The exterior walls are insulated metal siding. Roof deck with steel frame construction, rigid insulation, and four-ply, built-up roofing covers the roof. The ground floor is a slab at el 250 ft, with the mezzanine floor being steel-supported galvanized steel grating. The operating floor is concrete on metal deck and steel framing. A set of steel-framed galvanized grating stairs in each of the three heater areas provides access to the mezzanine and operating floors.

The main steam tunnel, which connects the east end of the turbine building with the reactor building, is a concrete structure. The thickness of the wall, slab, and roof concrete meets both structural and shielding requirements.

West of the heater bay and north of the turbine building is the regeneration and condensate demineralizer area, the offgas room, and building service equipment area. The structure is two stories of concrete and steel construction. The wall, floor, and roof concrete meet shielding and structural requirements. Where shielding is not required for the building service equipment area, insulated metal wall panels and steel roof deck are used. The roof is covered with rigid insulation and four-ply, built-up roofing.

There is a steel-framed railroad passage to the turbine building at el 261 ft under the building service equipment area. The exterior walls consist of insulated metal wall panels. A rolling steel door is provided in the outside wall and also where the railroad enters the turbine building. Personnel access doors are also provided.

3.8.4.2 Applicable Codes, Standards, and Specifications

Codes, specifications, standards, and regulatory guides that are used in establishing design methods, analytical techniques, and material properties for Category I structures are listed herein. The criteria for the structural design of Category I structures are developed using the following regulatory guides and Code of Federal Regulations:

RG 1.10, 1.12, 1.15, 1.19, 1.31, 1.37, 1.50, 1.54, 1.55, 1.60, 1.61, 1.66, 1.69, 1.76, 1.85, 1.94, 1.117.

Appendix A of 10CFR50, Criteria 1, 2, 4, and 5 of General Design Criteria for Nuclear Power Plants

The degree of compliance to these documents is discussed in Sections 1.8 and 3.1, respectively.

The codes and standards used in the structural design of concrete and steel components of the Category I structures are as follows:

ACI-211.1-1974, 1977	American Concrete Institute, Recommended Practice for Selecting Proportions for Concrete
ACI-214-1965, 1977	American Concrete Institute, Recommended Practice for Evaluation of Compression Test Results of Field Concrete
ACI-301-1972, 1975	American Concrete Institute, Specification for Structural

	Concrete for Buildings (Exceptions to this code are listed in Section 3.8.4.6.)
ACI-304-1973	American Concrete Institute, Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete
ACI-305-1972	American Concrete Institute, Recommended Practice for Hot Weather Concreting
ACI-306-1966	American Concrete Institute, Recommended Practice for Cold Weather Concreting
ACI-307-1979	American Concrete Institute, Specification for the Design and Construction of Reinforced Concrete Chimneys
ACI-315-1974	American Concrete Institute, Manual of Standard Practice for Detailing Reinforced Concrete Structures
ACI-318-1971, 1977	American Concrete Institute, Building Code Requirements for Reinforced Concrete (Exceptions to this code are listed in Section 3.8.4.6.)
ACI-347-1968	American Concrete Institute, Recommended Practice for Concrete Formwork
AISC 1969, 1978	American Institute of Steel Construction, Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, including Supplements 1, 2, and 3 (November 1, 1970, December 8, 1971, and June 12, 1974)
AISC 1972, 1976	Code of Standard Practice for Buildings and Bridges, AISC Manual
AISI 1968	American Iron and Steel Institute, Specification for the Design of Cold Formed Steel Structural Members, including 1972 Printing with Addendum No. 1.
ASME III - 1971	American Society of Mechanical

Divisions 1, 5, 9	Engineers, Boiler and Pressure Vessel Code, 1971 Edition through Summer 1973 Addenda, Sections II, III, Divisions 1, 5, and 9, including applicable code cases (Exceptions to this code are discussed in Section 3.8.1.)
ASME III - 1974 Division 1	American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, 1974 Edition through 1976 Addenda (Exceptions to this code are discussed in Section 3.8.1.)
ASME III - 1977 Subsection NF	American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, 1977 Edition, Subsection NF (Exceptions to this code are discussed in Section 3.8.3.)
AWS D1.1-1975 through 1982	American Welding Society, Structural Welding Code (Exceptions to this code are listed in Sections 3.8.4.6 and 3.8.3.2.)
AWS D1.1-83	American Welding Society, Structural Welding Code (This applies to the minimum fillet weld size for SMAW of studs only.)
AWS D12.1-75	American Welding Society, Recommended Practice for Welding Reinforcing Steel Metal Inserts, and Connections in Reinforced Concrete Construction
AWS D1.4-79	American Welding Society, Structural Welding Code - Reinforcing Steel
NCIG-01, Rev. 2, May 7, 1985	Visual Weld Acceptance Criteria (VWAC) for Structural Welding at Nuclear Power Plants, Prepared by Nuclear Construction Issues Group (NCIG)

U.S. Department of Labor, Occupational Safety and Health Administration. Occupational Safety and Health Standards (October 18, 1972).

New York State building codes, as required.

The analysis and design of plant structures follow the current dates of the applicable codes and specifications at the time of design, as listed herein.

3.8.4.3 Loads and Load Combinations

Except when otherwise noted herein and in Sections 3.8.1.3, 3.6.2.3, and 3.8.3.3, the load combinations for Category I reinforced concrete and steel structures are listed in Tables 3.8-11 and 3.8-10, respectively. The load combinations for intake tunnels are listed in Table 3.8-12.

The reinforced concrete structures within the primary containment, the exterior reactor building wall, and the reactor building foundation mat are also designed to withstand SRV and suppression pool hydrodynamic loading, using appropriate SRV load combinations listed in Table 3.8-11, Part II.

Additionally, the steel structures within the primary containment (e.g., BSW, star truss) and steel framing for floor and equipment supports in the reactor building are designed to withstand SRV and suppression pool hydrodynamic loading, using appropriate SRV load combinations listed in Table 3.8-10, Part II.

3.8.4.4 Design and Analysis Procedures

All other Category I structures are analyzed and designed as described herein. The structures are analyzed and designed for the load combinations as outlined in Section 3.8.4.3.

Category I structures are supported on reinforced concrete mat or wall footings. The design of reinforced concrete components of the structures follows ACI-318, whereas the structural steel components of the structures are designed using the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings.

The exterior walls and roof of the structures are of minimum 2-ft thick reinforced concrete and are designed to withstand the most critical loading, as applicable, including the tornado-generated missile impact loads. The exterior walls below grade are designed for earth pressure, hydrostatic pressure, and surcharge loads, as applicable, including the dynamic effect of these loadings during OBE or SSE events.

The roof and floors of the structures are generally supported on steel framing. The floor systems, including the roof, serve as shear diaphragm to transfer lateral loads to the exterior and interior concrete walls acting as shear walls. These walls are designed to withstand gravity loads, in addition to acting as shear walls, and transmit all loads to the foundation. These walls are designed for in-plane shear forces in accordance with the requirements of Section 11.16 of ACI-318.

Masonry wall construction of solid or hollow concrete blocks, bonded together by a layer of mortar, grout, or concrete to form a rigid wall, is not incorporated in Category I applications. However, for equipment access openings, removable, solid concrete blocks contained in position by structural steel supports and adjacent concrete structures are used in Category I areas for equipment replacement and shielding. These removable walls are not used to support Category I systems or components and are not considered to act as shear walls. These walls are designed so as not to damage any safety-related structure, system, or component by virtue of their being completely enclosed by structural steel elements. The structural steel members and adjacent concrete walls providing containment and lateral support for the concrete blocks are designed to withstand the applicable loads and load combinations listed in Section 3.8.4.3.

Category I structures are essentially considered nonvented structures for tornadic loading. Exterior walls, roof, and exterior doors are conservatively designed to withstand a maximum of 3 psi pressure drop and other tornadic loads, in addition to other applicable gravity loads. The exterior doors are either protected from postulated tornado-generated missile impingement by providing protective structures around them, or designed to withstand the tornado-generated missiles.

3.8.4.4.1 Reactor Building

The reactor building wall, floor levels, and superstructure are analyzed and designed for the load combinations outlined in Section 3.8.4.3.

The exterior wall is analyzed using SHELL 1, a finite difference computer program described in Appendix 3A. The building model consists of a concrete cylindrical shell with seven branches representing the concrete floor slabs. Since the roof support system consists of steel trusses supported by structural steel columns, the roof loads are applied as an external loading to the shell.

The reactor building wall is investigated for both seismic shear and lateral earth pressure loads that are generated during an earthquake by the backfill against the exterior wall. The forces, moments, and shears in the exterior wall are determined using the SHELL 1 computer program. The inertia loadings from the dynamic analysis outlined in Section 3.7A.2 are incorporated into the SHELL 1 results to account for the rigid attachment of the auxiliary bays to the cylindrical reactor building wall.

Wind pressure is distributed along the entire height of the reactor building and is analyzed using the SHELL 1 computer program. The wind pressure distribution used in the analysis is outlined in Section 3.3.1. To account for the structural steel roof framing in the SHELL 1 model, the wind pressure exerted on

the roof is applied to the top of the concrete shell as an external load.

The equivalent wind pressure from tornado conditions is presented in Section 3.3.2, and the reactor building is analyzed using the SHELL 1 computer program. The building is also analyzed for a pressure drop of 3 psi (Section 3.3.2) and for the impact of tornado-borne missiles using methods described in Section 3.5.3. The tornado loads are distributed on the structural steel roof framing and siding as outlined in Section 3.3.2.

The reactor building wall is not subjected to the direct pressure or temperature loading resulting from a DBA, but the discontinuity forces, moments, and shears at the base of the wall occur as a secondary effect resulting from the deformations of the mat during the DBA. These discontinuity forces are obtained from the mat analysis (Section 3.8.5) and are included in the wall design.

The design of reinforcing steel for the concrete components conforms to ACI-318. Arrangements of reinforcing steel for typical portions of the reactor exterior building wall are shown on Figure 3.8-18.

The structural steel components of the structure are designed using the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings.

The dead and live loads associated with heavy equipment and cask laydown are provided for in the analysis and design of the reactor building. The dynamic effect of impact caused by dropping a cask is not considered in the analysis and design since the Unit 2 plant makes use of a redundant crane. The consequences of dropping other heavy loads is described in Section 9.1.4.

The spent fuel pool and the reactor internals storage pool are supported by two nonprismatic deep girders that span the reactor building and are in turn supported on four pilasters that transmit vertical loads into the reactor building foundation mat (Figure 3.8-19). The ends of the girders are built integrally with the reactor building wall and thus are partially restrained against rotation at their ends.

The major portion of load on the girders is the contribution from the deadweight of the girders themselves plus various walls and floor slabs that frame into them. Additional sources of load on the girders include:

1. Water, fuel, and fuel racks in the spent fuel pool.

2. Water and live load in the reactor internals pool under refueling conditions.

3. Live loads on floors framing into the girders.

4. Temperature gradients through walls and floors.

5. Earthquake loads.

The sequence of design and analysis procedures followed to ensure the structural adequacy of the girders under the preceding loadings is as follows:

1. Perform a shell analysis of the reactor building using the SHELL 1 computer code to determine the degree of restraint at the ends of the girders. This was accomplished by applying unit loads to a mathematical model of the reactor building shell including various floor slabs. The loads were applied over the area of building shell covered by the girder ends, and in this way equivalent springs were generated to represent the restraint provided by the shell. These equivalent springs serve as the boundary conditions for the girder analysis described as follows.

2. Model the girder for the finite element analysis using the finite element capabilities of the STRUDL II computer code. Components of the girders were represented by plane strain finite elements to account for stretching and in-plane shear. In addition, plate bending elements were used to overlay the plane strain elements to account for transverse bending and shear.

3. Perform finite element analyses using the model generated in steps 1 and 2 for the loading cases previously described.

4. Integrate the stresses obtained from step 3 to obtain cross-sectional force resultants along the length of the girder and combine the individual loading conditions to obtain design values. The effects of earthquake-induced loads were incorporated into the design values by an equivalent static method where the applied loading was increased using the ZPA values from the seismic analysis.

5. Provide reinforcement in accordance with ACI-318 using ultimate strength methods to resist shears and moments generated above. Additional horizontal and vertical reinforcing was provided along the faces of the girder to resist shear loads as suggested in the ACI code under Special Provisions for Deep Girders. Bond and anchorage requirements were also determined based on ACI code allowables.

In addition to the preceding procedures, the effects of certain design variables that could influence the final results were

considered. These additional considerations were primarily the effects of shrinkage, creep, and concrete cracking on the resulting concrete strains and deflections and also the effect of the degree of end restraint on final design moments along the span of the girders. To ensure proper clearances, the deflections of the girders were calculated using the finite element results modified with appropriate factors for shrinkage, creep, and concrete cracking.

3.8.4.4.2 Control Room Building

The control room building is designed as a reinforced concrete structure supported on a mat foundation. Since the control building is structurally connected to the diesel generator building, a seismic analysis is performed by modeling both buildings as a unit.

The control building is designed for all postulated events and applicable load combinations outlined in Section 3.8.4.3 using conventional design procedures.

3.8.4.4.3 Diesel Generator Building

The diesel generator building is designed as a reinforced concrete structure supported on wall footings. The diesel generator building is integrally connected to and therefore modeled as a unit with the control building for determination of seismic response of the structures. The diesel generator fuel oil tanks are located underneath the structure and are encased in reinforced concrete.

The diesel generator support pedestals are isolated from the other portions of the structure at grade level and extend approximately 19 ft below grade, supported on rock. In addition to the dead and live loads and resulting moments and forces, the support pedestals are also designed to withstand seismic events and the moments and forces generated during the diesel generator startup.

The diesel generator building is designed for all postulated events and applicable load combinations outlined in Section 3.8.4.3, using conventional design procedures.

3.8.4.4.4 Screenwell Building

The screenwell building is used as a source of water to the recirculating and service water pumps. Reinforced concrete construction with a low water-cement ratio is used to minimize water leakage, and the reinforcing steel has a 3-in minimum protective cover where the concrete surface is in contact with water.

The structural components are designed for the load conditions that include dead loads, live loads, equipment loads, maximum

buoyant and uplift forces, crane load, loads during equipment handling and maintenance, and tornadic and seismic loads, as applicable, in accordance with the load combinations described in Section 3.8.4.3.

3.8.4.4.5 Intake Structures

The intake structures are designed for dead loads, live loads, maximum buoyant and uplift forces, forces generated by wave action (Section 3.4.2), and seismic loads.

Reinforced concrete construction with a low water-cement ratio is used to minimize water leakage, and the reinforcing steel has a 3-in minimum protective cover.

The concrete-encased steel tiedowns embedded into the rock are designed for maximum uplift forces generated due to the most critical environmental condition that can be postulated (Section 3.4).

3.8.4.4.6 Intake (and Discharge) Tunnels

The portions of the tunnels that encase the intake system are designed to withstand all possible loading effects including impact/impulse effects due to rock fall and thermal stresses. This encasement is analyzed and reinforced as an infinite beam resting on a continuous elastic foundation. The sides of the encasement are treated as columns with a maximum ductility factor of 1.3. All reinforcing steel has a 3-in minimum protective cover and high-strength concrete with a low water-cement ratio is used. The encasements are also designed for internal and external pressures and OBE and SSE seismic forces in combination with other loading, as described in Section 3.8.4.3.4.

3.8.4.4.7 Electrical Tunnels and Piping Tunnels

Electrical tunnels and piping tunnels housing safety-related systems are constructed of reinforced concrete. The tunnels are designed for gravity loads in addition to other applicable loads, such as hydrostatic loads, seismic loads, and earth pressure, in accordance with the load combinations described in Section 3.8.4.3. In addition to these loadings, the tunnel roof and walls are designed to carry the applicable crane loads resulting from the movement of cranes during and after construction. The tunnel roof is designed for surcharge loading, seismic or tornadic forces, whichever is critical, in addition to other applicable loadings in accordance with the load combinations outlined in Section 3.8.4.3.

3.8.4.4.8 Main Stack

The main stack (Figure 1.2-31) is analyzed in accordance with the methods and procedures outlined in ACI-307. This structure is not designed for a tornado; however, the distance between this

and other Category I plant structures is far enough to preclude damage to any Category I structures in the event the main stack structure collapses partially or completely (Figure 1.2-2). The main stack is designed to withstand seismic and other applicable forces from the load combinations of Section 3.8.4.3.

3.8.4.4.9 Standby Gas Treatment Building and Railroad Access Lock Area

The Category I portions of these structures are designed to meet all the load conditions described in Section 3.8.4.3, in addition to meeting shielding requirements for the area. The railroad access lock area is also designed to withstand moving wheel loads from a 66-ft long railroad car and a track-mobile engine.

3.8.4.4.10 Auxiliary Service Building

The auxiliary service building below el 261 ft msl is designed to withstand the loads and load combinations described in Section 3.8.4.3.

3.8.4.4.11 Radwaste Building

The radwaste building is designed as a reinforced concrete structure supported on a mat foundation. The foundation mat is analyzed and designed using the finite element capability of the STRUDL program.

The structural steel and reinforced concrete components of this building are designed in accordance with the AISC Manual of Steel Construction and ACI-318, respectively. The load combinations described in Section 3.8.4.3 are used in designing the structure, except that protection against tornadic events is not provided.

In the event of a postulated tank rupture, the base mat and exterior walls are designed to retain the spillage within the building. The base mat and the exterior concrete walls are lined with a steel liner up to el 242 ft 2 in to contain a spillage.

3.8.4.4.12 Turbine Building

The turbine building complex is constructed partially on spread footings and partially on a mat foundation. Structural steel and reinforced concrete components of this building are designed using load combinations in accordance with the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, and ACI-318, respectively. This building complex is constructed of reinforced concrete floors and walls up to the operating floor level. The structure above the operating floor level is constructed of a structural steel framing system braced by vertical and horizontal bracing systems up to roof level, enclosed by metal siding. A steel roof deck with roofing is provided at the top of the structure.

To prevent collapse of the steel superstructure on the nearby structures, the structural steel superstructure above the operating level is analyzed and designed using the finite element capability of the STRUDL computer program for the tornadic or seismic loads, whichever is critical, in addition to other gravity loads. The metal siding, roof decking, girts, etc., are assumed to blow away during a tornadic event; however, the main structural steel members, such as columns, beams, and bracing members, are designed to remain in place.

3.8.4.5 Structural Acceptance Criteria

For the steel structures, the allowable stresses and factors of safety are in accordance with the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, with the following exceptions:

1. Safety-related structures as identified in Table 3.2-1 are capable of withstanding the SSE loads in combination with applicable dead and live loads.

2. These safety-related structures are also checked using OBE loads in combination with applicable dead and live loads. For this loading condition, allowable stresses are the normal working stresses, instead of applying a 33-percent increase in allowable stresses, as allowed by AISC specification.

For concrete structures, the allowable stresses, load factors, and capacity reduction factors are in accordance with strength design methods of ACI-318-77. The required strength is expressed in terms of design loads, or their related internal moments and forces. Design loads are defined as loads that are multiplied by their appropriate load factors (safety factors). Calculated strength is that computed by the provisions of ACI-318, including the appropriate capacity reduction factors. Capacity reduction factors are taken as given in Section 9.3 of ACI-318.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

The materials for construction of Category I structures are procured, fabricated, and delivered to the site in accordance with the codes, standards, and specifications described in Section 3.8.4.2. The shipping, storage, and handling of materials during construction follow the requirements of ANSI N45.2.2. The major materials for construction of Category I structures are described herein. The major codes and the appropriate American Society for Testing and Materials (ASTM) standards used in procurement, fabrication, and testing of Category I materials are referenced herein, as applicable. The current editions of the ASTM standards adopted by the vendors' fabricating facilities at the time of procurement, fabrication, and testing of these materials are utilized.

3.8.4.6.1 Concrete

ACI-301, Specification for Structural Concrete for Buildings, together with ACI-347, Recommended Practice for Concrete Formwork, and ACI-318, Building Code Requirements for Reinforced Concrete, form the general basis for the concrete specifications. ACI-301 is supplemented as necessary with mandatory requirements relating to types and strengths of concrete, including minimum concrete densities, proportioning of ingredients, reinforcing steel requirements, joint treatments, and testing requirements.

Admixtures, types of cement, bonding of joints, embedded items, concrete curing, additional test specimens, additional testing services, cement and reinforcing steel mill test report requirements, and additional concrete test requirements are specified in detail.

All cement conforms to the Specification for Portland Cement, ASTM C 150, Type II, low alkali. Aggregates are tested initially and monitored throughout the construction phase of the project to assure that there is no adverse reaction between the cement and the aggregates. Initial examination of aggregates by ASTM C295 and tests by ASTM C289 and ASTM C227 (or ASTM C586) are used to make this determination. The ASTM C227 (or ASTM C586) test need not be completed prior to aggregate usage when ASTM C289 results are acceptable and when using low-alkali cement (Na_2O + 0.658 K_2O $\leq$0.6 percent). Certified copies of the mill test report, showing that the cement meets or exceeds the ASTM requirements for Portland cement, are furnished by the manufacturer. An independent testing laboratory is retained to perform periodic tests on the cement for compliance with the specifications.

An air-entraining agent is used in the concrete in an amount sufficient to satisfy ACI-301, Section 3.4.1. This agent conforms to the requirements of ASTM C260. Before using an air-entraining admixture, a certificate of compliance from the manufacturer is obtained stating that the admixture conforms to the applicable requirements, when tested in accordance with ASTM C233. The air content of concrete is tested at the site each time a set of concrete compressive strength specimens is made. Air-entrained cement is not used.

Mixing water and/or ice is clean and free from injurious amounts of oils, acids, alkalies, salts, organic materials, or other substances deleterious to concrete or steel. The mixing water is periodically checked and tested for suitability by comparing the results of ASTM C109, ASTM C151, and ASTM C191 with those obtained using distilled water. Each source of mixing water and/or ice is subject to these tests prior to use in production concrete, and every 6 months thereafter, to assure continued acceptability.

Fine and coarse aggregates conform to ASTM C33 when tested to the following ASTM and Corps of Engineers requirements:

ASTM C40	Organic Impurities
ASTM C88	Soundness
ASTM C117	Material Finer than the 200 Sieve
ASTM C123	Lightweight Pieces
ASTM C131 or C535	Los Angeles Abrasion
ASTM C136	Sieve Analysis
ASTM C142	Friable Particles
ASTM C227 and C289	Potential Alkali Reactivity
ASTM C295	Petrographic Examination
CRD C119	Flat and Elongated Particles

An independent testing laboratory tests the aggregates initially for conformance to all the above requirements. ASTM C87 is performed only after failure of ASTM C40 tests. ASTM C666 is performed only after failure of ASTM C88. In addition, during concrete production onsite, the independent testing laboratory performs ASTM C88, ASTM C289, and ASTM C131 or ASTM C535 tests every 6 months. Tests by ASTM C39 are performed only upon failure of ASTM C131 or ASTM C535 tests. When limestone aggregate is tested, ASTM C586 is used as a basis for acceptance.

The following tests are performed at the frequency indicated during concrete production onsite:

ASTM C136	Daily
ASTM C117	Daily
ASTM C566	Daily
ASTM C40	Weekly
ASTM C123	Monthly
ASTM C142	Monthly
ASTM C235	Monthly
CRD C119	At 6-month intervals

Sampling of aggregates conforms to ASTM D75. Tests for unit weight conform to ASTM C29. Tests for specific gravity and

absorption conform to ASTM C127 and ASTM C128 and are performed when these data are required. In addition to the above testing, appropriate tests are performed when a new source is to be used.

Proportioning of structural concrete conforms to ACI-301, Chapter 3. In general, structural concrete mixes have a 28-day minimum specified strength of 3,000 psi. When higher strength concrete is required, higher minimum specified strength concrete mixes are used.

Concrete used for shielding purposes, i.e., the majority of concrete used in floors, walls, roofs, and foundations, has a weight not less than 135 lb/cu ft, when air-dried in accordance with ACI-301 Section 3.3. Reference to lightweight concrete in ACI-301 Section 3.3 is also considered applicable to regular structural concrete in determining the unit weight of concrete. Proportions of ingredients for structural concrete mixes are determined and tests conducted in accordance with the method detailed in ACI-301 and ACI-211.1 for combinations of materials to be established by trial mixes.

Concrete protection for reinforcement, preparation, cleaning of construction joints, concrete mixing, delivering, placing, and curing is equal to or exceeds the requirements of ACI-301, with the following exceptions:

1. The maximum slump for massive concrete will be 3 in in general. However, up to 5-in maximum slump is permitted in congested areas to permit placing concrete in the heavily reinforced structures.

2. The minimum curing period is 1 week unless otherwise noted elsewhere in this section.

3. In lieu of Section 14.5.4, use Section 12.3.3.

4. Maximum placing temperature of the concrete when deposited conforms to the requirements of ACI-301 and ACI-305, Recommended Practice for Hot Weather Concreting, except for the placing of mass concrete. The placing temperature of mass concrete does not exceed 80°F. ACI-301 indicates placement of mass concrete sections to 70°F. This limit is based on concrete using standard or common Type I cement. Type II cement, which is used for this project, generates 80 to 85 percent of the heat of hydration of Type I cement. ACI-207 states that the heat-generating characteristic of Type II cement corresponds closely to that of Type I cement at 10°F lower placing temperatures.

5. All mass concrete placed at a temperature above 75°F is water cured in accordance with ACI-301, Chapter 12, and as described elsewhere in this section.

6. Section 4.3 of ACI-347 and Section 2.4 of ACI-347 form the basis for establishing formwork tolerances, except that when a steel plate (liner) is used for formwork, the liner tolerances will govern. Also, when the side of a wall opposite the steel liner is to be formed using something other than a liner, the theoretical form line will be established by measuring the thickness of the wall from the steel liner on the opposite face. Once the theoretical form line is established, the variation in thickness will be governed by the tolerances given in ACI-301 and ACI-347, as applicable.

Batching and mixing conform to ACI-301, Chapter 7, and ACI-304. Concrete ingredients are batched in a batch plant and transferred to transit mix trucks for mixing, agitating, and delivering to the point of placement, or are batched and mixed in a controlled mixer and transferred to a truck for delivery.

Placing of concrete is by bottom dump buckets, chuting, concrete pump, or conveyor belt. The rate of placing concrete is controlled so that concrete may be effectively placed and compacted by vibrating with particular attention given around embedded items and near the forms.

Vertical drops greater than 6 ft for any concrete are not permitted, except where suitable equipment is provided to prevent segregation.

In construction joints where keys are required by design, they are provided before the concrete has reached its final set. When keys are not provided, the surfaces of all construction joints are thoroughly cleaned by satisfactory means to remove laitance and to expose clean, sound aggregate. Excess water from joint cleaning not absorbed by the concrete is removed.

Horizontal construction joints are covered by a minimum 1/2-in thick layer of sand/cement grout, which has a compressive strength that is equal to or exceeds that of the concrete, and new concrete is then placed immediately against the fresh grout.

As an alternate to this procedure, a coating of surface retarder is applied to delay the setting of the concrete surface as described in subsections 6.1.4.2 and 6.1.4.3 of ACI-301. The horizontal surface is then prepared for the next pour by using high-pressure jet spray to remove the retarded mortar. The surface is then cut to expose the aggregates such that an irregular surface at least 1/4 in deep is exposed prior to placing the next layer of concrete.

Curing and protection of freshly deposited concrete conforms to the following:

1. Concrete to be cured with water is kept wet by covering with an approved water-saturated material, or by a system of perforated pipes or mechanical sprinklers, or by any other approved methods that will keep surfaces continuously wet. Water used for curing is generally clean and free from any elements that might cause objectionable effects.

2. The surfaces on which curing compounds may be used are specified. Curing compounds are not used on surfaces to which additional concrete is to be bonded.

3. All concrete is cured for at least 7 days unless otherwise noted in Items 5, 6, and 7 below. For all concrete, when the mean daily temperature of the surrounding air is less than 40°F, the temperature of the concrete is maintained at between 50°F and 70°F for the required curing period (7 days). Changes in temperature of the air immediately adjacent to the concrete during and immediately following the curing period are kept as uniform as possible and do not exceed 5°F in any 1-hr or 50°F in any 24-hr period.

4. For Category I watertight concrete, the reactor containment mat, the turbine pedestal and pedestal mat, and the large foundation mats 7 ft or greater in thickness, curing is with water for 7 days to maintain concrete surface moisture. For the dolosse used in the construction of the revetment ditch, the atmospheric pressure steam curing of concrete method is used in accordance with ACI-517-70.

5. For other Category I massive concrete sections not included in the preceding paragraph, the concrete is water cured for 48 hr following completion of the placement, then either a curing compound may be applied or water curing may be continued, and the temperature of the concrete maintained as above, until at least 7 days after the placement.

   Alternatively, for massive structures less than 7 ft thick, two field-cured cylinders kept adjacent to the placement and cured by the same methods may be tested for compressive strength at any time after the first 48 hr of curing. If the average compressive strength of the two cylinders tested equals or exceeds 70 percent of the specified compressive strength of the mix being used, then curing may be terminated.

6. Porous concrete containing calcium aluminate cement is water cured for a minimum of 24 hr. Two-course floor topping, deferred placement, is water cured for seven days.

7. All other Category I concrete not designated as massive concrete is cured for 7 days either by water curing or application of a curing compound immediately after removal of forms or finishing of exposed surface.

   However, for all other concrete, curing may be terminated when the average compressive strength of field-cured cylinders kept adjacent to the structure and cured by the same methods have reached 70 percent of the specified strength of the mix.

<u>Concrete Testing</u>

Compressive strength tests of concrete placed in Category I structures are performed in accordance with ACI-301, Technical Specifications, for every 100 cu yd of concrete or a minimum of one set per 8-hr shift, whichever is greater.

The test specimens for compressive strength are 6-in diameter by 12-in long cylinders. Each set consists of at least three specimens. At least one is tested after 7 days and two after 28 days or 60 days age, as applicable.

Concrete strength tests are evaluated in accordance with ACI-214, Recommended Practice for Evaluation of Compression Test Results of Field Concrete, and ACI-301, Chapter 17.

The strength of concrete is considered satisfactory as long as the frequency of occurrence of the following is less than 1 in 100 if:

1. The averages of all sets of three consecutive strength test results of the laboratory-cured specimens at the specified age is equal to or greater than the specified compressive strength, f'_c, of the concrete.

2. No individual strength test result falls below the specified strength, f'_c, by more than 500 psi.

The field tests for slump of concrete are in accordance with ASTM C143. Any batch not meeting specified requirements is rejected. Slump tests are made periodically during concrete placement and each time concrete compressive strength test specimens are taken.

If cylinders should fail to meet the concrete strength requirements at the specified age, strength development and design strength requirements are reviewed. An evaluation is performed and if required, core tests are conducted in accordance with ASTM C42, Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete. Should core tests be inconclusive or impracticable to obtain and structural analysis does not confirm the safety of the structure, load tests are performed. Concrete work judged inadequate by structural analysis or by load tests is

reinforced with additional construction or is removed and rebuilt.

Statistical quality control of the concrete is maintained by a computer program based on an article in ACI Publication SP-16, Computer Applications in Concrete Design and Technology. This program analyzes compressive strength test results by the testing laboratory in accordance with methods established by ACI-214, Recommended Practice for Evaluation of Strength Test Results of Concrete.

3.8.4.6.2 Reinforcing Steel

Reinforcing steel bars, sizes N3 through N6, conform to Grade 40, and N7 through N11 conform to Grade 40 or Grade 60 of the Standard Specification for Deformed Billet-Steel Bars for Concrete Reinforcement, ASTM A615 and Supplement S-1. For fuel pool beams and certain other areas of the reactor building, special large-size reinforcing bars, N14 and N18, conform to Grade 60 of ASTM A615. Grade 60 N8 reinforcing bars are used for missile enclosure of valves in the screenwell building.

Mill test reports showing actual chemical and physical properties, including bend tests, are furnished for each heat of steel used in making all reinforcing steel.

Except when otherwise noted herein, reinforcing bars N14 and N18 will be controlled chemistry steel of 50,000 psi minimum yield point, conforming to the Standard Specification for Deformed Billet-Steel Bars for Concrete Reinforcement, ASTM A615, as modified to meet the following chemical and physical requirements:

Carbon	0.35 percent maximum
Manganese	1.25 percent maximum
Silicon	0.15 to 0.25 percent
Phosphorous	0.05 percent maximum
Sulfur	0.05 percent maximum
Minimum yield strength	50,000 psi
Elongation	13 percent minimum in an 8-in test sample
Tensile strength	70,000 psi minimum

For these special chemistry bars, all ingots are identified and all billets are stamped with identifying heat numbers. All bundles of bars are tagged with a heat number as they come off

the cooling bed of the rolling mill. A special mark is rolled into all bars conforming to this special chemistry to identify them as possessing the chemical and mechanical qualities specified. The chemical variations allowed for special chemistry bars are in accordance with ASTM A29.

Placing of reinforcing steel conforms to the requirements of Chapter 5 of ACI-301, Structural Concrete for Buildings, and Chapter 7 of ACI-318, Building Code Requirements for Reinforced Concrete.

Tack welding of designed reinforcing steel that does not become an integral part of the weldment is not permitted.

Structural ductility is maintained by staggering critical splices wherever possible to ensure that small adverse effects of multiple splices in the same plane will not occur. Full-scale pressure tests conducted in May 1967 on a completed concrete containment structure, in which cadweld splices and welded splices were used in a similar manner to that proposed here, showed no stress concentrations or lack of structural ductility. Locations of splice groups were not discernible from inspection of the test crack patterns.

Reinforcing Steel Inspection and Testing

The engineers' inspectors witness, on a random basis, the pouring of the heats and the physical and chemical tests performed by the manufacturer for the special chemistry reinforcing steel. Bars failing to conform to required chemistry and physical requirements are rejected.

Mill test reports showing actual chemical ladle analysis, physical properties, bend test, and variations in weight will be obtained from the manufacturer for each heat. In addition, confirmatory tensile tests for each 50 tons of every heat of steel for every bar size will be made to determine physical properties.

Full-size test specimens of all rebars are tested on a testing machine using an 8-in gauge length. The loading rate for these tests is as specified in ASTM A370. The acceptance standards are in accordance with ASTM A615. At least one full diameter specimen from each bar size is tested for each 50 tons, or fraction thereof, of reinforcing bars produced from each heat.

The preceding frequency of testing, test procedures, and acceptance standards conform to RG 1.15. The degree of compliance to RG 1.15 is discussed in Section 1.8.

Mechanical Rebar Splicing Systems

The mechanical splice criteria follow the requirements of NRC RG 1.10, Rev. 1, as discussed in Section 1.8. The design basis

described in the Preliminary Safety Analysis Report (PSAR) and approved by the NRC in the Safety Evaluation Report (SER) for the construction permit included the use of this regulatory guide. Use of this regulatory guide provides adequate assurance that the Category I structures perform their intended safety function.

Cadweld Splices Cadweld reinforcing steel splices, manufactured by Erico Products, Inc., Cleveland, OH, are used to splice N14 and N18 reinforcing bars. All cadweld splices are made in accordance with the instructions for their use issued by the manufacturer. The degree of compliance to RG 1.10 is discussed in Section 1.8. In areas where space or other requirements make cadweld splices unsuitable, N14 and N18 reinforcing bars are butt-welded in a manner conforming to the requirements of AWS D12.1.

Reinforcing bars No. 11 and smaller are generally lap spliced. Where lap splicing is impractical, and where threaded rebar splices are not used, splicing is accomplished by:

1. Cadweld as manufactured by Erico Products, Inc., or equal, using the sleeves that develop the full tensile strength of the reinforcing bars, or

2. Butt-welding in accordance with the requirements of AWS D12.1.

In order to qualify Operators for making cadweld production joints, each Operator is required to prepare two satisfactory qualification splices for each of the splice positions to be used. Testing is by tensile testing a cadweld that simulates field conditions and uses the same materials as those to be used in the structure.

The ends of the reinforcing steel bars to be joined by the cadweld process are saw cut, flame cut, or shear cut. The ends of the bars are thoroughly cleaned of all rust, scale, grease, oil, water, or other foreign matter before the joints are made.

Cadweld Testing and Inspection Cadweld process splices are visually inspected in accordance with RG 1.10. Visual inspection includes random inspection of the ends of the bars for dryness and cleanliness prior to fitting the sleeve over the ends.

Inspection is made of the completed splice for properly filled joints that have filler metal visible at both ends of the sleeve for T-series splices and the exposed end for B-series splices and at the tap hole in the center of the sleeve. Splices that do not meet all these inspection criteria are rejected.

Randomly selected cadweld splices based on separate test cycles for horizontal, vertical, and diagonal bars, size of rebar, and cadwelder are removed from the structure and tensile tested, or a combination of production and sister splices are tested in

accordance with ASTM A370. Testing is in accordance with the following schedule if only production splices are tested:

1 out of first 10 splices.

1 out of next 90 splices.

2 out of next and each subsequent unit of 100 splices.

If combinations of production and sister splices are tested, the sample frequency is as follows:

1 production splice out of the first 10 production splices.

1 production and 3 sister splices out of the next 90 production splices.

3 splices, either production or sister splices, for the next and subsequent units of 100 splices. At least one-fourth of the total number of splices tested are production splices.

The sample frequency for splices in curved bars with a radius of curvature less than 60 ft is as follows:

1 sister splice for the first 10 production splices.

4 sister splices for the next 90 production splices.

3 sister splices for the next and subsequent units of 100 production splices.

Sister splices are made using straight bars.

The tensile strength of each sample tested should equal or exceed 125 percent of the specified minimum yield strength for the grade of reinforcing bar used. Failure of any splice to achieve 125 percent of the specified minimum yield strength is evaluated in accordance with Section 5 of the Procedure for Substandard Tensile Test Results as given in RG 1.10.

<u>Dywidag Threadbar System Splices</u> Dywidag threadbar system splicing is used, on a limited basis, for bar sizes No. 6 through and including No. 11 in structures, and will conform to the mechanical splice criteria of ACI-318 Building Code Requirements for Reinforced Concrete.

All Dywidag splices are made in accordance with the instructions for their use issued by the manufacturer, Dywidag Systems International, Lincoln Park, NJ.

Unstaggered Dywidag splices may be used in the radwaste building. Dywidag threadbar splice system is not used in any QA Category I structure.

Welding of Reinforcing Steel

All welding of reinforcement (i.e., rebar to plate or rebar to rebar) conforms to Recommended Practices for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction, AWS D12.1. Certified material test reports for welding electrodes are obtained from the electrode manufacturer. The ends of the bars to be joined by butt welding are prepared by saw cutting and dressing by grinding, where necessary. In order to qualify welders for work on the reinforcing steel bars, each welder makes test welds in each position he will be required to use during production. Each test weld is tension tested and each is required to meet or exceed the minimum tensile strength of the reinforcing bar. Structural ductility is maintained by staggering critical splices wherever possible to assure that small adverse effects of multiple splices in the same plane do not occur.

Generally, welding of reinforcing steel bars is employed in isolated instances only when mechanical (cadweld) splicing is not feasible due to space restrictions or clearance problems. Specifically, the reinforcing bars in size Nos. 4 to 18 are welded to the steel plates, etc., as required due to space restriction or clearance problems or to provide anchorage.

In all cases, the weld details are prequalified in accordance with AWS D12.1. Welding of reinforcing steel bars to plates is used in construction of primary containment, drywell floor, reactor pedestal, and screenwell building.

Although welding of N14 and N18 rebars is permitted, no rebar to rebar welding has been used on seismic Category I structures.

With the exception of main steam tunnel area, rebar to rebar butt welding has not been used in any other seismic Category I structure. In main steam tunnel area 1-No. 7 and 32-No. 8, rebar dowels at el 239'-0", 244'-0", and 250'-0" were butt welded to the same size bars.

Inspection and Testing of Reinforcing Steel Welds

All welds are visually inspected. Any cracks, porosity, or other defects are removed by chipping or grinding until sound metal is reached, and then repaired by welding. Peening is not permitted. Completed welded joints in reinforcing steel are selected on a random basis from Category I structures and radiographically inspected in accordance with the following schedule:

1 out of first 10 splices.

3 out of next 100 splices.

1 out of next and subsequent units of 100 splices.

Cracks and any excessive amount of contained voids, as specified in AWS D12.1, are cause for repair or removal and replacement. Replaced welds are examined in a similar manner. Reinforcing steel bars welded to steel embedments are tested by sister splice, in accordance with the following schedules:

1 sister splice out of the first 10 production splices.

4 sister splices for the next 90 production splices.

3 sister splices for the next and each subsequent unit of 100.

3.8.4.6.3 Structural Steel

Structural steel material and fabrication tolerances are in accordance with the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, including the supplements (Section 3.8.4.2). Structural steel erection tolerances are in accordance with Code of Standard Practice for Buildings and Bridges, except that for columns other than crane columns and elevator hoistway columns, 1 1/2 in tolerance at any point in its height is permitted. In general, steel used for structural framing conforms to ASTM A36. In areas where the design indicates that a higher strength steel is required, ASTM A440, A441, A572, A588, or A242 steel is used. In the suppression pool area, stainless structural steel conforming to ASTM A167 is used.

Certified copies of mill test reports showing actual chemical and physical properties are furnished for each heat of steel used in making Category I structural steel.

Welding of structural steel is in accordance with AWS D1.1 with the following clarifications:

AWS D1.1 Section 2.4.3 requires fillers 1/4 in and larger to be extended beyond the edge of the splice plate or connection material. However, fillers greater than or equal to 1/4 in are installed using requirements of AWS D1.1 Section 2.4.2 in the following conditions:

1. Restrictions due to space limitations.

2. Presence of tapered gaps due to fabrication or erection tolerances requiring use of multiple shims.

3. When stresses cannot be transferred through the filler plates.

The structural welding code contains the requirement that undercut will not exceed 0.01 in deep when the direction is transverse to the primary tensile stress in the part that is

undercut. Unless so noted, all welding performed under the AWS Code is inspected for a maximum undercut of 1/32 in.

Inspections to AWS D1.1 may be performed with the exception that the inspector need not identify with a distinguishing mark all parts or joints that he has inspected and accepted. These inspections must be documented by the Contractor's QA program.

ASTM A515 GR 65 may be considered an AWS D1.1, prequalified group no. 1 material.

AWS D1.1 Section 3.3.1 requires that the leg of a fillet weld be increased if the separation between the parts to be joined is 1/16 in or greater. Unless noted, all welding performed under the AWS Code shall have the leg of the fillet weld increased if the separation is greater than 1/16 in.

Alternatively, the visual inspection of AWS D1.1 structural welds for the items listed below may be performed using the criteria in Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants (VWAC), issued by the NCIG as described in NCIG-01, Rev. 2, May 7, 1985 (including the corresponding NCIG training). This document is accepted by the NRC in their letter of May 26, 1985, from J. P. Knight of NRC to D. E. Dutton of NCIG. The implementation date of NCIG-01 Revision 2 is documented on the appropriate specification change approval documents. The items are:

1. Structural steel.
2. Cable tray supports.
3. Conduit supports.
4. Duct supports.
5. Instrumentation supports.
6. Equipment supports.

The material installation and inspection of high-strength bolts conform to the requirements of the Specification for Structural Joints using ASTM A325 or A490 Bolts.

Welder performance qualification in accordance with ASME IX shall qualify the welder to perform welding to AWS D1.1. The welder performance qualification in accordance with ASME IX will meet all essential variables of AWS D1.1, with the following exceptions:

1. The performance test coupon with a 37 1/2 deg ± 2 1/2 deg bevel is acceptable in lieu of the 22 1/2 deg ± 2 1/2 deg bevel required by AWS D1.1.

2. In addition to welder performance qualification established on material permitted by AWS D1.1, welder performance qualification on an ASME IX P1-listed material shall qualify the welder to weld AWS D1.1 prequalified materials.

3.8.4.7 Testing and In-service Surveillance Requirements

No full-scale structural testing or in-service surveillance is anticipated for the structures described in Section 3.8.4.1. For testing of the materials used in construction, refer to Section 3.8.4.6.

3.8.5 Foundations and Concrete Supports

3.8.5.1 Description of the Foundation and Supports

Table 3.8-13 lists the foundation systems that are used for major Category I structures. Major Category I structures are founded on or below natural bedrock surface. Foundations with bases on or within the top 10 ft of the natural bedrock are designed for an allowable bearing load of 10 tons/sq ft. Foundations with bases deeper than 10 ft below the natural bedrock surface are designed for an allowable bearing load of 20 tons/sq ft. When Category I structures (such as electrical duct lines) are founded on Category I structural fill, they are designed to satisfy the Category I loading combinations as described in Section 3.8.4.3. Allowable bearing pressure on the Category I structural fill is 2 tons/sq ft.

The normal design level for groundwater is el 255 ft msl. The maximum design flood water level is el 261 ft msl. The foundations of most major Category I structures are below the groundwater level. Waterstops are provided at vertical and horizontal construction joints below the flood level of 261 ft msl to prevent seepage of water through the joints.

All Category I structures are constructed in such a manner that a minimum 6-in space is provided between the extremities of the foundations and the excavated rock surfaces. This space is filled with compressible materials such as vermiculite, vermiculite concrete, or compressible filler material. Additional information is contained in Section 2.5.4.

3.8.5.1.1 Reactor Building

The reactor building, primary containment, reactor support pedestal, and auxiliary bays are founded on a common mat, sitting on rock at el 164 ft. The mat is a reinforced concrete structure 10 ft thick and 183 ft in diameter with two rectangular auxiliary bay wings. Figure 3.8-20 shows the foundation mat configuration. The mat is reinforced with both top and bottom layers of reinforcing steel (Figures 3.8-22 and 3.8-23). Shear reinforcing

steel, for radial shear forces, is placed in the vertical direction. The reinforcement for the bottom of the mat is in an orthogonal grid pattern with layers at 90 deg to each other with some additional bars in the radial direction. Reinforcement for the top of the mat consists of concentric circular bars and radial bars. The reinforcing pattern for the top of the mat is arranged to maintain a uniform spacing of the bars which extend into the mat from the vertical walls above. Mat reinforcing bars are not spliced at the junction of the mat and vertical walls.

The reactor support pedestal, the primary containment wall, and the secondary containment wall are adequately connected to the mat to resist discontinuity moments and shears by the use of vertical reinforcing dowels (Figure 3.8-20).

At least 8 in of porous concrete topped by 1/4 in of seal concrete layer is placed under the reactor building mat to intercept the groundwater, which is then channeled through 6-in porous concrete pipes to the sumps located below the mat. In addition to this, an independent water collection piping system, using half-round 8-in diameter pipes, is provided near the top of the mat to collect the leakage into two standpipes in the reactor building. The details of the groundwater drainage system for the reactor building and vicinity area are covered in Section 2.4.13.

3.8.5.1.2 Foundations for Other Structures

Foundations for all major Category I structures and the turbine building are either reinforced concrete mats or spread footings. In some instances the Category I structures are supported by the underlying Category I tunnels, which in turn are founded on bedrock using a reinforced concrete mat system. These building foundations are listed in Table 3.8-13.

Since major Category I structures are founded on natural bedrock, no potential for liquefaction exists. Category I structures, when founded on Category I structural fill, are evaluated against the possibility of any liquefaction (Section 2.5.4.8). Figure 3.8-21 shows a typical reinforcing pattern at the junction of reinforced concrete vertical structural elements and a foundation mat.

3.8.5.2 Applicable Codes, Standards, and Specifications

The design codes, standards, specifications, and regulations that are used for the design and construction of Category I foundations, including the reactor building mat, are listed in Section 3.8.4.2.

3.8.5.3 Loads and Load Combinations

The loads and loading combinations for the reactor building mat are the same as those for the primary containment structure (Section 3.8.1.3). The loads and loading combinations for the

foundations of other Category I structures are the same as those used in designing the Category I structures (Section 3.8.4.3).

In addition, the following load combinations are used to check against sliding and overturning due to earthquakes, winds, and tornadoes, and against flotation due to floods:

1. D + H + OBE

2. D + H + W

3. D + H + SSE

4. D + H + W_t

5. D + F'

Where:

D, OBE, W, SSE, and W_t (defined in Section 3.8.4.3)

H = Lateral earth pressure

F' = Buoyant force of the design basis flood

3.8.5.4 Design and Analysis Procedures

The reactor building mat is analyzed and designed for the loading combinations defined in Section 3.8.5.3. The MAT-6 program, a digital computer program based upon the general methods described in Appendix 3A, is used to determine the stresses in the mat due to statically applied axisymmetric loads. This program analyzes an axisymmetrically loaded circular plate on an elastic foundation and maintains compatibility between the plate and concentric walls supported by the plate. The mat analysis includes the effects of the primary containment pressure loads generated by the DBA, hydrodynamic loads, loads from temperature due to operating conditions and the DBA, stiffness characteristics of the cylindrical shells that are considered as elastic constraints on the mat, dead loads, and characteristics of the supporting media. The subgrade stiffness is based upon the Boussinesq theory, which assumes the subgrade to be a homogeneous isotropic elastic medium. The discontinuity moments and shears at the junctions of the primary containment, secondary containment, and reactor pedestal wall with the mat are computed by the program by applying compatibility conditions at the interface of the mat with each of the above. Appendix 3A presents the design control measures that have been employed to demonstrate the applicability and validity of the MAT-6 program.

Dynamic analysis of the reactor building provides acceleration profiles for the reactor building which are applied as static loads on the structure. Since these loads are asymmetric, the

mat is analyzed using SHELL 1, a finite-difference computer program (Appendix 3A).

Since both the MAT-6 and SHELL 1 programs can handle only axisymmetric structures, the effect of auxiliary bays on the mat is investigated by three-dimensional finite element analysis using the ICES STRUDL II computer program. The results of this analysis are incorporated in the mat design. The mat is analyzed for hydrodynamic loads which are described in the Design Assessment Report for Hydrodynamic Loads (DAR) (Appendix 6A).

The foundations of other major Category I structures are analyzed by either hand calculations or using the finite element capability of the STRUDL II/STARDYNE computer programs (Appendix 3A). The foundations of small structures such as tunnels are analyzed as framed structures either using the STRUDL II computer program or by hand calculations. The loads on spread footings for walls and columns are obtained by analyzing the structure using the frame analysis approach. All the foundations of Category I structures are analyzed and designed for the applicable load combinations (Section 3.8.4.3).

3.8.5.5 Structural Design Criteria

Structural design of all foundations, including the reactor building mat, is in accordance with ACI-318, using ultimate strength design. Capacity reduction factors are used as given in Chapter 9 of ACI-318.

The structural analysis of the Category I structures provides factors of safety against overturning, sliding, and flotation and are as follows:

	Minimum Factors of Safety		
Load Combination	Overturning	Sliding	Flotation
1. D + H + OBE	1.5	1.5	NA
2. D + H + W	1.5	1.5	NA
3. D + H + SSE	1.1	1.1	NA
4. D + H + W_t	1.1	1.1	NA
5. D + F'	NA	NA	1.1

Major Category I plant structures are checked for overturning, sliding, and flotation. Table 3.8-14 provides a summary of stability analyses of major Category I structures.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

Materials, quality control, and special construction techniques used for the construction of foundations are the same as for other Category I structures (Section 3.8.4.6).

3.8.5.7 Testing and In-service Surveillance Requirements

Testing and ISI is not planned for any foundation structure.

3.8.6 Reference

1. Timoshenko, S. P. and Gere, J. M. Theory of Elastic Stability, 2nd Edition. McGraw-Hill Book Company, New York, NY, 1961.

TABLE 3.8-1
(Sheet 1 of 6)

LOAD COMBINATIONS FOR CONCRETE PRIMARY CONTAINMENT

PART I

Load Combinations Without SRV Discharge Loading

Service Conditions	Load Combination
Test	S = 1.0 D + 1.0 L + 1.15 Pa + 1.0 To
Construction	S = 1.0 D + 1.0 L + 1.0 To
Design Conditions	
Normal	U = 1.4 D + 1.7 L + 1.3 (To+Ro)
Severe environmental	U = 1.4 D + 1.7 L + 1.9 E + 1.3 (To+Ro)
Extreme environmental	U = 1.0 (D+L+To+Ro+E')
Abnormal	U = 1.0 (D+L+Ta+Ra) + 1.5 Pa
Abnormal/severe environmental	U = 1.0 (D+L+Ta+Ra+Rj+Rr+Rm) + 1.25 (E+Pa) U = 1.0 (D+DF+L+TF+E)
Abnormal/extreme environmental	U = 1.0 (D+L+Ta+Ra+Rj+Rr+Rm+Pa+E')

PART II

Load Combinations With SRV Discharge Loading

To account for the effects of SRV discharge loading and suppression pool hydrodynamic loading, the primary containment is checked for the following combinations:

Design Conditions	Load Combination
Normal without temperature	U = 1.4 D + 1.7 L + 1.0 Po + 1.5 SRV_{seq} U = 1.4 D + 1.7 L + 1.0 Po + 1.5 SRV_{asy}

TABLE 3.8-1
(Sheet 2 of 6)

LOAD COMBINATIONS FOR CONCRETE PRIMARY CONTAINMENT

Design Conditions	Load Combination
Normal with temperature	U = 1.0 D + 1.3 L + 1.0 Po + 1.0 To + 1.0 Ro + 1.3 SRV_{seq} U = 1.0 D + 1.3 L + 1.0 Po + 1.0 To + 1.0 Ro + 1.3 SRV_{asy}
Normal/severe environmental	U = 1.0 D + 1.0 L + 1.0 Po + 1.0 To + 1.0 Ro + 1.25 E + 1.25 SRV_{seq} U = 1.0 D + 1.0 L + 1.0 Po + 1.0 To + 1.0 Ro + 1.25 E + 1.25 SRV_{asy}
Abnormal	U = 1.0 D + 1.0 L + 1.25 Pb + 1.0 Ta + 1.0 Ra + 1.25 SRV_{ads} U = 1.0 D + 1.0 L + 1.25 Pb + 1.0 Ta + 1.0 Ra + 1.25 SRV_{asy} U = 1.0 D + 1.0 L + 1.25 Pa + 1.0 Ta + 1.0 Ra + 1.0 SRV_{sngl}
Abnormal/severe environmental	U = 1.0 D + 1.0 L + 1.1 E + 1.1 Pb + 1.0 Ta + 1.0 Ra + 1.1 SRV_{ads} U = 1.0 D + 1.0 L + 1.1 E + 1.1 Pb + 1.0 Ta + 1.0 Ra + 1.1 SRV_{asy} U = 1.0 D + 1.0 L + 1.1 E + 1.1 Pa + 1.0 Ta + 1.0 Ra + 1.0 SRV_{sngl}
Normal/extreme environmental	U = 1.0 D + 1.0 L + 1.0 Po + 1.0 To + 1.0 Ro + 1.0 E' + 1.0 SRV_{seq} U = 1.0 D + 1.0 L + 1.0 Po + 1.0 To + 1.0 Ro + 1.0 E' + 1.0 SRV_{asy}
Abnormal/extreme environmental	U = 1.0 D + 1.0 L + 1.0 E' + 1.0 Pb + 1.0 Ta + 1.0 Ra + 1.0 Rj + 1.0 Rm + 1.0 Rr + 1.0 SRV_{ads} U = 1.0 D + 1.0 L + 1.0 E' + 1.0 Pb + 1.0 Ta + 1.0 Ra + 1.0 Rj + 1.0 Rm + 1.0 Rr + 1.0 SRV_{asy} U = 1.0 D + 1.0 L + 1.0 E' + 1.0 Pa + 1.0 Ta + 1.0 Ra + 1.0 Rj + 1.0 Rm + 1.0 Rr + 1.0 SRV_{sngl}

TABLE 3.8-1
(Sheet 3 of 6)

LOAD COMBINATIONS FOR CONCRETE PRIMARY CONTAINMENT

NOTES

1. Normal wind and tornado loads do not have an effect on the structure since it is surrounded by the reactor building wall, which is designed for tornadic loading (including missiles).

2. Variations in the dead load of the structure are provided for by using 5 percent of the dead load coefficient provided in the above formulas.

3. Normal Loads Loads encountered during plant operations and shutdown.

   D = Dead loads, including hydrostatic and permanent equipment loads.

   L = Live loads, including any movable equipment loads and other loads that vary with intensity and occurrence, such as soil pressures.

   To = Thermal effects and loads during normal operating or shutdown conditions as applicable, based on the most critical transient or steady-state condition.

   Ro = Pipe reactions during normal operating or shutdown conditions as applicable, based on the most critical transient or steady-state condition.

   Po = Pressure load during normal operating condition.

4. Construction Loads Loads applied to the structure from start to completion of construction including the loads generated during the erection of the RPV. D, L, and To are defined in Note 3. Normal loads are applicable, but the construction value is used.

5. Test Loads Loads applied to the structure during the structural integrity test. D, L, and To are defined in Note 3. Normal loads are applicable. Pa is the DBA pressure load as defined in Note 8.

6. Severe Environmental Loads Events and the resulting loads occurring only infrequently. The loads associated with flooding the containment with water and the OBE are included in this category. The definitions in Note 3 apply, in addition to the following:

TABLE 3.8-1
(Sheet 4 of 6)

LOAD COMBINATIONS FOR CONCRETE PRIMARY CONTAINMENT

E = Loads due to acceleration from the OBE and including the lateral or vertical acceleration, or a combination of both, where the effects (as measured by the stresses resulting from the separate acceleration components) of lateral and vertical ground accelerations are combined algebraically.

DF = Hydrostatic loads associated with flooding the containment with water.

TF = Thermal loads associated with flooding the containment with water.

7. Extreme Environmental Loads Highly improbable events and the resulting loads.

8. Abnormal Loads Loads generated by the DBA.

Pa = Design pressure load within the containment generated by the DBA, including LOCA.

Pb = Design pressure load generated by SBA or IBA.

NOTE: Pa and Pb include the suppression pool hydrodynamic loads from effects of chugging and/or condensation oscillation.

Ta = Thermal effects and loads generated by the DBA including To.

Ra = Pipe reaction from thermal conditions generated by the DBA including Ro.

Rr = Load on the containment generated by the DBA, e.g., reaction of a ruptured high-energy pipe during the postulated event. The time-dependent nature of the load and the ability of the structure to deform beyond yield are considered in establishing the structural capacity necessary to resist the effects of Rr.

Rj = Load on the containment generated by the DBA, e.g., jet impingement from a ruptured high-energy pipe during the postulated event. The time-dependent nature of the load and the ability of the structure to deform beyond yield are considered in establishing the structural capacity necessary to resist the effects of Rj.

TABLE 3.8-1
(Sheet 5 of 6)

LOAD COMBINATIONS FOR CONCRETE PRIMARY CONTAINMENT

Rm = Equivalent static missile impact load acting on a structure generated by or during the postulated accident. Load includes an appropriate dynamic load factor applied to the peak of the missile impact-time curve.

E' = Load due to acceleration from the safe shutdown earthquake and including the lateral or vertical acceleration, or combination of both, where the effects (as measured by the stresses resulting from the separate acceleration components) of lateral and vertical ground accelerations are combined algebraically.

9. Abnormal/Severe Environmental Loads Combinations that result from the postulated combined occurrence of abnormal and severe environmental effects, when the occurrence of a specified severe environmental category load condition at the plant site imposes effects that significantly increase the probability of abnormal category load conditions, or when the specified abnormal or severe environmental category load condition is of such extended duration that a significant probability exists that loads in these two categories will occur simultaneously.

10. Abnormal/Extreme Environmental Loads Combinations that result from the postulated combined occurrences of abnormal and extreme environmental effects, when the occurrence of a specific extreme environmental category load condition at the plant site imposes effects that significantly increase the probability of the occurrence of abnormal category load conditions, or when the specified abnormal or extreme environmental category load condition is of such extended duration that a significant probability exists that loads in these two categories will occur simultaneously.

11. U is the required section strength based on the strength design methods described in ACI-318.

    In computing the required section strength, actual compressive strength of concrete may be used in place of specified minimum compressive strength of concrete. The actual compressive strength of concrete shall be determined from the compressive strength test reports of the concrete pour under consideration. This provision may be used to establish design adequacy in isolated

TABLE 3.8-1
(Sheet 6 of 6)

LOAD COMBINATIONS FOR CONCRETE PRIMARY CONTAINMENT

cases where the specified minimum strength of the material is potentially exceeded.

12. S is the required section strength based on alternate design method and the allowable stress described in ACI-318.

13. SRV discharge loads are defined as shown below:

SRV_{sea} = SRV loads due to sequential (i.e., all valves) actuation.

SRV_{ads} = SRV loads due to automatic depressurization system (ADS) (seven valves) actuation.

SRV_{asy} = SRV loads due to asymmetric (three valves) actuation.

SRV_{sngl} = SRV loads due to single valve actuation.

TABLE 3.8-2
(Sheet 1 of 2)

LOAD COMBINATIONS FOR CONTAINMENT WALL LINER, FLOOR LINER, CORNER TRANSITION REGIONS, AND EMBEDMENT LINER INSERT PLATES

Category	Load Combination	Stress Comparison
Design I	a. D + PD + OBE + SRV + LOCA b. D + PD + OBE + SRV + LOCA + J	Pm ≤ Sm @ TD Pl ≤ 1.5 Sm @ TD Pl + Pb ≤ 1.5 Sm @ TD Same as Item a. except Sm = 85% of ASME III Appendix F allowables
Design II	a. D + PD + SSE + SRV + LOCA b. D + PD + SSE + SRV + LOCA + J	Pm ≤ larger of 1.2 Sm or Sy @ TD Pl ≤ larger of 1.8 Sm or 1.5 Sy @ TD Pl + Pb ≤ larger of 1.8 Sm or 1.5 Sy @ TD Same as Item a. except Sm = 85% of ASME III Appendix F allowables
Design III	a. D + PE + OBE b. D + PE + SSE	Pm ≤ Sm @ TE Pl ≤ 1.5 Sm @ TE Pl + Pb <1.5 Sm @ TE ASME III instability criteria Pm ≤ larger of 1.2 Sm or Sy @ TE Pl ≤ Pb ≤ larger of 1.8 Sm or 1.5 Sy @ TE 1.2 times (ASME III instability criteria)
Design IV	D + PF + OBE	Pm ≤ Sm @ TF Pl ≤ larger of 1.8 Sm or 1.5 Sy @ TF Pl + Pb ≤ larger of 1.8 Sm or 1.5 Sy @ TF
Operating I	D + Poc + To + OBE + SRV	Pl + Pb + Q ≤ 3 Sm @ To Pl + Pb + Q + F (fatigue evaluation)
Operating II	D + Poc + To + DBE + SRV	Pl + Pb + Q ≤ 3 Sm
Test	D + PT	Pm ≤ 0.85 Sy @ TT Pl + Pb ≤ 1.25 Sy @ TT

TABLE 3.8-2
(Sheet 2 of 2)

LOAD COMBINATIONS FOR CONTAINMENT WALL LINER, FLOOR LINER, CORNER TRANSITION REGIONS, AND EMBEDMENT LINER INSERT PLATES

LOAD AND SYMBOL DEFINITIONS:

PD	=	Design internal pressure of containment
PE	=	Containment external pressure
Poc	=	Operating internal pressure of containment
PF	=	Design pressure for flooded condition
PT	=	Primary containment pneumatic test pressure
D	=	Deadweight and other sustained loads
J	=	Local and general effects from jet impingement due to pipe break (PD and TD)
RT	=	Pipe rupture load
LOCA	=	Maximum dynamic loads from the suppression pool due to the postulated loss-of-coolant accident
SRV	=	Maximum dynamic loads from the suppression pool due to safety/relief valve blowdown
OBE	=	Load due to operating basis earthquake
SSE	=	Load due to safe shutdown earthquake
TO	=	Thermal loads from containment and/or from piping due to piping system expansion and anchor movements
Pm, PL, Pb, Q, F	=	Calculated stresses as defined in ASME Section III, NE-3200
Sm	=	Allowable stress intensity values at temperature of the consideration from ASME Section III Tables I-1.0 and I-10.0 for Class 1 and Class MC, respectively
Sy, Su	=	Yield strength and ultimate strength from Appendix I of ASME Section III

TABLE 3.8-3
(Sheet 1 of 4)

LOAD COMBINATIONS FOR ACCESS LOCKS AND HATCHES

PART I

Where Structure Is Integral and Continuous

Design Category	Load Combination	Stress Comparisons	ASME Paragraph
Design I	D + PD + OBE + SRV + LOCA	Pm ≤1.0 Sm @ TD P ≤1.5 Sm @ TD Pl + Pb ≤1.5 Sm @ TD	NE-3131 (C)
Design II	D + PD + SSE + SRV + LOCA	Pm ≤larger of 1.2 Sm or Sy @ TD Pl ≤larger of 1.8 Sm or 1.5 Sy @ TD Pl + Pb ≤larger of 1.8 Sm or 1.5 Sy @ TD	NE-3131 (C)
Design III	D + PE + OBE	Pm ≤1.0 Sm @ TE Pl ≤1.5 Sm @ TE Pl + Pb ≤1.5 Sm @ TE ASME III instability criteria	NE-3131 (C) NE-3133, NE-3131 (C)
	D + PE + SSE	Pm ≤larger of 1.2 Sm or 1.0 Sy @ TE Pl ≤larger of 1.8 Sm or 1.5 Sy @ TE Pl + Pb ≤larger of 1.8 Sm or 1.5 Sy @ TE 1.2 (Instability value from NE-3133)	NE-3131 (C) NE-3133, NE-3131 (C)
Design IV	D + PF + OBE	Pm ≤1.5 Sm @ TF Pl ≤larger of 1.8 Sm or 1.5 Sy @ TF Pl + Pb ≤larger of 1.8 Sm or 1.5 Sy @ TF	
Operating I	D + Poc + To + OBE + SRV + LOCA	Pl + Pb + Q ≤3 Sm Pl + Pb + Q + F (fatigue evaluation)	NE-3222
Operating II	D + Poc + To + SSE + SRV + LOCA	Pl + Pb + Q ≤3 Sm	NE-3222
Test	D + PT	Pm ≤0.85 Sy @ TT Pl + Pb ≤1.25 Sy @ TT	NE-6322

TABLE 3.8-3
(Sheet 2 of 4)

LOAD COMBINATIONS FOR ACCESS LOCKS AND HATCHES

PART II

Where Structure Is Integral and Continuous

Design Category	Load Combination	Stress Comparisons	ASME Paragraph
Design I	D + PD + OBE + SRV + LOCA	Pm ≤1.0 Sm @ TD Pl ≤1.5 Sm @ TD Pl + Pb ≤1.5 Sm @ TD	NE-3131 (C)
Design II	D + PD + SSE + SRV + LOCA	Pm ≤ Sm @ TD Pl ≤1.5 Sm @ TD Pl + Pb ≤1.5 Sm @ TD	NE-3131 (C)
Design III	D + PE + OBE D + PE + SSE	Pm ≤1.0 Sm @ TE Pl ≤1.5 Sm @ TE Pl + Pb ≤1.5 Sm @ TE	NE-3131 (C)
Design IV	D + PF + OBE	Pm ≤1.5 Sm @ TF Pl ≤ larger of 1.8 Sm or 1.5 Sy @ TF Pl + Pb ≤ larger of 1.8 Sm or 1.5 Sy @ TF	
Operating I	D + Poc + To + OBE + SRV + LOCA	Pl + Pb + Q ≤3 Sm Pl + Pb + Q + F (fatigue evaluation)	NE-3222
Operating II	D + Poc + To + SSE + SRV + LOCA	Pl + Pb + Q ≤3 Sm	NE-3222
Test	D + PT	Pm ≤0.85 Sy @ TT Pl + Pb ≤1.25 Sy @ TT	NE-6322

TABLE 3.8-3
(Sheet 3 of 4)

PART III

Special Stress Limits
(NE-3227)

A. Bearing Stress 1. Crushing failure Design Category Design I Design II Design III Design IV Operating I Operating II Test	Load Combination D + PD + OBE D + PD + SSE D + PE + OBE D + PF + OBE D + Poc + To + OBE D + Poc + To + SSE D + PT	Stress Comparisons Average bearing stress ≤Sy @ T for the largest of these load combinations. When distance to free edge is greater than distance over which load is applied, the average bearing stress ≤1.5 Sy @ T.
2. Shear failure (bearing applied near free edges) As above	As above	Average shear stress ≤0.6 Sm @ T.
3. Pin bearing As above	As above	Average bearing stress ≤Sy @ T. If no credit is given to bearing area within one pin diameter from a plate edge, the average bearing stress ≤1.5 Sy @ T.
B. Pure Shear As above	As above	Average primary shear ≤0.6 Sm Maximum primary shear ≤0.8 Sm
C. Progressive Distortion Design Category Operating I Operating II	Load Combination D + Poc + To + OBE D + Poc + To + SSE	Stress Comparisons Pl + Pb + Q ≤Sy

TABLE 3.8-3
(Sheet 4 of 4)

LOAD COMBINATIONS FOR ACCESS LOCKS AND HATCHES

LOAD AND SYMBOL DEFINITIONS:

D	=	Deadload.
SSE	=	Safe shutdown earthquake.
OBE	=	Operating basis earthquake.
PD	=	Design internal pressure.
PE	=	Design external pressure.
PF	=	Post-DBA flooding internal pressure.
PT	=	Test pressure.
Poc	=	Operating pressure.
TD	=	Design temperature associated with PD.
TE	=	Design temperature associated with PE.
TF	=	Design temperature associated with PF.
To	=	Operating temperature associated with Poc.
TT	=	Test temperature.
SRV	=	Maximum values of dynamic loads from suppression pool due to safety/relief valve blowdown.
LOCA	=	Maximum values of dynamic loads from suppression pool due to postulated loss-of-coolant accident. Includes chugging, condensation oscillation, and pool swell.
Pm, Pl, Pb, Q, F	=	Calculated stresses as defined in ASME Section III, NB-3200.
Sm	=	Allowable stress intensity at temperature as defined in ASME Section III, NB-3200.
Sy	=	Yield strength of material as defined in ASME Section III.

NOTE: Jet impingement (J) loads are not postulated for the locks and hatches.

TABLE 3.8-4
(Sheet 1 of 4)

LOAD COMBINATIONS FOR THE DRYWELL HEAD

PART I

Where Structure Is Integral and Continuous

Design Category	Load Combination	Stress Comparisons	ASME Paragraph
Design I	D + PD + OBE + SRV + LOCA	Pm ≤1.0 Sm @ TD Pl ≤1.5 Sm @ TD Pl + Pb ≤1.5 Sm @ TD	NE-3131 (C)
	D + PD + OBE + SRV + LOCA + J D + PD[1] + OBE + SRV + LOCA + J	Comparisons to 85% of ASME III Appendix F allowables	NE-3131.2
Design II	D + PD + SSE + SRV + LOCA	Pm ≤larger of 1.2 Sm or Sy @ TD Pl ≤larger of 1.8 Sm or 1.5 Sy @ TD Pl + Pb ≤larger of 1.8 Sm or 1.5 Sy @ TD	NE-3131 (C)
	D + PD + SSE + SRV + LOCA + J D + PD[1] + SSE + SRV + LOCA + J	Comparisons to 85% of ASME III Appendix F allowables	NE-3131.2
Design III	D + PE + OBE	Pm ≤1.0 Sm @ TE Pl ≤1.5 Sm @ TE Pl + Pb ≤1.5 Sm @ TE	NE-3131 (C)
		ASME III instability criteria	NE-3133 & NE-3131 (C)
	D + PE + SSE	Pm ≤larger of 1.2 Sm or 1.0 Sy @ TE Pl ≤larger of 1.8 Sm or 1.5 Sy @ TE Pl + Pb ≤larger of 1.8 Sm or 1.5 Sy @ TE	NE-3131 (C)
		1.2 (ASME III instability criteria)	NE-3133 & NE-3131 (C)
Operating I	D + Po + To + OBE + SRV + LOCA	Pl + Pb + Q ≤3 Sm Pl + Pb + Q + F (fatigue evaluation)	NE-3222
Operating II	D + Po + To + SSE + SRV + LOCA	Pl + Pb + Q ≤3 Sm	NE-3222
Test	D + PT	Pm ≤0.85 Sy @ TT Pl + Pb ≤1.25 Sy @ TT	NE-6322

TABLE 3.8-4
(Sheet 2 of 4)

LOAD COMBINATIONS FOR THE DRYWELL HEAD

PART II

Where Structure Is Not Integral and Continuous (Closure Joint Region Only)

Design Category	Load Combination	Stress Comparisons	ASME Paragraph
Design I	D + PD + OBE + SRV + LOCA	Pm ≤ 1.0 Sm @ TD Pl ≤ 1.5 Sm @ TD Pl + Pb ≤ 1.5 Sm @ TD	NE-3131 (C)
	D + PD + OBE + SRV + LOCA + J	Pm ≤ larger of 1.2 Sm or Sy @ TD or TD[1]	NE-3131-2
	D + PD[1] + OBE + SRV + LOCA + J	Pl ≤ larger of 1.8 Sm or 1.5 Sy @ TD or TD[1] Pl + Pb ≤ larger of 1.8 Sm or 1.5 Sy @ TD or TD[1]	
Design II	D + PD + SSE + SRV + LOCA	Pm ≤ Sm @ TD Pl ≤ 1.5 Sm @ TD Pl + Pb ≤ 1.5 Sm @ TD	NE-3131 (C)
	D + PD + SSE + SRV + LOCA + J	Pm ≤ larger of 1.2 Sm or Sy @ TD or TD[1]	NE-3131-2
	D + PD[1] + SSE + SRV + LOCA + J	Pl ≤ larger of 1.8 Sm or 1.5 Sy @ TD or TD[1] Pl + Pb ≤ larger of 1.8 Sm or 1.5 Sy @ TD or TD[1]	
Design III	D + PE + OBE	Pm ≤ 1.0 Sm @ TE	NE-3131 (C)
	D + PE + SSE	Pl ≤ 1.5 Sm @ TE Pl + Pb ≤ 1.5 Sm @ TE	
		ASME III instability criteria	NE-3133 & NE-3131 (C)
Operating I	D + Po + To + OBE + SRV + LOCA	Pl + Pb + Q ≤ 3 Sm Pl + Pb + Q + F (fatigue evaluation)	NE-3222
Operating II	D + Po + To + SSE + SRV + LOCA	Pl + Pb + Q ≤ 3 Sm	NE-3222
Test	D + PT	Pm ≤ 0.85 Sy @ TT Pl + Pb ≤ 1.25 Sy @ TT	NE-6322

TABLE 3.8-4
(Sheet 3 of 4)

LOAD COMBINATIONS FOR THE DRYWELL HEAD

PART III
Special Stress Limits
(NE-3227)

A. Bearing Stress		
1. Crushing failure		
Design Category	Load Combinations	Stress Comparisons
Design I	D + PD + OBE + J + SRV + LOCA D + PD[1] + OBE + J + SRV + LOCA	Average bearing stress ≤Sy @ T for the largest of these load combinations. When distance to free edge is greater than distance over which load is applied, the average bearing stress ≤1.5 Sy @ T.
Design II	D + PD + SSE + J + SRV + LOCA D + PD[1] + SSE + J + SRV + LOCA	
Design III	D + PE + OBE D + PE + SSE	
Operating I	D + Po + To + OBE + SRV + LOCA	
Operating II	D + Po + To + SSE + SRV + LOCA	
Test	D + PT	
2. Shear failure (bearing applied near free edges)		
As above	As above	Average shear stress ≤0.6 Sm @ T.
3. Pin bearing		
As above	As above	Average bearing stress ≤Sy @ T. If no credit is given to bearing area within one pin diameter from a plate edge, the average bearing stress ≤1.5 Sy @ T.
B. Pure Shear		
As above	As above	Average primary shear ≤0.6 Sm Maximum primary shear ≤0.8 Sm
C. Progressive Distortion		
Design Category	Load Combinations	Stress Comparisons
Operating I	D + Po + To + OBE + SRV + LOCA	Pl + Pb + Q ≤Sy
Operating II	D + Po + To + SSE + SRV + LOCA	

TABLE 3.8-4
(Sheet 4 of 4)

LOAD COMBINATIONS FOR THE DRYWELL HEAD

LOAD AND SYMBOL DEFINITIONS:

D	=	Deadload
SSE	=	Safe shutdown earthquake
J	=	Local and general effects from jet impingement due to pipe break (PD and TD) and reactor core spray pipe break (PD^1 and TD^1)
LOCA	=	Dynamic loads in the suppression pool due to the postulated loss-of-coolant accident
OBE	=	Operating basis earthquake
SRV	=	Dynamic loads in suppression pool due to safety/relief valve blowdown
PD	=	Design internal pressure
PE	=	Design external pressure
Po	=	Operating internal pressure
PT	=	Acceptance test internal pressure
PD^1	=	Design internal pressure, core spray pipe break
TD	=	Design temperature associated with PD
TT	=	Design temperature associated with PT
TD^1	=	Design internal temperature associated with PD^1
To	=	Operating temperature associated with Po
Pm, Pl, Pb, Q, F	=	Calculated stresses as defined in ASME Section III Subarticle NB-3200
Sm	=	Allowable stress intensity at temperature as defined in ASME Section III
Sy	=	Yield strength of material as defined in ASME Section III

TABLE 3.8-5
(Sheet 1 of 2)

LOAD COMBINATIONS AND STRESS LIMITS FOR SAFETY CLASS 1 PIPE PENETRATIONS

Plant Condition	Load Combination	Stress Limits	ASME Paragraph
Design operating	a. PD + D + (FLT+OBEI+SRV) SRSS	PL + Pb $\leq$ 1.5 Sm @ TD	NB-3652, Eq 9
Normal and upset	b. Po + D + To + ΔT + (FLT+[OBEI+OBEA] ABS +SRV) SRSS	For stress range Sn = PL + Pb + Pe + Q $\leq$ 3.0 Sm	NB-3653, Eq 10 or Eq 12 and 13
		Sp = PL + Pb + Pe + Q + F For fatigue analysis U $\leq$ 1.0	NB-3653, Eq 11 NB-3653.5
Emergency	c. PE + D + (FLT+OBEI+SRV+LOCA) SRSS	PL + Pb $\leq$ 2.25 Sm	NB-3655
Faulted	d. PF + D + (FLT+SSEI+SRV+LOCA+RT) SRSS	PL + Pb $\leq$ 3.0 Sm	NB-3656
	e. PF + D + (FLT+RT+SSEI+AP) SRSS	PL + Pb $\leq$ 3.0 Sm	NB-3656
Test	f. Pt + D	PL + Pb $\leq$ 1.35 Sy Pm $\leq$ 0.9 Sy	NB-3226 NB-3226
	g. Pt + D + ΔT	PL + Pb + Pe + Q $\leq$ 3.0 Sm PL + Pb + Pe + Q + F $\leq$ Sa Fatigue	NB-3226 If more than 10 tests are performed
Pipe rupture exclusion case			
Normal and upset	Same as Item b.	Sn = PL + Pb + Pe + Q $\leq$ 2.4 Sm or 2.4 Sm < Sn $\leq$ 3.0 Sm $\overline{U}$ $\leq$ 0.1 or Sn $\geq$ 3 Sm Eq 12 $\leq$ 2.4 Sm Eq 13 $\leq$ 2.4 Sm U $\leq$ 0.1	NB-3653, Eq 10 BTP MEB 3-1 NB-3653, Eq 12 and 13
Faulted	Same as Items d. and e.	PL + Pb $\leq$ 2.25 Sm	NB-3652, Eq 9

TABLE 3.8-5
(Sheet 2 of 2)

LOAD COMBINATIONS AND STRESS LIMITS FOR SAFETY CLASS 1 PIPE PENETRATIONS

LOAD AND SYMBOL DEFINITIONS:

PD, PE, PF, Po, Pmax	=	Internal pressure for design, emergency, faulted and normal/upset operating conditions, respectively.
Pmax	=	Maximum of Po.
Pt	=	Pipe test pressure.
RT	=	Pipe rupture load and local and general effects from jet impingement, if applicable.
LOCA	=	Dynamic loads from the suppression pool due to the postulated loss-of-coolant accident.
D	=	Deadweight and other sustained loads.
FLT	=	Pipe load due to fluid transient, such as thrust from relief and safety valve loads from pressure and flow transients.
OBEI, OBEA	=	Loads due to OBE from structure or piping inertia and piping anchor movement effects, respectively.
SRV	=	Dynamic loads from the suppression pool due to safety/relief valve blowdown.
SSEI	=	Load due to safe shutdown earthquake (SSE) from structure or piping inertia effect.
ΔT	=	Thermal stresses due to thermal gradient at local region.
To	=	Thermal loads from containment and/or from piping due to a system expansion and anchor movements.
Sm	=	Allowable stress values at temperature of the consideration from ASME Section III Table I-1.0 for Safety Class 1 components.
Sa	=	Allowable stress intensity value from fatigue curves from Appendix I of ASME III.
Sy	=	Yield strength from Appendix I of ASME Section III.
Sn, Sp, Pm, PL, Pb, Pe, Q, F	=	Calculated stresses as defined in ASME Section III, NB-3200 and NB-3653.
U	=	Cumulative usage factor.

Superscripts:

ABS	=	Absolute sum.
SRSS	=	Square root of sum of squares.

NOTES:

1. Stress evaluation of this portion of the penetration will basically follow EQ 9-14 of NB-3652 and NB-3653, and the load combinations conform to Regulatory Guide 1.48.
2. If the stress evaluation of the penetration exceeds the above limits, Subarticle NB-3200 will be applied.

TABLE 3.8-6
(Sheet 1 of 2)

LOAD COMBINATIONS AND STRESS LIMITS FOR SAFETY CLASS 2 PIPE PENETRATIONS

Plant Condition	Load Combinations	Stress Limits	ASME Paragraph
Design operating	a. PD + D	$\frac{P_D D_o}{4t} + 0.75I \frac{M_A}{z} \leq 1.0S_h$	NC-3652.1 Eq 8
Normal and upset	b. Pmax + D + $(FLT+OBEI+SRV)^{SRSS}$	$\frac{P_{max} D_o}{4tn} + 0.75I \frac{M_A + M_B}{z} \leq 1.2S_h$	NC-3652.2 Eq 9
	c. (To+OBEA) + (PD+D)	$\frac{i M_c}{z} + \left(\frac{P_o D_o}{4tn} + 0.75i \frac{M_A}{Z} \right) \leq S_A + S_h$	NC-3652.3 Eq 11
Emergency	d. PE + D + $(FLT+OBEI+SRV+LOCA)^{SRSS}$	$\frac{P_E D_o}{tn} + 0.75i \frac{M_A + M_B}{Z} \leq 1.8S_h$	NC-3611.3(c)
Faulted	e. PF + D + $(FLT+SSEI+SRV+RT+LOCA)^{SRSS}$	$\frac{P_F D_o}{4tn} + 0.75i \frac{M_A + M_B}{Z} \leq 2.4S_h$ (See Note)	ASME Section III Code Case N-53 (1606-1)
	f. PF + D + $(FLT+SSEI+RT+AP)^{SRSS}$	$\frac{P_F D_o}{4tn} + 0.75i \frac{M_A + M_B}{Z} \leq 2.4S_h$ (See Note)	
Test	g. Pt + D	$\frac{P_t D_o}{4tn} + 0.75i \frac{M_A}{Z} \leq 1.35S_y$	NC-3218.1
Pipe break exclusion			
Normal and upset	P_{max} + D + $(FLT+OBEI+SRV)^{SRSS}$ + (To+OBEA)	$\frac{P_{max} D_o}{4t_n} + 0.75i \frac{M_A + M_B}{Z} + i \frac{M_c}{Z} \leq 0.8\ (1.2S_h + S_A)$	NRC Branch Technical Position MEB 3-1
	PF + D + $(FLT+SSEI+SRV+LOCA+RT)^{SRSS}$	$\frac{P_f D_o}{4t_n} + 0.75i \frac{M_A + M_B}{Z} \leq 1.8S_h$	
	PF + D +$(FLT+SSEI+RT+AP)^{SRSS}$		

TABLE 3.8-6
(Sheet 2 of 2)

LOAD COMBINATIONS AND STRESS LIMITS FOR SAFETY CLASS 2 PIPE PENETRATIONS

LOAD AND SYMBOL DEFINITIONS:

Mi (i=A,B,C)	=	Resultant moment, i.e., Mi = $(Mix^2+Miy^2+Miz^2)^{1/2}$, for each load combination except Mc which is due to (To+OBEA).
Z	=	Pipe section modulus per NC-3654.4.
i	=	Stress intensification factor, if 0.75 i <1, take 0.75 i = 1.
Do	=	Outside diameter of pipe.
tn	=	Nominal wall thickness of component.
PD,PE,PF, Po	=	Internal pressure for design, emergency, faulted, and normal/upset operating conditions, respectively.
Pmax	=	Maximum of Po.
Pt	=	Test pressure.
D	=	Deadweight and other sustained loads.
FLT	=	Pipe load due to fluid transient, such as thrust from relief and safety valve loads from pressure and flow transients.
RT	=	Pipe rupture loads and local and general effects from jet impingement if applicable.
LOCA	=	Dynamic loads from suppression pool due to the postulated loss-of-coolant accident.
OBEI,OBEA	=	Loads due to OBE from structure or piping inertia and piping anchor movement effects, respectively.
AP	=	Effect of vibration of reactor pressure vessel and biological shield wall due to annulus pressurization.
SRV	=	Dynamic loads from suppression pool due to safety/relief valve blowdown.
SSEI	=	Load due to safe shutdown earthquake (SSE) from structure or piping inertia effect.

Superscripts:

ABS	=	Absolute sum.
SRSS	=	Square root of sum of squares.

Sy	=	Yield strength from Appendix I of ASME Section III.
Sh	=	Allowable stresses from Table I-7.0 of ASME Section III at maximum (hot) temperature.
SA	=	Allowable stress range for expansion stresses (see ASME Section III, NC-3611.2(c)).

NOTE: The faulted stress limits and analysis techniques specified in ASME Section III, Appendix F, can be applied in lieu of Code Case N-53 (1606-1). Inelastic methods can be used as allowed by the Code.

TABLE 3.8-7
(Sheet 1 of 2)

LOAD COMBINATIONS AND STRESS LIMITS FOR PENETRATIONS - CLASS MC PORTIONS

Plant Condition	Load Combinations	Stress Limits	ASME Paragraph*
Design I	a. PD + D + L + SRV + OBEI	Pm ≤ 1.0 Smc @ TD PL ≤ 1.5 Sm @ TD PL + Pb ≤ 1.5 Sm @ TD	NE-3131(a), (b)
Design II	b. PD + D + L + SRV + SSEI	Pm ≤ greater of Sy or 1.2 Sm @ TD PL ≤ 1.5 (greater of Sy or 1.2 Sm) @ TD PL + Pb ≤ 1.5 (greater of Sy or 1.2 Sm) @ TD	NE-3131(c)(2)
Design III	c. PEc + D + L + OBEI	Pm ≤ smaller of Sm or 1.0 B @ TD PL ≤ smaller of 1.5 Sm or 1.5 B @ TD PL + Pb ≤ smaller of 1.5 Sm or 1.5 B @ TD	NE-3112.4 NE-3131(c)(2) NE-3133
	d. PEc + D + L + SSEI	Pm ≤ greater of 1.2 Sm or Sy or 1.2 B @ TD PL ≤ greater of 1.8 Sm or 1.5 Sy or 1.8 B @ TD PL + Pb < greater of 1.8 Sm or 1.5 Sy, or 1.8 B @ TD	NE-3131(c)(2) NE-3133
Design IV	e. PF + D + L + OBEI	Pm ≤ 1.0 SM @ TD PL ≤ 1.5 SM @ TD PL + Pb ≤ 1.5 Smc @ T	NE-3131(a)(b)(d)
Operation	f. Poc + D + L + (OBEI + OBEA) + SRV + ΔT + To	PL + Pb + Pe + Q ≤ 3 Sm PL + Pb + Pe + Q + F = Sa For fatigue analysis	NE-3227
	g. Poc max + D + L + SSEI + RT + Ap or (SRV + LOCA)	Pm ≤ 0.85 SF @ Toc PL + Pb ≤ 1.5 (0.85 SF)	NE-3131.2
Test	h. PNEU + D	Pm ≤ 0.90 Sy @ TT PL + Pb ≤ 1.25 Sy @ TT	NE-6322
	i. Pt + D + T	PL + Pb + Pe + Q ≤ 3 Sm PL + Pb + Pe + Q + F ≤ Sa Fatigue	If more than 10 tests are performed (NE-3226)
Special Limit	For Items a. through g.	≤ 4 Sm	NE-3227.4

* NE denotes Subsection NE of ASME Boiler and Pressure Vessel Code, Section III.

TABLE 3.8-7
(Sheet 2 of 2)

LOAD COMBINATIONS AND STRESS LIMITS FOR PENETRATIONS - CLASS MC PORTIONS

LOAD AND SYMBOL DEFINITIONS:

PD	=	Design internal pressure of containment and piping.
PEc	=	Containment external pressure.
Poc	=	Operating internal pressure of containment and piping (Poc max is maximum of Poc).
PF	=	Design pressure for flooded condition.
Pt	=	Pipe test pressure.
PNEU	=	Primary containment pneumatic test pressure.
AP	=	Effect of vibration of reaction pressure vessel and ecological shield wall due to annulus pressurization.
D	=	Deadweight and other sustained loads.
L	=	Live load, fluid transient loads, jet impingement loads, if applicable.
RT	=	Pipe rupture load.
LOCA	=	Dynamic loads from the suppression pool due to postulated loss-of-coolant accident.
SRV	=	Dynamic loads from the suppression pool due to safety/relief valve blowdown.
OBEI, OBEA	=	Loads due to OBE from structure to piping inertia and piping anchor movement effects, respectively.
SSEI	=	Load due to safe shutdown earthquake (SSE) from structure or piping inertia effect.
T	=	Thermal stresses due to thermal gradient at local region.
To	=	Thermal loads from containment and/or from piping due to piping system expansion and anchor movements.
Pm, Pl, Pb, Pe, Q, F	=	Calculated stresses as defined in ASME Section III, NE-3200.
SF	=	0.7 Su for ferritic steel; lesser of 2.4 Sm or 0.7 Su for austenitic steel.
Sm	=	Allowable stress values at temperature of the consideration from ASME Section III, Table I-10.0 for Class MC components.
Sa	=	Allowable stress valves from fatigue curves from Appendix I.
Su	=	Ultimate strength from Appendix I of ASME Section III.
Sy	=	Yield strength from Appendix I of ASME Section III.
TD	=	Design temperature associated with PD, PEc, or PF.
TT	=	Test temperature associated with PT and PNEU.
TOC	=	Operating temperature of containment.
B	=	Stress determined from Appendix VII of ASME Section III.
σ_N	=	N = 1, 2, 3 Principal stress.

Superscript:

ABS = Absolute sum.

TABLE 3.8-8
(Sheet 1 of 1)

STRESS

Stress Component	Location (el)	Governing Equation	Stress Actual	Stress Allowable
Concrete Stress				
Membrane and bending	175'	U = 1.0 D + 1.0 L + 1.0 Ta + 1.5 Pa + 1.0 Ra	2,800 psi	3,000 psi
Radial shear	180'	U = 1.0 D + 1.0 L + 1.0 Ta + 1.25 Pb + 1.0 Ra + 1.25 SRV	vu = 37 psi	vc = 37 psi
Reinforcement Stress				
Meridional				
Inside layer	175'	U = 1.0 D + 1.0 L + 1.0 Ta + 1.5 Pa + 1.0 Ra	44.9 ksi	45 ksi
Outside layer	201'	U = 1.0 D + 1.0 L + 1.0 Ta + 1.1 Pb + 1.0 Ra + 1.1 E + 1.1 SRV	44.5 ksi	45 ksi
Hoop				
Inside layer	220'	U = 1.0 D + 1.0 L + 1.0 Ta + 1.5 Pa + 1.0 Ra	43.8 ksi	45 ksi
Outside layer	220'	U = 1.0 D + 1.0 L + 1.0 Ta + 1.1 Pa + 1.0 Ra	43.7 ksi	45 ksi
Radial shear	260'	U = 1.0 D + 1.0 L + 1.0 Ta + 1.25 Pa + 1.0 Ra + 1.25 E + 1.0 (Rj + Rm + Rr)	35.7 ksi	42.5 ksi
Diagonal	201'	U = 1.0 D + 1.0 L + 1.0 Ta + 1.5 Pa + 1.0 Ra	45.0 ksi	45.0 ksi

KEY: vu = Nominal design shear stress resisted by concrete.
vc = Nominal permissible shear stress carried by concrete.

NOTE: Symbols are identified in Table 3.8-1.

TABLE 3.8-9
(Sheet 1 of 2)

LOAD COMBINATIONS FOR THE BIOLOGICAL SHIELD WALL

Normal Operating Load

1. S = 1.0 D + 1.0 E

2. S = (1.0)D + (1.0)To + (1.0)Ro + E

Abnormal and/or Extreme Environmental Load

3. 1.6 S = (1.0)D + (1.0)To + (1.0)Ro + E'

4. 1.6 S = (1.0)D + (1.0)Td + (1.0)Ra' + (1.0)Pd

5. 1.8 S = (1.0)D + (1.0)Td + (1.0)Ra' + (1.0)Pd + 1.0(Rr+Rj+Rm) + E

6. 2.0 S = (1.0)D + (1.0)Td + (1.0)Ra' + (1.0)Pd + 1.0(Rr+Rj+Rm) + E'

KEY:

D	=	Dead load of shield wall, equipment, and attached piping
E	=	Load due to operating basis earthquake (OBE)
E'	=	Load due to safe shutdown earthquake (SSE)
Pd	=	Pressure differential across biological shield wall due to a pipe rupture
Ra'	=	Pipe reactions under thermal conditions generated by the postulated accident
Rj	=	Equivalent static jet impingement load acting on a structure generated by a ruptured high-energy pipe during the postulated accident. The peak value of Rj is used unless a time-history analysis is performed to justify otherwise.
Rm	=	Equivalent static missile impact load on a structure generated by or during the postulated accident. The peak value of Rm is used unless a time-history analysis is performed to justify otherwise.
Ro	=	Pipe reaction during operating condition.
Rr	=	Equivalent static reaction load from the ruptured high-energy pipe during the postulated accident. The peak value of Rr is used unless a time-history analysis is performed to justify otherwise.
S	=	Required section strength is based on elastic design methods which are used in design as the allowable stresses defined in the AISC specification except that the 33 1/3 percent

TABLE 3.8-9
(Sheet 2 of 2)

LOAD COMBINATIONS FOR THE BIOLOGICAL SHIELD WALL

		increase in allowable stresses for seismic loading is not used.
Td	=	Effect of temperature gradient across biological shield wall, including increase of temperature due to pipe rupture and pressure buildup.
To	=	Effect of temperature gradient across biological shield wall during operating condition.

TABLE 3.8-10
(Sheet 1 of 3)

DESIGN STRENGTH FOR LOAD COMBINATIONS ON CATEGORY I STEEL STRUCTURES

PART I

Load Combinations Without SRV Discharge Loading[1,2,3]

Operating Conditions

Normal Loading:

1. S = 1.0[D+L]

Severe Environmental Loading:

2. S = 1.0[D+L+E]
3. S = 1.0[D+L+W]

Design Conditions

Extreme Environmental Loading:

4. 1.6 S = 1.0[D+L+To+Ro+E']
5. 1.6 S = 1.0[D+L+To+Ro+Wt]

Abnormal Loading:

6. 1.6 S = 1.0[D+L+Ta+Ra+Pa]
7. 1.8 S = 1.0[D+L+Ta+Ra+Rr+Rj+Rm+Pa+E]
8. 2.0 S = 1.0[D+L+Ta+Ra+Rr+Rj+Rm+Pa+E']
9. 1.6 S = 1.0[D+Ls]

PART II

Load Combinations With SRV Discharge Loading[1,2,3]

To account for the effect of SRV discharge loading and suppression pool hydrodynamic loading, the steel structures and steel framing within the reactor building are checked for the following combinations:

TABLE 3.8-10
(Sheet 2 of 3)

DESIGN STRENGTH FOR LOAD COMBINATIONS ON CATEGORY I STEEL STRUCTURES

Design Conditions

Normal Without Temperature:

1. $S = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ SRV_{seq}$

2. $S = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ SRV_{asy}$

Normal With Temperature:

3. $S = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.0\ SRV_{seq}$

4. $S = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.0\ SRV_{asy}$

Normal/Severe Environmental:

5. $S = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ Ro + 1.0\ To + 1.0\ E + 1.0\ SRV_{seq}$

6. $S = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ Ro + 1.0\ To + 1.0\ E + 1.0\ SRV_{asy}$

Abnormal:

7. $1.6\ S = 1.0\ D + 1.0\ L + 1.0\ Pb + 1.0\ Ta + 1.0\ Ra + 1.0\ SRV_{ads}$

8. $1.6\ S = 1.0\ D + 1.0\ L + 1.0\ Pb + 1.0\ Ta + 1.0\ Ra + 1.0\ SRV_{asy}$

9. $1.6\ S = 1.0\ D + 1.0\ L + 1.0\ Pa + 1.0\ Ta + 1.0\ Ra + 1.0\ SRV_{sngl}$

Abnormal/Severe Environmental:

10. $1.8\ S = 1.0\ D + 1.0\ L + 1.0\ E + 1.0\ Pb + 1.0\ Ta + 1.0\ Ra + 1.0\ SRV_{ads}$

11. $1.8\ S = 1.0\ D + 1.0\ L + 1.0\ E + 1.0\ Pb + 1.0\ Ta + 1.0\ Ra + 1.0\ SRV_{asy}$

12. $1.8\ S = 1.0\ D + 1.0\ L + 1.0\ E + 1.0\ Pa + 1.0\ Ta + 1.0\ Ra + 1.0\ SRV_{sngl}$

TABLE 3.8-10
(Sheet 3 of 3)

DESIGN STRENGTH FOR LOAD COMBINATIONS ON CATEGORY I STEEL STRUCTURES

Normal/Extreme Environmental:

13. $1.6\ S = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.0\ E' + 1.0\ SRV_{seq}$

14. $1.6\ S = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.0\ E' + 1.0\ SRV_{asy}$

Abnormal/Extreme Environmental:

15. $2.0\ S = 1.0\ D + 1.0\ L + 1.0\ E' + 1.0\ Pb + 1.0\ Ta + 1.0\ Ra + 1.0\ Rj + 1.0\ Rm + 1.0\ Rr + 1.0\ SRV_{ads}$

16. $2.0\ S = 1.0\ D + 1.0\ L + 1.0\ E' + 1.0\ Pb + 1.0\ Ta + Ra + 1.0\ Rj + 1.0\ Rm + 1.0\ Rr + 1.0\ SRV_{asy}$

17. $2.0\ S = 1.0\ D + 1.0\ L + 1.0\ E' + 1.0\ Pa + 1.0\ Ta + 1.0\ Ra + 1.0\ Rj + 1.0\ Rm + 1.0\ Rr + 1.0\ SRV_{sngl}$

(1) If thermal stresses due to To and Ro are present and are self-limiting in nature, a 50-percent increase in allowable stresses will be permitted.

(2) The peak values of Pa, Pb, Ta, Ra, Rr, Rj, and Rm are used unless a time-history analysis is performed to justify otherwise.

(3) Local stresses due to the concentrated load Rj, Rm, or Rr may exceed the allowables, but there will be no loss of function.

NOTES:

1. Key to notations follows Table 3.8-12.

2. Loads resulting from thermal stratification, if applicable, are included wherever temperature loads are considered.

TABLE 3.8-11
(Sheet 1 of 3)

REQUIRED STRENGTH FOR LOAD COMBINATIONS ON CATEGORY I CONCRETE STRUCTURES

PART I

Load Combinations Without SRV Discharge Loading

Operating Conditions

Normal Loading:

1. U = 1.4 D + 1.7 L + 1.3[To+Ro]

Severe Environmental Loading:

2. U = 1.4 D + 1.7[L+W] + 1.3[To+Ro]

3. U = 1.4 D + 1.7 L + 1.9 E + 1.3[To+Ro]

Design Conditions

Extreme Environmental Loading:

4. U = 1.0[D+L+E'+To+Ro]

5. U = 1.0[D+L+Wt+To+Ro]

6. U = 1.0[D+L+F+To+Ro]

Abnormal Loading:

7. U = 1.0[D+L+Ta+Ra] + 1.5 Pa(1)

Abnormal/Severe Environmental Loading:

8. U = 1.0[D+L+Ta+Ra+Rj+Rr+Rm] + 1.25[Pa+E](1,2)

Abnormal/Extreme Environmental Loading:

9. U = 1.0[D+L+Ta+Ra+Rj+Rr+Rm+Pa+E'](1-3)

10. U = 1.0[D+Ls]

TABLE 3.8-11
(Sheet 2 of 3)

REQUIRED STRENGTH FOR LOAD COMBINATIONS ON CATEGORY I CONCRETE STRUCTURES

PART II

Load Combinations With SRV Discharge Loading

To account for the effect of SRV discharge loading and suppression pool hydrodynamic loading, the concrete structures within the reactor building are checked for the following combinations:

Design Conditions

Normal Without Temperature:

1. $U = 1.4\ D + 1.7\ L + 1.0\ Po + 1.5\ SRV_{seq}$

2. $U = 1.4\ D + 1.7\ L + 1.0\ Po + 1.5\ SRV_{asy}$

Normal With Temperature:

3. $U = 1.0\ D + 1.3\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.3\ SRV_{seq}$

4. $U = 1.0\ D + 1.3\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.3\ SRV_{asy}$

Normal Severe Environmental:

5. $U = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.25\ E + 1.25\ SRV_{seq}$

6. $U = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.25\ E + 1.25\ SRV_{asy}$

Abnormal:

7. $U = 1.0\ D + 1.0\ L + 1.25\ Pb + 1.0\ Ta + 1.0\ Ra + 1.25\ SRV_{ads}$

8. $U = 1.0\ D + 1.0\ L + 1.25\ Pb + 1.0\ Ta + 1.0\ Ra + 1.25\ SRV_{asy}$

9. $U = 1.0\ D + 1.0\ L + 1.25\ Pa + 1.0\ Ta + 1.0\ Ra + 1.0\ SRV_{sngl}$

Abnormal/Severe Environmental:

10. $U = 1.0\ D + 1.0\ L + 1.1\ E + 1.1\ Pb + 1.0\ Ta + 1.0\ Ra + 1.1\ SRV_{ads}$

TABLE 3.8-11
(Sheet 3 of 3)

REQUIRED STRENGTH FOR LOAD COMBINATIONS ON CATEGORY I CONCRETE STRUCTURES

11. $U = 1.0\ D + 1.0\ L + 1.1\ E + 1.1\ Pb + 1.0\ Ta + 1.0\ Ra + 1.1\ SRV_{asy}$

12. $U = 1.0\ D + 1.0\ L + 1.1\ E + 1.1\ Pa + 1.0\ Ta + 1.0\ Ra + 1.0\ SRV_{sngl}$

Normal/Extreme Environmental:

13. $U = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.0\ E' + 1.0\ SRV_{seq}$

14. $U = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.0\ E' + 1.0\ SRV_{asy}$

15. $U = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.0\ Wt + 1.0\ SRV_{seq}$

16. $U = 1.0\ D + 1.0\ L + 1.0\ Po + 1.0\ To + 1.0\ Ro + 1.0\ Wt + 1.0\ SRV_{asy}$

Abnormal/Extreme Environmental:

17. $U = 1.0\ D + 1.0\ L + 1.0\ E' + 1.0\ Pb + 1.0\ Ta + 1.0\ Ra + 1.0\ Rj + 1.0\ Rm + 1.0\ Rr + 1.0\ SRV_{ads}$

18. $U = 1.0\ D + 1.0\ L + 1.0\ E' + 1.0\ Pb + 1.0\ Ta + 1.0\ Ra + 1.0\ Rj + 1.0\ Rm + 1.0\ Rr + 1.0\ SRV_{asy}$

19. $U = 1.0\ D + 1.0\ L + 1.0\ E' + 1.0\ Pa + 1.0\ Ta + 1.0\ Ra + 1.0\ Rj + 1.0\ Rm + 1.0\ Rr + 1.0\ SRV_{sngl}$

(1) The peak values of Pa, Ta, Ra, Rr, Rj, and Rm will be used unless a time-history analysis is performed to justify otherwise.

(2) Local stresses due to the concentrated load Rm may exceed the allowables, but there will be no loss of function.

(3) Use capacity reduction factor = 1.0.

NOTE: Key to notations follows Table 3.8-12.

TABLE 3.8-12
(Sheet 1 of 4)

LOAD COMBINATIONS FOR INTAKE TUNNELS*

Operating Conditions

Normal Loading:

1. U = 1.4 D + 1.7 L + 1.3 To + 1.4 Pd

Severe Environmental Loading:

2. U = 1.4 D + 1.7 L + 1.9 E + 1.3 To + 1.4 Pd

Design Conditions

Extreme Environmental Loading:

3. U = 1.0[D+L+E'+To] + 1.4 Pd

Abnormal Loading:

4. U = 1.0[D+L] + 1.4 Pd + 1.7 QL + 1.25 E

* These load combinations are used in designing the Category I reinforced concrete portions (e.g., concrete encasement) of intake tunnels.

In addition to these conditions, the concrete encasement is designed for the test (or preoperational) conditions considering the construction truck loading, the internal test pressure (Pi), or the external test pressure (Pe), as applicable.

KEY TO NOTATIONS FOR TABLES 3.8-10 THROUGH 3.8-12:

D = Dead loads and their related moments and forces, including any permanent loads and hydrostatic loads.
E = Loads generated by the operating basis earthquake.
E' = Loads generated by the safe shutdown earthquake.
F = All forces and moments related to hydrostatic and saturated soil pressures due to the postulated maximum flood (PMF). For this loading, exterior walls and foundations are designed for a hydrostatic head to el 261 ft.
L = Live loads and their related moments and forces, including:
1. Movable equipment loads.
2. Lateral soil pressures.
3. Snow load during normal operating or severe environment conditions.

TABLE 3.8-12
(Sheet 2 of 4)

LOAD COMBINATIONS FOR INTAKE TUNNELS

4. Pressure differences due to variation in heating and cooling and outside atmospheric changes.
5. Any other loads that vary with intensity and occurrence.

Pa = Pressure equivalent static load within or across a compartment and/or building, generated by the postulated accident (i.e., design basis pipe break accident with LOCA), and including an appropriate dynamic load factor applied to the peak of the pressure-time curve.

Pb = Pressure equivalent static load within or across a compartment and/or building, generated by the small pipe break accident (SBA) or the intermediate pipe break accident (IBA) events.

Pd = Pressure differential between the intake and discharge water (i.e., between inside and outside of the encasement) during the most critical of the operating or design conditions.

Po = Pressure loads on a structure during operating condition.

QL = Load caused by a postulated rock fall condition.

Ra = Pipe reactions under thermal conditions generated by the postulated accident.

Rj = Equivalent static jet impingement load acting on a structure generated by a ruptured high energy pipe during the postulated accident. Load includes an appropriate dynamic load factor applied to the peak of the jet pressure load time-history.

Rm = Equivalent static missile impact load acting on a structure generated by or during the postulated accident. Load includes an appropriate dynamic load factor applied to the peak of the missile impact-time curve.

Ro = Pipe reactions during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

Rr = Reaction equivalent static load on the ruptured high-energy pipe during the postulated accident, and including an appropriate dynamic load factor applied to the peak of the reaction-time curve.

TABLE 3.8-12
(Sheet 3 of 4)

LOAD COMBINATIONS FOR INTAKE TUNNELS

S = Allowable design strength when using the allowable stresses defined in Part I of the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, 1969 and 1978. The 33 1/3 percent increase in allowable stresses for seismic or wind loadings is not permitted.

In computing the allowable design strength, actual yield stress of the material may be used in place of specified minimum yield stress of the material. The actual yield stress of the material shall be determined from the test reports of the material under consideration. This provision may be used to establish design adequacy in isolated cases where the specified minimum strength of the material is potentially exceeded.

SRV = SRV loads are defined as follows:

SRV_{seq} = SRV loads due to sequential (i.e., all valves) actuation.
SRV_{ads} = SRV loads due to automatic depressurization system (ADS) (seven valves) actuation.
SRV_{asy} = SRV loads due to asymmetric (three valves) actuation.
SRV_{sngl} = SRV loads due to single valve actuation.

Ta = Thermal effects and loads generated by the postulated accident.
To = Thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.
U = Required strength to resist the design loads based on the strength design methods, as modified by the application of capacity reduction factors as described in Section 9.3, ACI-318-77.

In computing the required section strength, actual compressive strength of concrete may be used in place of specified minimum compressive strength of concrete. The actual compressive strength of

TABLE 3.8-12
(Sheet 4 of 4)

LOAD COMBINATIONS FOR INTAKE TUNNELS

concrete shall be determined from the compressive strength test reports of the concrete pour under consideration. This provision may be used to establish design adequacy in isolated cases where the specified minimum strength of the material is potentially exceeded.

W = Loads generated by the design wind specified for the unit (Section 3.3.1).

Wt = Loads generated by the design tornado specified for the unit. They include loads due to the tornado wind pressure, the tornado-created differential pressure, and the tornado-generated missiles (Sections 3.3.2 and 3.5).

Ls = Snow load or probable maximum precipitation (PMP) during abnormal/extreme environmental condition.

TABLE 3.8-13
(Sheet 1 of 1)

MAJOR STRUCTURES FOUNDATION SYSTEMS

Structure	Foundation Method	Approx. Contact Area (ft x ft)	Approx. Founding Elevation (ft, msl)	Depth of Burial of Structure Below Finished Grade (ft)
Reactor building including auxiliary bays	Reinforced concrete mat	182 ft diameter[1]	164	97
Radwaste building	Reinforced concrete mat	150 x 110	232	29
Control building	Reinforced concrete mat	96 x 134	210	51
Diesel generator building	Reinforced concrete mat and spread footings	-	242	19
Turbine building[2]	Reinforced concrete mat and spread footings	-	236 except at C.W. enclosure	25
Screenwell building[2]	Reinforced concrete mat and spread footings	-	222 except at C.W. enclosure	39
Intake structures	Tremie concrete slab with tie-downs in rock	Hexagonal with face-to-face dimension of 33'-6"	222	Structures in lake
Intake tunnels	Reinforced concrete mat	14' x 1,340'±	Varies from 132 to 146	Structures in lake
Main stack	Reinforced concrete mat	60 ft diameter	232'-5"	28.5
Offgas, regeneration, and condensate demineralizer area[2]	Reinforced concrete mat, spread footings, and grade beam arrangement	-	236	25
Standby gas treatment building and railroad access lock[2]	Reinforced concrete mat and spread footings and partially supported on tunnel	-	247 except 233 at top of tunnel	14 or 28
Auxiliary service building[2]	Slab supported on the top of tunnel	-	233 (top of tunnel)	28

(1) Actual contact area at auxiliary bays exceeds 182 ft diameter.
(2) Certain portions of these structures (including foundations) are designated Category I, whereas the remaining portions are designated non-Category I (Section 3.8.4).

TABLE 3.8-14
(Sheet 1 of 1)

FACTORS OF SAFETY FOR OVERTURNING, SLIDING AND FLOATATION OF MAJOR CATEGORY I STRUCTURES

	Overturning				Sliding				Floatation
	Loading Conditions				Loading Conditions				Loading Condition
Structure	1	2	3	4	1	2	3	4	5
Control and Diesel Generator Buildings	2.9	2.4	1.7	2.3	2.1	1.5	1.1	2.3	1.1
North and South Electrical Tunnels	1.5	(1)	(1)	(1)	2.1	(1)	(1)	(1)	5.1
Main Stack	(1)	4.2	(1)	(1)	(1)	19.8	(1)	(1)	11.6
Reactor Building	(1)	(1)	1.1	(1)	(1)	(1)	1.43	(1)	1.7
Screenwell Building	1.8	(1)	(1)	(1)	1.5	(1)	(1)	(1)	1.3
Standby Gas Treatment Building	3.6	(1)	1.2	(1)	1.5	(1)	1.1	(1)	1.2

LEGEND:

1 D+H+E
2 D+H+W
3 D+H+E'
4 D+H+W+
5 D+F'

(1) Indicates that the loading condition yields a factor of safety higher than those listed herein.

TABLE 3.8-15
(Sheet 1 of 2)

PREDICTED STRUCTURAL RESPONSE - CONTAINMENT PRESSURE TEST

Elevation (ft)	Displacements(1) (in)		Strains (Micro-in/in)				
			Meridional Bar		Circumferential Bar		
	Radial	Vertical	Inner	Outer	Inner	Outer	Shear Bar
A. Containment Response							
322.5	0.128	0.282	-	-	-	-	-
302	0.115	-	-	-	-	-	-
273	0.235	-	-	-	-	-	-
240.6	0.123	-	373	43	212	212	417
236	-	-	-	-	-	-	390
208	0.324	-	-	-	-	-	-
190.75	0.167	-	-	-	-	-	-
175	-	-	456	-114	36	36	261

Displacements(1) (in)		
Point	R	T
B. Equipment Hatch Response (Az 135°)		
37	0.209	0.0
38	0.226	0.0
39	0.235	0.0
40	0.241	-0.003
41	0.241	-0.009
42	0.241	-0.010
43	0.169	0.0
44	0.203	0.0
45	0.227	0.0
46	0.241	-0.003
47	0.241	-0.009
48	0.241	-0.010

TABLE 3.8-15
(Sheet 2 of 2)

PREDICTED STRUCTURAL RESPONSE - CONTAINMENT PRESSURE TEST

Strains	
Gauge	Strain (Micro-in/in)
139	251
140	251
141	247

[(1)] Radial displacement is considered positive outward along the horizontal. Vertical displacement is considered positive upward.

T, the tangential displacement (hoop direction), is measured positive from the center of the opening in both directions.

R, the radial displacement, is measured positive outward along the horizontal.