5.1 GENERAL DESIGN CRITERIA

5.1.1 INTRODUCTION

Appendix I to the original FSAR contained a comparison to 70 General Design Criteria (GDC) for Nuclear Power Plant Construction Permits issued by AEC Press Release K-172 on July 10, 1967. These criteria had been released after initiation of plant design. In the Request For Full Term Operating License and Application for an Increase in Power Level, submitted to the NRC on January 22, 1974, Consumers provided an update to compare the Palisades design with the GDC as they appeared in 10CFR50 Appendix A on July 7, 1971. It was this updated discussion including the identified exceptions which formed the original plant Licensing Basis for future compliance with the GDC. Although final NRC action to convert the license to a FTOL did not occur until 1991, this docketed accounting of GDC compliance was and, for the most part, continues to be the plant Licensing Basis for GDC.

Since the plant was designed, the NRC has issued the Standard Review Plan, Regulatory Guides and other documents which specify designs and methods acceptable to the NRC to show compliance with the GDC. These later interpretations and later revisions to the GDC themselves have not automatically become part of the Palisades Licensing Bases. The following discussions, therefore, specifically refer to the GDC and their interpretations which existed on July 7, 1971. These discussions should not be construed as commitments to comply with any interpretation document issued after this date. Any commitments to later design requirements are documented separately in the FSAR.

The Systematic Evaluation Program (SEP) also reexamined topics addressed by selected GDC. Differences between design and then-current (circa 1979-1982) regulatory criteria were evaluated for safety impact. Procedure and design changes which were concluded to be cost beneficial enhancements to safety were imposed. The result of this effort was documented in NUREG 0820 and its Supplement I. In general Palisades was shown to meet the intent of then-current NRC criteria. Some differences were identified between the existing design and the guidance (including GDC interpretations) in force at the time but, in general, few changes were shown to provide cost effective improvements in safety.

Provided below are the texts of the General Design Criteria as they appeared in 10 CFR 50, Appendix A on July 7, 1971 and the current Consumers responses. Some changes to the originally-submitted responses have been made to reflect later commitments which have been systematically backfit to the plant, to update the locations for referenced information to be consistent with the updated FSAR, and to incorporate some editorial changes for clarity, but the response texts are otherwise essentially as submitted in 1974. Also provided is a reference to where Plant design bases information related to the criteria are found in the FSAR Update, Technical Specifications, or SEP Topic Number.

5.1.2 GROUP I: OVERALL REQUIREMENTS (CRITERIA 1-5)

5.1.2.1 Criterion 1 - Quality Standards and Records

CRITERION - Structures, systems and components important to safety shall be designed, fabricated, erected and tested to quality standards commensurate with the importance of the safety functions to be performed. Where generally recognized codes and standards are used, they shall be identified and evaluated to determine their applicability, adequacy, and sufficiency and shall be supplemented or modified as necessary to assure a quality product in keeping with the required safety function. A Quality Assurance program shall be established and implemented in order to provide adequate assurance that these structures, systems and components will satisfactorily perform their safety functions. Appropriate records of the design, fabrication, erection, and testing of structures, systems and components important to safety shall be maintained by or under the control of the nuclear power unit licensee throughout the life of the unit.

RESPONSE - The intent of Criterion 1 has been satisfied in that those structures, systems and components which are essential to the prevention, or the mitigation of the consequences, of nuclear accidents which could cause undue risk to the health and safety of the public, were identified and designed as Class 1 (noted in the FSAR Update as Consumers Design Class) structures, systems and components as described in Section 5.2. The applicable codes and standards and additional measures taken beyond these codes and standards are discussed in the appropriate FSAR sections for these systems. Quality Assurance programs, test procedures and inspection acceptance criteria are given in the appropriate FSAR sections. Where no applicable codes or standards exist, a discussion of the design is given in the appropriate section.

References: Chapters 3, 4, 5, 6, 7, 8, 9, 10 and 11, and SEP Topics III-1 and III-7B

5.1.2.2 Criterion 2 - Design Bases for Protection Against Natural Phenomena

CRITERION - Structures, systems and components important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami and seiches without loss of capability to perform their safety functions. The design bases for these structures, systems and components shall reflect: (1) appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity and period of time in which the historical data have been accumulated, (2) appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena, and (3) the importance of the safety functions to be performed.

RESPONSE - This criterion has been met by designing, fabricating and erecting those structures, systems and components important to safety to withstand the effects of extraordinary natural phenomena. A discussion of the magnitude of these forces and the design bases derived there from is contained in Chapters 2 and 5. In addition, design bases for the various structures, systems and components for natural phenomena are listed in individual chapters.

References: Chapters 2, 3, 4, 5, 6, 7, 8, 9, 11, and SEP Topics II-2A, II-3B, II-3C, II-4D, II-4F, III-2, III-3A, III-4A and III-6

5.1.2.3 Criterion 3 - Fire Protection

CRITERION - Structures, systems and components important to safety shall be designed and located to minimize, consistent with other safety requirements, the probability and effect of fires and explosions. Noncombustible and heat resistant materials shall be used wherever practical throughout the units, particularly in locations such as the containment and control room. Fire detection and fighting systems of appropriate capacity and capability shall be provided and designed to minimize the adverse effects of fires on structures, systems and components important to safety. Fire fighting systems shall be designed to assure that their rupture or inadvertent operation does not significantly impair the safety capability of these structures, systems and components.

RESPONSE - This criterion is met by designing the plant so that buildings containing critical portions of the plant such as the containment building, control room and auxiliary building are constructed of noncombustible, flame retardant and heat resistant materials. Plant areas critical for achieving safe and stable conditions have been divided into fire areas such that a fire in any given area will not propagate to other areas and will not impair the Plant's ability to achieve safe and stable conditions.

Through a series of modifications including installation of fire stops, cable separation, addition of sprinklers, addition of designated fire brigade, procedure changes and others, the Palisades Plant has established conformance to the requirements of 10 CFR 50.48 and National Fire Protection Association (NFPA) 805.

Noncombustible and fire resistant or retardant materials are used throughout the balance of the Plant with minimal exceptions such as conventional applications of turbine generator lube and seal oil, generator hydrogen and outdoor transformer oil. Electric cable insulation has been considered as combustible material and appropriate measures have been taken to provide safe reactor shutdown in the event fires involve electrical raceways.

Equipment and facilities for fire protection, including detection, alarm and extinguishment, are provided in selected areas throughout the Plant. Deluge and sprinkler systems are provided in areas containing potentially combustible materials, and where required to meet cable separation criteria. Hose lines and portable extinguishers are located throughout the Plant.

Fire-fighting systems are designed with the shutoff valves for isolation in case of rupture or inadvertent operation. The rooms are supplied with drain systems to prevent flooding, and the cabinet top openings are sealed to prevent water ingress during sprinkler system operation.

The Fire Protection System is designed in accordance with the requirements of the National Fire Protection Association.

References: Chapters 7, 8, 9, and SEP Topic IX-6

5.1.2.4 Criterion 4 - Environmental and Missile Design Bases

CRITERION - Structures, systems and components important to safety shall be designed to accommodate the effects of and to be compatible with the environmental conditions associated with normal operation, maintenance, testing and postulated accidents including Loss of Coolant Accidents. These structures, systems and components shall be appropriately protected against dynamic effects, including the effect of missiles, pipe whipping and discharging fluids, that may result from equipment failures and from events and conditions outside the nuclear power unit.

RESPONSE - Structures, systems and components important to safety have been designed to be compatible with the environmental conditions associated with normal operation, maintenance and testing. If they must remain functional to mitigate the consequences of accidents, they are designed to be compatible with the environmental conditions or are protected against the effects of postulated accidents including Loss of Coolant Accidents. Components, piping and structures are protected against the dynamic effects of pipe whip through modifications pursuant to Consumers Special Report 6, Revision 3A, dated July 2007, "Analysis of Postulated High Energy Line Breaks Outside of Containment." Subsequent Plant modifications resulting from IEB 79-02, "Pipe Support Base Plate Design Using Concrete Expansion Bolts," and IEB 79-14, "Seismic Analysis for As-Built Safety-Related Piping Systems" have further enhanced plant capabilities in this area.

References: Chapters 1, 5, 6, 7, 8, 9, and SEP Topics III-4A, III-4B, III-4C, III-4D, III-5A, III-5B, III-8C, III-10B and VI-2

5.1.2.5 Criterion 5 - Sharing of Structures, Systems and Components

CRITERION - Structures, systems and components important to safety shall not be shared among nuclear power units unless it can be shown that such sharing will not significantly impair their ability to perform their safety functions, including, in the event of an accident in one unit, an orderly shutdown and cooldown of the remaining units.

RESPONSE - Since Palisades is a single-unit Plant, there is no sharing of facilities.

5.1.2.6 Conclusions

Based on the various parts of Subsection 5.1.2, it is concluded that the Palisades Plant is designed in conformance with the intent of the Group I criteria.

5.1.3 GROUP II: PROTECTION BY MULTIPLE FISSION PRODUCT BARRIERS (CRITERIA 10-19)

5.1.3.1 Criterion 10 - Reactor Design

CRITERION - The reactor core and associated coolant, control and protection systems shall be designed with appropriate margin to assure that specified acceptable fuel design limits are not exceeded during any condition of normal operation, including the effects of anticipated operational occurrences.

RESPONSE - Appropriate design bases were applied in the design of the fuel for all expected conditions of normal operation and for transient situations which can be anticipated. Refer to Section 3.2.3 for specific design criteria used for Palisades fuel.

Thus, the reactor core will function throughout its design lifetime without exceeding acceptable fuel damage limits. Design margins allow for deviations of temperature, pressure, flow, reactor power and reactor-turbine power mismatch. Manual control of the reactor by licensed operators will maintain the reactor operating parameters within preset limits and the Reactor Protective System will shut down the reactor if the specified operating limits are exceeded by preset amounts.

References: Chapters 3, 4, 7, 14, SEP Topics XV-8, XV-22 and Technical Specifications Section 2.1

5.1.3.2 Criterion 11 - Reactor Inherent Protection

CRITERION - The reactor core and associated coolant systems shall be designed so that in the power operating range the net effect of the prompt inherent nuclear feedback characteristics tends to compensate for a rapid increase in reactivity.

RESPONSE - The reactor core has a negative Doppler (fuel temperature) coefficient and a moderator temperature coefficient that ranges from near zero or slightly positive at the beginning of a cycle to quite negative near the end of a cycle. The overall power coefficient is always negative, so that any rapid increase in core reactivity will be promptly compensated for through Doppler and moderator temperature feedback.

Reference: Chapter 3

5.1.3.3 Criterion 12 - Suppression of Reactor Power Oscillations

CRITERION - The reactor core and associated coolant, control and protection systems shall be designed to assure that power oscillations which can result in conditions exceeding specified acceptable fuel design limits are not possible or can be reliably and readily detected and suppressed.

RESPONSE - Power oscillations from spatial xenon effects may occur in the reactor. These oscillations, however, are characterized by long periods and slow changes in power distribution. The transients are easily detectable on both incore and excore instrumentation and may be controlled, if necessary, with full-length regulating control rods.

Reference: Chapter 3, 7

5.1.3.4 Criterion 13 - Instrumentation and Control

CRITERION - Instrumentation and control shall be provided to monitor variables and systems over their anticipated range for normal operation and accident conditions, and to maintain them within prescribed operating ranges, including those variables and systems which can affect the fission process, the integrity of the reactor core, the reactor coolant pressure boundary, and the containment and its associated systems.

RESPONSE - Instrumentation is provided to monitor and maintain significant process variables. Controls are provided for the purpose of maintaining these variables within the limits prescribed for safe operation.

The principal process variables monitored include neutron flux (reactor power), primary coolant temperature, flow, pressure, pressurizer liquid level and steam generator level. In addition, instrumentation is provided for continuous automatic monitoring of radiation level.

The following are provided to monitor and maintain control over the fission process during both transient and steady-state periods over the lifetime of the core:

1. Nine channels of nuclear instrumentation which constitute the primary monitor of the fission process. Of these channels, two source range and two wide range channels are used to monitor the reactor during start-up, four channels monitor the reactor in the power range and are used to initiate a reactor shutdown in the event of overpower, and one channel is provided in the Alternate Shutdown Panel See Chapter 7 for more detailed description.

2. Two independent control rod position indicating systems.

3. Manual control of reactor power by means of control rods.

4. Manual regulation of coolant boron concentrations.

Incore instrumentation is provided to give information on core power distribution.

Soluble poison concentration is determined by sampling and analysis of primary coolant water.

In addition, instrumentation and controls are provided to maintain the integrity of the reactor core, the primary coolant pressure boundary and the containment structure and its associated systems during all modes of operation including start-up and shutdown.

References: Chapters 4, 6, 7, 9 and Technical Specifications Section 3.3

5.1.3.5 Criterion 14 - Reactor Coolant Pressure Boundary

CRITERION - The reactor coolant pressure boundary shall be designed, fabricated, erected and tested so as to have an extremely low probability of abnormal leakage, of rapidly propagating failure, and of gross rupture.

RESPONSE - The reactor coolant pressure boundary meets this criterion based on the following:

1. Material selection, design, fabrication, inspection, testing and certification were in keeping with the ASME (Section III) and USASI (B31.1) Codes.

2. In addition to the code requirements listed, the Primary Coolant System was designed to meet the cyclic loading and transient conditions specified in Chapter 4.

3. Quality Assurance during fabrication included weld qualification test plates, permanent identification of materials, welder qualification tests and extensive production nondestructive testing.

4. The reactor vessel and other coolant boundary materials were chosen to retain metallurgical stability of the material during the service life considering cyclic effects of mechanical shock and vibratory loadings, radiation effects and the increase in nil ductility transition temperature as a result of neutron irradiation.

References: Chapters 4, 15 and SEP Topic V-6

5.1.3.6 Criterion 15 - Reactor Coolant System Design

CRITERION - The reactor coolant system and associated auxiliary, control and protection systems shall be designed with sufficient margin to assure that the design conditions of the reactor coolant pressure boundary are not exceeded during any condition of normal operation, including anticipated operational occurrences.

RESPONSE - The Reactor Coolant System, including its auxiliary, control and protection systems, was designed in compliance with this criterion by using appropriate codes and standards as detailed in Section 5.2. The combination of systems and equipment assures that the reactor coolant pressure boundary design conditions are not exceeded by normal operation including anticipated operational occurrences.

At normal pressure and temperature, the boundary protection is provided by the primary safety valves. An automatic reactor trip is provided on high primary coolant pressure to prevent excessive blowdown of the coolant by relief action through the primary safety valves. During start-up and shutdown, overpressure protection is provided by the pressurizer power operated relief valves automatically opening upon sensing a low-temperature/high-pressure condition.

References: Chapters 3, 4, 5, 7, 9, 10 and Technical Specifications Sections 2.1 and 3.4

5.1.3.7 Criterion 16 - Containment Design

CRITERION - Reactor containment and associated systems shall be provided to establish an essentially leak tight barrier against the uncontrolled release of radioactivity to the environment and to assure that the containment design conditions important to safety are not exceeded for as long as postulated accident conditions require.

RESPONSE - The containment building is a post-tensioned concrete cylinder and dome connected to a conventionally reinforced slab foundation. Its entire inner surface is lined with welded steel plate to ensure a high degree of leak tightness. The containment building completely encloses the reactor and the Primary Coolant System. The design of the containment building (Section 5.8) in conjunction with the design of the engineered safeguards systems (Chapters 6 and 7) ensure that the leak-tight integrity of the reactor building is maintained under normal and accident conditions. Therefore, even if a gross failure of the Primary Coolant System were to occur, the leakage of radioactive materials to the environment would not exceed the limits of 10 CFR 100.

References: Chapters 5, 6, 7, 14, Technical Specifications Section 3.6 and SEP Topic III-7B

5.1.3.8 Criterion 17 - Electrical Power Systems

CRITERION - An onsite electric power system and an offsite electric power system shall be provided to permit functioning of structures, systems and components important to safety. The safety function for each system (assuming the other system is not functioning) shall be to provide sufficient capacity and capability to assure that (1) specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded as a result of anticipated operational occurrences and (2) the core is cooled and containment integrity and other vital functions are maintained in the event of postulated accidents.

The onsite electric power supplies, including the batteries, and the onsite electric distribution system, shall have sufficient independence, redundancy and testability to perform their safety functions assuming a single failure. Electric power from the transmission network to the onsite electric distribution shall be supplied by two physically independent circuits (not necessarily on separate rights of way) designed and located so as to minimize to the extent practical the likelihood of their simultaneous failure under operating and postulated accident and environmental conditions. A switchyard common to both circuits is acceptable. Each of these circuits shall be designed to be available in sufficient time following a loss of all onsite alternating current power sources and the other offsite electric power circuit, to assure that specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded. One of these circuits shall be designed to be available within a few seconds following a Loss of Coolant Accident to assure that core cooling, containment integrity, and other vital safety functions are maintained.

Provisions shall be included to minimize the probability of losing electric power from any of the remaining supplies as a result of, or coincident with, the loss of power generated by the nuclear power unit, the loss of power from the transmission network, or the loss of power from the onsite electric power supplies.

<u>RESPONSE</u> - The 2,400 volt engineered safeguards electrical power system was designed to permit functioning of structures, systems and components which assure that: (1) specified fuel design limits and design conditions of the primary coolant pressure boundary are not exceeded as the result of operational occurrences; and (2) the core is cooled and containment structure integrity and other vital functions are maintained in the event of postulated accidents.

Electrical power from the offsite transmission network to the onsite Class 1E distribution system is fed from one switchyard by two separate circuits into the Plant. One circuit is routed on overhead transmission towers while the other is routed underground. Both circuits are considered immediately accessible regardless of power generation. Upon loss of power from the normal offsite power circuit, the onsite electrical power system is switched to the second immediate access circuit. If this source is unavailable, the onsite emergency power systems are immediately available to feed the Plant systems and components necessary for Plant safety. Capability is also provided to power the onsite electrical system from the main turbine generator. A delayed access circuit is also available when the turbine generator is out of service. This delayed access circuit is designed to be available in sufficient time to assure the specified acceptable fuel design limits and design conditions of the primary coolant pressure boundary are not exceeded.

The emergency (onsite) generators, distribution systems and controls for this equipment are independent and redundant so that they can perform their safety function assuming a single failure. In addition, the onsite power system can be periodically tested to assure that it can function properly.

In addition to the onsite emergency diesel generator power supplies, a station emergency dc 125 volt battery power supply is available as a backup to supply dc minimum I&C functions, critical lighting, and preferred ac loads for 2 hours and up to 4 hours with load shedding of certain nonessential loads giving ample time for repair of the power sources.

References: Chapter 8, Technical Specifications Section 3.8, and SEP Topics VI-7C1, VIII-1A and VIII-2

5.1.3.9 Criterion 18 - Inspection and Testing of Electrical Power Systems

CRITERION - Electric power systems important to safety shall be designed to permit appropriate periodic inspection and testing of important areas and features, such as wiring, insulation, connections and switchboards to assess the continuity of the systems and the condition of their components. The systems shall be designed with a capability to test periodically (1) the operability and functional performance of the components of the systems, such as onsite power sources, relays, switches and buses, and (2) the operability of the systems as a whole and, under conditions as close to design as practical, the full operation sequence that brings the systems into operation, including operation of applicable portions of the protection system and the transfer of power among the nuclear power unit, the offsite power system, and the onsite power system.

RESPONSE - Electrical power systems important to safety can be periodically inspected and tested. Continuous monitoring is provided on power systems that are normally energized. Emergency power sources can be tested periodically to assure that they are operable and functional and their output can be monitored.

The operability of the power systems as a whole including transfer of power from one source to another may be checked while the reactor is noncritical.

References: Chapter 8, Technical Specifications Section 3.8, and SEP Topic VIII-3A

5.1.3.10 Criterion 19 - Control Room

CRITERION - A control room shall be provided from which actions can be taken to operate the nuclear power unit safely under normal conditions and to maintain it in a safe condition under accident conditions, including Loss of Coolant Accidents. Adequate radiation protection shall be provided to permit access and occupancy of the control room under accident conditions without personnel receiving radiation exposures in excess of 5 Rem whole body, or its equivalent to any part of the body, for the duration of the accident.

Equipment at appropriate locations outside the control room shall be provided (1) with a design capability for prompt hot shutdown of the reactor, including necessary instrumentation and controls to maintain the unit in a safe condition during hot shutdown, and (2) with a potential capability for subsequent cold shutdown of the reactor through the use of suitable procedures.

RESPONSE - The control room habitability systems include radiation protection, air purification, climatically controlled ventilation and air conditioning systems, lighting and power systems. Collectively, these habitability systems ensure that the control room operators can remain in the control room and take action to operate the Plant safely under normal conditions and to maintain it in a safe condition under all accident conditions including an SSE or design tornado.

The control room HVAC system functions to prevent air in-leakage during normal and post-accident conditions by filtration of airborne radioactive iodines in the control room atmosphere and also has the ability to purge the room of smoke in the event of a fire. Loss of offsite power will not impair the system's ability to perform its function.

There are controls and instrumentation located outside the control room which enable hot shutdown from outside the control room in the event of a major fire. Cold shutdown is capable of being provided from the control room and from locations remote from the control room.

The original design basis for control room shielding was to limit whole-body dose to plant personnel to less than 25 rem after 30 days following a Design Basis Accident (DBA). As a result of NUREG 0737 Item III.D.3.4 and II.B.2.2, Licensing Bases were changed, Control Room doses were reanalyzed and HVAC was modified. The current Licensing Basis for Control Room habitability is to assure that occupancy can be maintained under accident conditions without personnel receiving radiation exposures in excess of 5 rem whole body or its equivalent to any part of the body, for the duration of an accident.

The References: Chapters 6, 7, 11, SEP Topic VI-8 and NUREG-0737, Item III.D.3.4

5.1.3.11 Conclusions

Based on the discussions in various parts of Subsection 5.1.3, it is concluded that the Palisades Plant was designed in conformance with the intent of Group II criteria, except Criterion 17 has since been interpreted to require separate transmission towers for the circuits bringing power into the Plant from the substation. As discussed in 5.1.3.8 above and Chapter 8, the plant has been modified to provide two independent, immediate access sources of offsite power from the switchyard.

5.1.4 GROUP III: PROTECTION AND REACTIVITY CONTROL SYSTEMS (CRITERIA 20-29)

5.1.4.1 Criterion 20 - Protection System Functions

CRITERION - The protection system shall be designed (1) to initiate automatically the operation of appropriate systems including the reactivity control systems, to assure that specified acceptable fuel design limits are not exceeded as a result of anticipated operational occurrences and (2) to sense accident conditions and to initiate the operation of systems and components important to safety.

RESPONSE - The reactor is protected by the Reactor Protective System (RPS), the engineered safeguards actuation system and the auxiliary feedwater actuation system from reaching a condition that could result in exceeding acceptable fuel damage limits.

The RPS is designed to monitor the reactor operating conditions and initiate a fast shutdown if any of measured variables exceed the operating limits. The parameters and conditions which will initiate a trip are the following:

1. High neutron level (reactor power)
2. High start-up rate (at < 15% {nominal} power)
3. High pressurizer pressure
4. Thermal margin/low primary coolant pressure
5. Loss of turbine load (at > 15% {nominal} power)
6. Low reactor coolant flow
7. Low steam generator level
8. Steam generator low pressure
9. Containment high pressure

The engineered safeguards actuation system senses low primary coolant pressure and high containment building pressure, either of which initiates safety injection. The containment building pressure signal also initiates emergency containment spray cooling and isolation of the building. A signal of high radiation in the containment building also initiates isolation of the building. The auxiliary feedwater actuation system senses low steam generator level which initiates emergency feed to at least one steam generator for continued reactor decay heat removal.

Analyses of all accident situations examined, including the postulated DBA, high energy pipe breaks and fires, indicate that the system monitoring sensors provided in the design initiate the operation of necessary emergency systems to protect the reactor core and Primary Coolant System and to mitigate radiation releases to the environment.

References: Chapters 6 and 7, Technical Specifications Section 3.3, and SEP Topics VI-7A3 and VII-2

5.1.4.2 Criterion 21 - Protection System Reliability and Testability

CRITERION - The protection system shall be designed for high functional reliability and inservice testability commensurate with the safety functions to be performed. Redundancy and independence designed into the protection system shall be sufficient to assure that (1) no single failure results in loss of the protection function and (2) removal from service of any component or channel does not result in loss of the required minimum redundancy unless the acceptable reliability of operation of the protection system can be otherwise demonstrated. The protection system shall be designed to permit periodic testing of its functioning when the reactor is in operation, including a capability to test channels independently to determine failures and losses of redundancy that may have occurred.

RESPONSE - The protection systems' design meets this criterion by specification of high quality components, ample design capacity, component redundancy and inservice testability. The following principal design criteria have been applied to the design of the instrumentation:

1. No single component failure shall prevent the protection systems from fulfilling their protective function when action is required.

2. No single component failure shall initiate unnecessary protection system action, provided implementation does not conflict with the criterion above. Exceptions to this criteria include the reactor trips on high startup rate and loss of turbine load.

Testing facilities are built into the protection systems to provide for:

1. Preoperational testing to give assurance that the protection systems can fulfill their required functions.

2. On-line testing to assure operability and to demonstrate reliability.

Each channel of the protection systems, including the sensors up to the final protection element, is capable of being checked during reactor operation. Measurement sensors of each channel used in protection systems can be checked by observing outputs of similar channels which are presented on indicators and recorders on the control board. Protective actuation units and logic can be tested by inserting a signal into the measurement channel ahead of the readout and, upon application of a protective actuation signal level input, observing that a signal is passed through the trip units and the logic to the logic output relays. The logic output relays can be tested individually for initiation of appropriate action.

References: Chapter 7, Technical Specifications Section 3.3, SEP Topics VI-7C, VI-10A, VII-2 and VII-1

5.1.4.3 Criterion 22 - Protection System Independence

CRITERION - The protection system shall be designed to assure that the effects of natural phenomena, and of normal operating, maintenance, testing and postulated accident conditions on redundant channels do not result in loss of the protection function, or shall be demonstrated to be acceptable on some other defined basis. Design techniques, such as functional diversity or diversity in component design and principles of operation, shall be used to the extent practical to prevent loss of the protection function.

RESPONSE - The instrumentation systems provided for the initiation of protection functions use redundancy as one of the means to assure that protective action can be effected. Four channels are available to monitor each critical parameter. The channels are independent, eg, with respect to piping, wiring, mounting and the supply of power. This independence permits testing and removal from service of any component or channel without loss of the protective function. Where feasible, protection functions initiated by one critical parameter are backed up by trips initiated on different conditions.

References: Chapter 7, Technical Specifications Section 3.3, and SEP Topic VII-2

5.1.4.4 Criterion 23 - Protection System Failure Modes

CRITERION - The protection system shall be designed to fall into a safe state or into a state demonstrated to be acceptable on some other defined basis if conditions such as disconnection of the system, loss of energy (eg, electric power, instrument air), or postulated adverse environments (eg, extreme heat or cold, fire, pressure, steam, water and radiation) are experienced.

RESPONSE - The Reactor Protective System is designed so that loss of power to each reactor protection channel will trip that individual channel because most of the signals in each channel trip on loss of power. Loss of power to two or more channels will trip the reactor protection system, thereby releasing the control rods. The system is designed for continuous operation under credible adverse environments.

Other protective systems follow the same principle and are activated upon loss of instrument power. Some systems, such as safety injection, are active and therefore require power for full activation. Full activation is not desired for these systems during power failure.

References: Chapter 7, Technical Specifications Section 3.8, and SEP Topic VII-2

5.1.4.5 Criterion 24 - Separation of Protection and Control Systems

CRITERION - The protection shall be separated from control systems to the extent that failure of any single control system component or channel, or failure or removal from service of any single protection system component or channel which is common to the control and protection system leaves intact a system satisfying all reliability, redundancy and independence requirements of the protection system. Interconnection of the protection and control systems shall be limited so as to assure that safety is not significantly impaired.

RESPONSE - The protection systems are separated from the control instrumentation systems so that failure or removal from service of any control instrumentation system component or channel does not inhibit the function of the protection system.

Interconnections between the protection and control systems are provided with isolation as discussed in Chapter 7.

References: Chapter 7 and SEP Topic VII-1A

5.1.4.6 Criterion 25 - Protection System Requirements for Reactivity Control Malfunctions

CRITERION - The protection system shall be designed to assure that specified acceptable fuel design limits are not exceeded for any single malfunction of the reactivity control systems, such as accidental withdrawal (not ejection or dropout) of control rods.

RESPONSE - The reactor protective and other protection systems provide adequate reactor protection to assure that acceptable fuel design limits are not exceeded for any single malfunction in the reactivity control systems.

Two independent rod position indication systems are provided with interlocks on rod motion to prevent misalignment of control rods.

References: Chapters 7, 14, Technical Specifications Section 3.8, and SEP Topics IV-2 and XV-8

5.1.4.7 Criterion 26 - Reactivity Control System Redundancy and Capability

CRITERION - Two independent reactivity control systems of different design principles shall be provided. One of the systems shall use control rods, preferably including a positive means for inserting the rods and shall be capable of reliably controlling reactivity changes to assure that under conditions of normal operations, including anticipated operational occurrences, and with appropriate margin for malfunctions such as stuck rods, specified acceptable fuel design limits are not exceeded. The second reactivity control system shall be capable of reliably controlling the rate of reactivity changes resulting from planned, normal power changes (including xenon burnout) to assure acceptable fuel design limits are not exceeded. One of the systems shall be capable of holding the reactor core subcritical under cold conditions.

RESPONSE - The reactivity control system employs two separate methods of adjusting reactivity: (1) mechanically driven control element assemblies; and (2) adjustment of the concentration of boric acid in the primary coolant.

The control rod system controls short-term reactivity changes, such as the reactivity change required for power changes and power distribution shaping, and is also used for reactor protection. The boric acid shim control compensates for long-term reactivity changes such as those associated with fuel burnup, variation in the xenon and samarium concentrations and plant cooldown and heatup.

References: Chapters 3, 4, 7, 9, 14 and SEP Topic IX-4

5.1.4.8 Criterion 27 - Combined Reactivity Control Systems Capability

CRITERION - The reactivity control systems shall be designed to have a combined capability, in conjunction with poison addition by the emergency core cooling system, of reliably controlling reactivity changes to assure that under postulated accident conditions and with appropriate margin for stuck rods the capability to cool the core is maintained.

RESPONSE - The control rods are capable of making the reactor subcritical both under normal operating conditions and under the accident conditions set forth in Chapter 14. The reactor is designed with the capability of providing an adequate shutdown margin with the single most reactive control rod fully withdrawn at any point in core life with the reactor at a hot, zero power condition. The rate of reactivity compensation from boron addition is greater than the reactivity change associated with the maximum allowable reactor cooldown rate of 100°F/hour. Thus, subcriticality is assured during cooldown with the most reactive control rod totally unavailable.

References: Chapters 3, 4, 7, 9, 14 and SEP Topic IX-4

5.1.4.9 Criterion 28 - Reactivity Limits

CRITERION - The reactivity control systems shall be designed with appropriate limits on the potential amount and rate of reactivity increase to assure that the effects of postulated reactivity accidents can neither (1) result in damage to the reactor coolant pressure boundary greater than limited local yielding nor (2) sufficiently disturb the core, its support structures or other reactor pressure vessel internals to impair significantly the capability to cool the core. These postulated reactivity accidents shall include consideration of rod ejection (unless prevented by positive means), rod dropout, steam line rupture, changes in reactor coolant temperature and pressure, and cold water addition.

RESPONSE - The basis for selecting the number of control rods in the core includes that of assuring that the reactivity worth of any one rod is within a preselected maximum value. Ejection of the maximum worth control rod will not lead to further coolant boundary rupture or internals damage which would interfere with emergency core cooling.

The control rods are separated into two groups: a shutdown group and a regulating group which are further subdivided into groups as necessary. Administrative procedures and interlocks are used to permit only one shutdown group to be withdrawn at a time and to permit withdrawal of the regulating groups only when the shutdown groups are fully withdrawn. The regulating groups are programmed and interlocked to move in sequences and within limits that prevent the rates of reactivity change and the worth of individual control rods from exceeding the selected limiting values.

References: Chapters 3, 7, 14, Technical Specifications Section 3.1, and SEP Topic XV-12

5.1.4.10 Criterion 29 - Protection Against Anticipated Operational Occurrences

CRITERION - The protection and reactivity control systems shall be designed to assure an extremely high probability of accomplishing their safety functions in the event of anticipated operational occurrences.

RESPONSE - The protection and reactivity control systems are designed to assure that their functional capability can be accomplished during anticipated operational occurrences.

The protective system instrumentation is designed for continuous operation at the maximum expected temperature and humidity during normal operation, and components which are located inside the containment building and which are designed to operate following a DBA will withstand the post-DBA environment for the required period of time.

References: Chapters 7 and 8

5.1.4.11 Conclusions

Based on the discussions in various parts of Subsection 5.1.4, it is concluded that the Palisades Plant is designed in conformance with the intent of Group III criteria.

5.1.5 GROUP IV: FLUID SYSTEMS (CRITERIA 30-46)

5.1.5.1 Criterion 30 - Quality of Reactor Coolant Pressure Boundary

CRITERION - Components which are part of the reactor coolant pressure boundary shall be designed, fabricated, erected and tested to the highest quality standards practical. Means shall be provided for detecting and, to the extent practical, identifying the location of the source of reactor coolant leakage.

RESPONSE - The primary coolant pressure boundary components were designed, fabricated, erected and tested to the highest quality standards available at that time. The codes utilized were ASME, Section III, including all addenda through Winter 1965 and the Code for Pressure Piping, ASA B31.1, 1955.

The primary coolant pressure boundary is monitored for detecting leakage of primary coolant by the following means:

1. Containment Radiation Level - One gas monitor, four area monitors and two high-range area monitors are located in the containment for continuous monitoring of the atmosphere.

2. Condenser Off-Gas - A gas monitor is provided to detect any radioactive noncondensible gases in the condenser vacuum system discharge.

3. Steam Generator Blowdown Water and Steam - The blowdown tank drain and steam lines are monitored continuously.

4. Containment Humidity and Temperature - The humidity and temperature of the air in the containment are continuously monitored.

5. Containment Sump Level - Primary coolant leakage reaching the containment building sump would be found by the operator's shift surveillance of the containment level indicators, or annunciated in the control room by activation of the sump high-level or high-high alarms. The sump high level alarm is annunciated by signals from LS-0358 and LS-0360. These instruments are not environmentally or seismically qualified and are relied on for information only.

Alarms will sound in the control room to notify the operator of the existence of larger leaks by low pressurizer level. Abnormal operation of the Chemical and Volume Control System will be indicative of leakage.

References: Chapters 4, 9, 11, Technical Specifications LCO 3.4.15, and SEP Topics III-1, V-6 and VI-3

5.1.5.2 Criterion 31 - Fracture Prevention of Reactor Coolant Pressure Boundary

CRITERION - The reactor coolant pressure boundary shall be designed with sufficient margin to assure that when stressed under operating, maintenance, testing and postulated accident conditions (1) the boundary behaves in a nonbrittle manner and (2) the probability of rapidly propagating fracture is minimized. The design shall reflect consideration of service temperatures and other conditions of the boundary material under operating, maintenance, testing and postulated accident conditions and the uncertainties in determining (1) material properties, (2) the effects of irradiation on material properties, (3) residual, steady-state and transient stresses, and (4) size of flaws.

RESPONSE - The primary coolant pressure boundary design meets this criterion by the following:

1. Brittle failure should not occur if the peak stresses do not exceed the yield stresses in the brittle fracture range. The establishment of temperature-pressure limitations for operation below NDT temperature + 60°F is based on not exceeding yield for the peak stresses. Consumers meets the requirements of 10 CFR 50, Appendix G (May 1983) as amended November 6, 1986, for protection against nonductile failure.

2. Stress limitations are used to establish pressure-temperature operating curves for the Plant. The stress limitations are reflected in Plant heatup and cooldown rates and in inservice leak rate testing. The pressure-temperature operating curves consider heatup and cooldown in both critical and noncritical reactor conditions. Protection against overpressurization of the Primary Coolant System as a function of coolant temperature has been provided.

3. Quality control procedures include permanent identification of materials and nondestructive testing for flaw identification.

4. Operating restrictions are to prevent failure resulting from increase in brittle fracture transition temperature due to neutron irradiation, including a material irradiation surveillance program. Consumers meets the requirements of 10 CFR 50, Appendix H, with regard to reactor surveillance programs. Additionally, Consumers complies with the requirements of 10 CFR 50.61a for protection against pressurized thermal shock events.

5. The primary coolant pressure boundary components were designed, fabricated, erected and tested to the highest quality standards available at that time. The codes utilized were ASME, Section III, including all addenda through Winter 1965 and the code for pressure piping, ASA B31.1, 1955.

References: Chapters 4, 7, Technical Specifications Section 3.4, and SEP Topics III-1, V-6 and VI-3

5.1.5.3 Criterion 32 - Inspection of Reactor Coolant Pressure Boundary

CRITERION - Components which are part of the reactor coolant pressure boundary shall be designed to permit (1) periodic inspection and testing of important areas and features to assess their structural and leak tight integrity, and (2) an appropriate material surveillance program for the reactor pressure vessel.

RESPONSE - The primary coolant pressure boundary design meets this criterion since space has been provided to permit nondestructive testing of critical areas during unit shutdown. A reactor pressure vessel material surveillance program conforming to ASTM-E-185-66 has been established. In addition, the Plant performs inspections according to the Inservice Inspection Program pursuant to 10 CFR 50.55a.

References: Chapter 4, Technical Specifications Section 3.4, and SEP Topic V-5

5.1.5.4 Criterion 33 - Reactor Coolant Makeup

CRITERION - A system to supply reactor coolant makeup for protection against small breaks in the reactor coolant pressure boundary shall be provided. The system safety function shall be to assure that specified acceptable fuel design limits are not exceeded as a result of reactor coolant loss due to leakage from the reactor coolant pressure boundary and rupture of small piping or other small components which are part of the boundary. The system shall be designed to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished using the piping, pumps and valves used to maintain coolant inventory during normal reactor operation.

RESPONSE - During normal operation, the water level in the Primary Coolant System is maintained by the Chemical and Volume Control System. For small breaks in the Primary Coolant System, the charging pumps may be used to supply adequate makeup capability with either onsite or offsite power being available. However, per the Small Break LOCA analysis, the High Pressure Injection System is the system credited for fulfilling this safety function.

References: Chapters 9, 14 and SEP Topic XV-19

5.1.5.5 Criterion 34 - Residual Heat Removal

CRITERION - A system to remove residual heat shall be provided. The system safety function shall be to transfer fission product decay heat and other residual heat from the reactor core at a rate such that specified acceptable fuel design limits and the design conditions of the reactor coolant pressure boundary are not exceeded.

Suitable redundancy in components and features, and suitable interconnections, leak detection and isolation capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure.

RESPONSE - During postulated accident conditions, the long-term cooling of the core is provided by the safety injection pumps. Part of the low pressure safety injection pump flow may be passed through the shutdown cooling heat exchangers which are in turn cooled by the component cooling water system. After passing through the shutdown cooling heat exchangers, containment spray pump discharge is routed to the suction of operating high pressure safety injection pumps via the subcooling line. The low-pressure safety injection pumps can also be used for long-term cooling if the Primary Coolant System pressure is low enough.

During normal reactor shutdown, the low-pressure safety injection pumps and the shutdown heat exchangers are used to cool the Primary Coolant System from 300°F to refueling temperature and maintain this temperature.

For either accident or normal conditions, there is suitable redundancy in components, isolation capability and power supplies to assure that with either onsite or offsite power, the safety function can be accomplished, assuming a single failure.

References: Chapters 6, 7, 8, 9, 14, Technical Specifications Sections 3.4 and 3.9, and SEP Topics VI-7, VII-2 and VIII-3

5.1.5.6 Criterion 35 - Emergency Core Cooling

CRITERION - A system to provide abundant emergency core cooling shall be provided. The system safety function shall be to transfer heat from the reactor core following any loss of reactor coolant at a rate such that (1) fuel and clad damage that could interfere with continued effective core cooling is prevented and (2) clad metal-water reaction is limited to negligible amounts.

Suitable redundancy in component and features, and suitable interconnections, leak detection, isolation and containment capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure.

RESPONSE - As discussed in Chapter 6, The Palisades Emergency Core Cooling System complies with 10CFR50.46 and Appendix K.

During postulated accident conditions, the long term cooling of the core is provided by the safety injection pumps. Part of the low pressure safety injection pump flow may also be passed through the shutdown cooling heat exchangers which are in turn cooled by the component cooling water system. After passing through the shutdown cooling heat exchangers, containment spray pump discharge is routed to the suction of operating high pressure safety injection pumps via the subcooling line. The low pressure safety injection pumps can also be used for long term cooling if the primary coolant system pressure is low enough.

For either accident or normal conditions, there is suitable redundancy in components, isolation capability, and power supplies to assure that with either onsite or offsite power, the safety function can be accomplished assuming a single failure.

References: Section 6, 7, 8, 9 and 14, and Technical Specifications
Section 3.5

5.1.5.7 Criterion 36 - Inspection of Emergency Core Cooling System

CRITERION - The emergency core cooling system shall be designed to permit appropriate periodic inspection of important components, such as spray rings in the reactor pressure vessel, water injection nozzles, and piping, to assure the integrity and capability of the system.

RESPONSE - Important components of the Emergency Core Cooling System can be periodically inspected. All piping and valves inside the containment building can be inspected when the reactor is shut down. Piping, pumps, valves and electrical equipment outside of the containment building can be inspected at any time, but Technical Specifications, Limiting Conditions of Operations, preclude certain testing until Plant shutdown.

Reference: Technical Specifications Section 3.5

5.1.5.8 Criterion 37 - Testing of Emergency Core Cooling System

CRITERION - The emergency core cooling system shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leak tight integrity of its components, (2) the operability and performance of the active components of the system, and (3) the operability of the system as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the system into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of the associated cooling water system.

RESPONSE - The Emergency Core Cooling System and its auxiliaries receive periodic functional testing. The transfer between normal and emergency power sources is accomplished when the reactor is shut down; however, all other active components, except for certain primary system instrumentation sensors, are tested during reactor operation.

With the Plant at operating pressure, the high-pressure and low-pressure safety injection pumps and valves may be tested by recirculating borated water back to the SIRW tank. This verifies flow path continuity in the high-pressure injection lines. The borated water flow path from the safety injection tanks can be tested to verify flow path continuity from each tank via its associated main safety injection header to the reactor vessel.

References: Chapters 6 and 7, Technical Specifications Sections 3.4, 3.5, and 3.9, and SEP Topics VI-7.A.3 and VI-10A

5.1.5.9 Criterion 38 - Containment Heat Removal

CRITERION - A system to remove heat from the reactor containment shall be provided. The system safety function shall be to reduce rapidly, consistent with the functioning of other associated systems, the containment pressure and temperature following any Loss of Coolant Accident and maintain them at acceptable low levels.

Suitable redundancy in components and features, and suitable interconnections, leak detection, isolation and containment capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure.

RESPONSE - The containment building design includes two accident heat removal systems each with redundant components, the Containment Spray System and the Containment Air Recirculation and Cooling System. Redundant equipment combinations are provided that will prevent the containment design pressure from being exceeded with the failure of any active component except as discussed in FSAR Section 14.18 and elsewhere. The containment response analyses described in Section 14.18 consider cases both with and without offsite power, as the affects of primary coolant pumps continuing to operate with offsite power available will make these cases more limiting under most conditions.

This safety function is activated by the engineered safeguards actuation system and can be accomplished by either system with either onsite or offsite power.

Under SEP Topic VI-3, the NRC determined that a single failure of the MSIV in the unbroken line to close could allow blowdown of both generators via reverse flow through the swing disc MSIV check valve in the broken line for a break upstream of the MSIV. This failure had not been considered in the Main Steam Line Break (MSLB) analysis. Another single failure not considered in the analysis is a failure of a main feedwater isolation valve or feedwater bypass valve to close. Consumers Power Company transmitted a probabilistic risk assessment and a cost benefit evaluation for modifications to prevent these single failures. In an SER dated February 28, 1986, the NRC found that a double steam generator blowdown or a single steam generator blowdown with continued feedwater, although more severe than the licensing basis MSLB, is not expected to result in unacceptable consequences. The SER determined that the potential offsite consequences are low and that the proposed modifications would not provide a substantial improvement in plant safety. These potential single failures, therefore, are not required to be considered in this analysis.

References: Chapters 5, 6, 7, 8, 14, Technical Specifications LCO 3.6.6, and SEP Topic VI-3

5.1.5.10 Criterion 39 - Inspection of Containment Heat Removal System

CRITERION - The containment heat removal system shall be designed to permit appropriate periodic inspection of important components, such as the torus, sumps, spray nozzles and piping to assure the integrity and capability of the system.

RESPONSE - The Containment Spray System essential equipment, except risers, distribution header piping, spray nozzles and the containment sump, is located outside of the containment and may be inspected at any time. The containment sump and the spray pipe and nozzles can be inspected during shutdowns.

The Containment Air Cooling System has cooler-condensers and blowers inside the containment building and they can be inspected during shutdowns.

References: Chapter 6, Technical Specifications LCO 3.6.6, and SEP Topic VI-3

5.1.5.11 Criterion 40 - Testing of Containment Heat Removal System

CRITERION - The containment heat removal system shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leak tight integrity of the components, (2) the operability and performance of the active components of the system, and (3) the operability of the system as a whole, and, under conditions as close to the design as practical, the performance of the full operational sequence that brings the system into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources and the operation of the associated cooling water system.

RESPONSE - The Containment Air Recirculation and Cooling System is normally in service. Valving on the coils can be cycled, thus placing the coils into emergency configuration periodically during operation. The Containment Spray System is tested on a periodic basis as follows:

Containment Spray Pumps - These pumps are tested singly by establishing a recirculation flow path to the Safety Injection and Refueling Water (SIRW) tank. Each pump, in turn, is manually started and checked for flow.

Containment Spray Nozzles - With the unit shut down, air or smoke is blown through the test connections with visual observation of the nozzles.

References: Chapter 6 and Technical Specifications LCO 3.6.6

5.1.5.12 Criterion 41 - Containment Atmosphere Cleanup

CRITERION - Systems to control fission products, hydrogen, oxygen and other substances which may be released into the reactor containment shall be provided as necessary to reduce, consistent with the functioning of other associated systems, the concentration of hydrogen or oxygen and other substances in the containment atmosphere following postulated accidents to assure that containment integrity is maintained.

Each system shall have suitable redundancy in components and features, and suitable interconnections, leak detection, isolation and containment capabilities to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) its safety function can be accomplished assuming a single failure.

RESPONSE - The Sodium Tetraborate (NaTB) addition in conjunction with the Containment Spray System acts to reduce the post-accident level of fission products in the containment atmosphere. Before the onset of the recirculation phase after a LOCA, the NaTB would be dissolved in the containment sump solution in order to establish a neutral pH and to provide for iodine retention. The NaTB addition is completely passive, requiring no active mechanical or operator action. Therefore, the safety function can be accomplished with either onsite or offsite power available and assuming a single failure.

References: Chapters 6, 7, 8, 14, Technical Specifications LCO 3.5.5 and SEP Topic VI-5

5.1.5.13 Criterion 42 - Inspection of Containment Atmosphere Cleanup Systems

CRITERION - The containment atmosphere cleanup systems shall be designed to permit appropriate periodic inspection of important components, such as filter frames, ducts and piping to assure the integrity and capability of the systems.

RESPONSE - The baskets of sodium tetraborate are located inside the containment building and can be inspected during operation, if necessary.

References: Chapters 6 and 7, and Technical Specifications LCO 3.5.5

5.1.5.14 Criterion 43 - Testing of Containment Atmosphere Cleanup Systems

CRITERION - The containment atmosphere cleanup systems shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leak tight integrity of its components, (2) the operability and performance of the active components of the systems such as fans, filters, dampers, pumps and valves and (3) the operability of the systems as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the systems into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of associated systems.

RESPONSE - The baskets of sodium tetraborate are completely passive and not subject to pressure or functional testing.

References: Chapter 6 and Technical Specifications LCO 3.5.5

5.1.5.15 Criterion 44 - Cooling Water

CRITERION - A system to transfer heat from structures, systems and components important to safety, to an ultimate heat sink shall be provided. The system safety function shall be to transfer the combined heat load of these structures, systems and components under normal operating and accident conditions.

Suitable redundancy in components and features, and suitable interconnections, leak detection, and isolation capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure.

RESPONSE - Structures, systems and components important to safety have their heat removed by air-to-water or water-to-water heat exchangers cooled by the Component Cooling Water and the Service Water Systems. The SWS is an "open" system utilizing Lake Michigan as its continuous water supply (ultimate heat sink) by taking suction from the lake through the SWS pumps which are located in the intake pump house. These pumps draw water from the pump house bay which is supplied with water from the offshore intake crib pipeline.

The cooling water systems are designed to remove the heat generated under normal and accident conditions. As part of the engineered safeguards actuation system, equipment, controls and power supplies are sufficiently independent and redundant so that the systems' safety functions can be accomplished with either onsite or offsite power available and assuming a single failure.

References: Chapters 6, 7, 8 and 9, Technical Specifications Section 3.7 and SEP Topic III-3.C

5.1.5.16 Criterion 45 - Inspection of Cooling Water System and Criterion 46 - Testing of Cooling Water System

CRITERION 45 - The cooling water system shall be designed to permit appropriate periodic inspection of important components, such as heat exchangers and piping, to assure the integrity and capability of the system.

CRITERION 46 - The cooling water system shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leak tight integrity of its components, (2) the operability and the performance of the active components of the system, and (3) the operability of the system as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the system into operation for reactor shutdown and for Loss of Coolant Accident, including operation of applicable portions of the protection system and the transfer between normal and emergency power sources.

RESPONSE - The Component Cooling and Service Water Systems are in service at all times and can be inspected at any time except for sections of each system which are either inside containment or underground. Inside containment systems are inspected when the Plant is shut down; underground systems are pressure tested during Plant shutdown.

There are no special testing capabilities designed into the systems since they are in service at all times. Functional and performance testing of systems and their components can be performed during operational and shutdown modes.

References: Chapter 9, Technical Specifications Section 3.7 and SEP Topic IX-3

5.1.5.17 Conclusions

Based on the discussions in various parts of Subsection 5.1.5, it is concluded that the Palisades Plant is designed in conformance with the intent of Group IV criteria.

5.1.6 GROUP V: REACTOR CONTAINMENT (CRITERIA 50-57)

5.1.6.1 Criterion 50 - Containment Design Basis

CRITERION - The reactor containment structure, including access openings, penetrations and the containment heat removal system shall be designed so that the containment structure and its internal compartments can accommodate, without exceeding the design leakage rate and, with sufficient margin, the calculated pressure and temperature conditions resulting from any Loss of Coolant Accident. This margin shall reflect consideration of (1) the effects of potential energy sources which have not been included in the determination of the peak conditions, such as energy in steam generators and energy from metal-water and other chemical reactions that may result from degraded emergency core cooling functioning (2) the limited experience and experimental data available for defining accident phenomena and containment responses, and (3) the conservatism of the calculational model and input parameters.

RESPONSE - The containment building and engineered safeguards systems have been analyzed for various combinations of credible energy releases. The analysis includes system energy and decay heat. The containment structure, including access openings and penetrations, is designed for a leak rate of 0.1% by weight per day at the design pressure of 55 psig and the design temperature of 283°F. Under Design Basis Accident (DBA) conditions, the site boundary and offsite doses are below the guidelines of 10 CFR 100. The predicted transient peak pressures, associated with either a postulated Main Steam Line Break or a rupture of the piping in the Primary Coolant System and the calculated effects of a metal-water reaction, do not exceed the 55 psig design pressure. However, the predicted transient peak temperatures associated with these events do briefly exceed the design temperature of 283 °F. The brevity of the temperature excursions ensure that they have a negligible effect on the containment structure. See Sections 5.8 and 14.18 for additional information.

The use of ECCS for core flooding limits the reactor building pressure to less than the design pressure.

The high-pressure and low-pressure injection systems and safety injection tanks have redundancy of equipment to ensure availability of capacity.

References: Chapters 5, 6, 11, 14, Technical Specifications Section 3.6, and SEP Topics VI-2, VI-3, VI-7, VII-2, XV-6 through XV-20

5.1.6.2 Criterion 51 - Fracture Prevention of Containment Pressure Boundary

CRITERION - The reactor containment boundary shall be designed with sufficient margin to assure that under operating, maintenance, testing and postulated accident conditions (1) its ferritic materials behave in a nonbrittle manner and (2) the probability of rapidly propagating fracture is minimized. The design shall reflect consideration of service temperatures and other conditions of the containment boundary material during operation, maintenance, testing and postulated accident conditions, and the uncertainties in determining (1) material properties, (2) residual, steady state and transient stresses, and (3) size of flaws.

RESPONSE - The containment structure was designed and constructed in accordance with ACI-318-63, ACI-301-72 (proposed) and the ASME Pressure Vessel Code, Sections III, VIII and IX, 1965. All penetrations were designed, fabricated, inspected and installed in accordance with Subsection B, Section III of the ASME Boiler and Pressure Vessel Code, 1965.

References: Chapter 5, Technical Specifications Section 3.6, and SEP Topic III-7

5.1.6.3 Criterion 52 - Capability for Containment Leakage Rate Testing

CRITERION - The reactor containment and other equipment which may be subjected to containment test conditions shall be designed so that periodic integrated leakage rate testing can be conducted at containment design pressure.

RESPONSE - The containment structure and equipment located inside the containment building were designed so that the preoperational integrated leak rate test could be conducted with air at design pressure. Periodic integrated leak rate tests (ILRTs) are conducted in accordance with 10CFR50.54(o) and Appendix J.

References: Chapter 5, Technical Specifications Section 3.6, and SEP Topic VI-6

5.1.6.4 Criterion 53 - Provisions for Containment Testing and Inspection

CRITERION - The reactor containment shall be designed to permit (1) appropriate periodic inspection of all important areas, such as penetrations, (2) an appropriate surveillance program, and (3) periodic testing at containment design pressure of the leak tightness of penetrations which have resilient seals and expansion bellows.

RESPONSE - The containment building is designed to permit inspection of areas such as penetrations and air locks.

The personnel air lock, the emergency air lock and the equipment hatch contain double seals which can be pressurized from outside the containment building. This feature allows periodic local leak rate testing to be conducted during both operation and shutdown. The electrical penetration canisters also can be pressurized from outside the containment. The piping and ventilation penetrations are of the rigid welded type and are solidly anchored to the containment wall, thus precluding any requirement for expansion bellows.

Reference: Chapter 5

5.1.6.5 Criterion 54 - Piping Systems Penetrating Containment

CRITERION - Piping systems penetrating primary reactor containment shall be provided with leak detection, isolation and containment capabilities having redundancy, reliability and performance capabilities which reflect the importance to safety of isolating theses piping systems. Such piping systems shall be designed with a capability to test periodically the operability of the isolation valves and associated apparatus and to determine if valve leakage is within acceptable limits.

RESPONSE - The containment structure is designed so that leakage through all fluid penetrations not serving accident-consequence-limiting systems is minimized by a double barrier so that no single, credible failure or malfunction of an active component can result in loss-of-isolation or intolerable leakage. The installed double barriers take the form of closed piping systems, both inside and outside the containment building, and various types of isolation devices. Provisions have been made for pressure testing between all isolation devices to check for leakage. Automatic isolation valves can be tested at any time while the Plant is in operation or shut down.

References: Chapter 5 and Technical Specifications Section 3.6

5.1.6.6 Criterion 55 - Primary Coolant Pressure Boundary Penetrating Containment

CRITERION - Each line that is part of the reactor coolant pressure boundary and that penetrates primary reactor containment shall be provided with containment isolation valves as follows, unless it can be demonstrated that the containment isolation provisions for a specific class of lines, such as instrument lines, are acceptable on some other defined basis:

a. One locked closed isolation valve inside and one locked closed isolation valve outside containment; or

b. One automatic isolation valve inside and one locked closed isolation valve outside containment; or

c. One locked closed isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment; or

d. One automatic isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment.

Isolation valves outside containment shall be located as close to containment as practical and upon loss of actuating power, automatic isolation valves shall be designed to take the position that provides greater safety.

Other appropriate requirements to minimize the probability or consequences of an accidental rupture of these lines or of lines connected to them shall be provided as necessary to assure adequate safety. Determination of the appropriateness of these requirements, such as higher quality in design, fabrication and testing, additional provisions for inservice inspection, protection against more severe natural phenomena and additional isolation valves and containment, shall include consideration of the population density, use characteristics and physical characteristics of the site environs.

RESPONSE - For lines that are a part of the primary coolant pressure boundary and penetrate the containment structure, the following design bases were implemented:

1. If the line is normally open or may be opened to the Primary Coolant System during power operation, it contains two valves in series. If the line is part of a closed system external to the containment building and is designed for pressure equal to or greater than containment design pressure, at least one of the valves closes automatically when required. If the system external to the containment building is designed for pressure less than containment design pressure, both valves close automatically. Check valves are considered automatic. All other valves are power operated and can be remotely closed from the control room.

2. If the line is connected to the primary system but never opened during power operation, the lines contain two normally closed valves in series. A lock is provided on each valve.

Automatic isolation is performed by the containment isolation subsystem of the engineered safeguards actuation system.

References: Chapters 6 and 7, and Technical Specifications Sections 3.3.4, 3.6.3, and 5.5.2

5.1.6.7 Criterion 56 - Primary Containment Isolation

CRITERION - Each line that connects directly to the containment atmosphere and penetrates primary reactor containment shall be provided with containment isolation valves as follows, unless it can be demonstrated that the containment isolation provisions for a specific class of lines, such as instrument lines, are acceptable on some other defined basis:

a. One locked closed isolation valve inside and one locked closed isolation valve outside containment; or

b. One automatic isolation valve inside and one locked closed isolation valve outside containment; or

c. One locked closed isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment; or

d. One automatic isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment.

Isolation valves outside containment shall be located as close to containment as practical and upon loss of actuating power, automatic isolation valves shall be designed to take the position that provides greater safety.

RESPONSE - For lines that connect directly to the containment atmosphere and penetrate the containment structure, the following design bases were implemented:

1. If the line is normally open or may be opened to the containment atmosphere during power operation, it contains two valves in series which close automatically when required. Check valves are considered automatic. All other valves are power operated and can be remotely closed from the control room.

2. If the line is connected to the containment atmosphere but never opened during power operation, and

   a. If the line is part of a closed system outside the containment building that is designed for a pressure equal to or greater than containment design pressure, the line contains one normally closed manual valve. A mechanical lock is provided on the valve.

   b. If the line is part of a system outside the containment building that is either an open system, or a closed system whose design pressure is less than containment design pressure, the line contains two normally closed manual valves or one normally closed manual valve and a blind flange in series. A mechanical lock is provided on each valve.

Automatic isolation is performed by the containment isolation subsystem of the engineered safeguards actuation system.

References: Chapters 6 and 7, and Technical Specifications Section 3.6

5.1.6.8 Criterion 57 - Closed System Isolation Valves

CRITERION - Each line that penetrates primary reactor containment and is neither part of the reactor coolant pressure boundary nor connected directly to the containment atmosphere shall have at least one containment isolation valve which shall be either automatic, or locked closed, or capable of remote manual operation. This valve shall be outside containment and located as close to the containment as practical. A simple check valve may not be used as the automatic isolation valves.

RESPONSE - Lines that penetrate the containment structure and which are not connected to the primary coolant pressure boundary or directly to the containment atmosphere have the following design bases:

1. If the lines are normally open or may be opened during power operation and are protected from missiles originating inside the containment building, one remote manually operated valve, locked closed manual valve, or automatic isolation valve is provided with check valves considered as automatic.

2. If the lines are normally open or may be opened during power operation and are not missile protected inside the containment building, and:

   a. If the line is part of a closed system external to the containment building that is designed for pressure equal to or greater than containment design pressure, the line contains one remote manually operated valve, locked closed manual valve, or automatic isolation valve with check valves considered as automatic.

   b. If the line is part of a closed system external to the containment building that is designed for pressure less than containment design pressure, the line contains two automatic isolation valves in series with check valves considered as automatic.

3. If the lines are never opened during power operation, they contain two normally closed manual valves in series with a mechanical lock on each valve.

Automatic isolation is performed by the containment isolation subsystem of the engineered safeguards actuation system.

References: Chapters 6 and 7, and Technical Specifications Section 3.6

5.1.6.9 Conclusions

Based on the discussions in various parts of Subsection 5.1.6, it is concluded that the Palisades Plant is designed in conformance with the intent of Group V criteria except for Criteria 55 and 57 exceptions summarized below:

Criterion 55 Exceptions

1. The primary coolant letdown line has one automatic isolation valve closed by the containment isolation signal. Criterion 55 requires two automatic isolation valves.

2. The primary coolant charging line has one remote manual valve and one check valve. This is considered acceptable since the charging line is used during a Safety Injection System actuation as a path for concentrated boric acid to the Primary Coolant System.

The NRC concluded in NUREG-0820 (SEP topic VI-4) that both the letdown line containment isolation arrangement (Penetration 36) and the charging line containment isolation arrangement (Penetration 45) were acceptable and that back fitting to conform to GDC 55 was not required.

Criterion 57 Exceptions

Penetration 65, which provides instrument air to containment, does not comply with General Design Criterion 57. The penetration, which is not missile protected, has a check valve and a normally open control valve, both of which are external to containment. The control valve does not close on a containment isolation signal. The normal function of air-operated components will cause the instrument air system to depressurize outside of containment if the air compressors are de-energized (eg, loss of offsite power). For this reason, the external portion of the system will be treated as if the design pressure was less than the containment building design pressure. The design criteria for this penetration would require two automatic isolation valves in series with check valves considered as automatic.

In NUREG-0820 (SEP Topic VI-4), the NRC accepted the design configuration of containment isolation systems, including penetration 65. The acceptance was based, in part, on a probabilistic risk assessment which found that the containment isolation design issues contributed only 10 percent to the overall containment leakage probability and resolution of the issues had little effect.

The isolation capability of Penetration 65 was reviewed under NUREG-0737 Topic II.E.4.2 and was found to be acceptable. The review concluded that isolation is provided in a way believed to be functionally equivalent to automatic isolation on a containment isolation signal. This conclusion was based on the penetration line being designed for a pressure higher than containment design pressure and the check valve providing isolation should a break occur upstream of the check valve (Reference: Letter from DPHoffman (CPCo) to DMCrutchfield (NRC), December 19, 1980). The conclusion was formally accepted by issuance of the NUREG 1424, the Safety Evaluation Report for the Palisades full term operating license.

Penetration 33 is for the common safety injection tank fill and drain line header. This penetration has two manual valves for containment isolation. These two valves may be opened during power operation as noted by Technical Specification 3.6.3 for the sampling of the safety injection tanks.

Both the feedwater and the auxiliary feedwater lines have check valves. This is considered adequate since the lines are protected from missiles inside containment and are designed for a higher pressure than the containment building design pressure. In addition, the feedwater and auxiliary feedwater control valves can be remotely closed.

The inlet and bypass valves for steam traps upstream of the Main Steam Isolation Valves may be open. This is considered acceptable because any release through these flow paths is already included in the radiological consequences for the bounding accident analyses.

5.1.7 GROUP VI: FUEL AND RADIOACTIVITY CONTROL (CRITERIA 60-64)

5.1.7.1 Criterion 60 - Control of Releases of Radioactive Materials to the Environment

CRITERION - The nuclear power unit design shall include means to control suitably the release of radioactive materials in gaseous and liquid effluents and to handle radioactive solid wastes produced during normal reactor operation, including anticipated operational occurrences. Sufficient holdup capacity shall be provided for retention of gaseous and liquid effluents containing radioactive materials, particularly where unfavorable site environmental conditions can be expected to impose unusual operational limitations upon the release of such effluents to the environment.

RESPONSE - The original liquid and gaseous Radwaste Systems were designed to limit releases from the Plant to a small percentage of the requirements of 10 CFR 20. Modifications were made to the liquid and gaseous Radwaste Systems in 1971-1973 so that these systems could meet the requirements of Appendix I to 10 CFR 50.

Under normal operating conditions, the liquid waste will be treated and recycled for use in the Plant. Under certain abnormal operating conditions when liquid volumes become excessive, the dirty waste will be treated, sampled and released on a batch basis to the environment at less than 10 CFR 20 limits. All processed wastes discharged are monitored prior to release. The gaseous wastes are compressed, held up and monitored prior to release. The system is designed to assure that even under the assumptions of 1% defective fuel, a minimum 60-day holdup period can be utilized. The 60-day holdup period will limit the release of gaseous activity to levels well below the limits of 10 CFR 20 and the potential dose at the site boundary will be below the limits of Appendix I to 10 CFR 50.

The original solid waste system was designed to handle all solid wastes such as evaporator concentrates, spent resins and miscellaneous solids. It was replaced by a bitumen solidification system using an asphalt evaporator/extruder (E/E) system which is capable of handling evaporator concentrates, spent resins and miscellaneous filters. The bitumen solidification system was functionally replaced by a concentrated waste drying system (Reference Section 11.4). The spent resins and filters may also be shipped in a dewatered state in high integrity containers.

Dry solid waste such as contaminated clothing, rags, buffer pads, mops, etc., will be compacted and placed into metal boxes. Non-compactible waste such as pumps, angle iron, etc., will also be placed in metal boxes. Waste placed in storage will be packaged in accordance with NRC, DOT and burial site requirements.

Reference: Chapter 11

5.1.7.2 Criterion 61 - Fuel Storage and Handling and Radioactivity Control

CRITERION - The fuel storage and handling, radioactive waste and other systems which may contain radioactivity shall be designed to assure adequate safety under normal and postulated accident conditions. These systems shall be designed (1) with a capability to permit appropriate periodic inspection and testing of components important to safety, (2) with suitable shielding for radiation protection, (3) with appropriate containment, confinement and filtering systems, (4) with a residual heat removal capability having reliability and testability that reflects the importance to safety of decay heat and other residual heat removal, and (5) to prevent significant reduction in fuel storage coolant inventory under accident conditions.

RESPONSE - The fuel storage and handling, radioactive waste and other systems which may contain radioactivity have been designed so that under normal or postulated accident conditions, adequate safety is provided.

The spent fuel and waste-storage area is shielded to permit operation within limits of 10 CFR 20. It is designed so that accidental releases of radioactivity to the environment from postulated accidents are below 10 CFR 100 guideline values. Since the components are located outside the containment building, they may be inspected while the Plant is in operation; no special testing capabilities are provided.

The normal means of decay heat removal from the spent fuel is by the spent fuel cooling system. Cooling may be augmented by utilizing the Shutdown Cooling System. The most serious failure would be complete loss of water in the storage pool. This is prevented by placing the cooling connections near or above the water level so that the pool cannot be gravity drained. However, a backup water supply is available from the fire system to refill the pool in the unlikely event of a considerable loss of water. Cooling is not required for waste storage tanks due to the low level of radioactive heating expected.

References: Chapters 9, 11, 14, Technical Specifications Section 4.0, and SEP Topic IX-1

5.1.7.3 Criterion 62 - Prevention of Criticality in Fuel Storage and Handling

CRITERION - Criticality in the fuel storage and handling system shall be prevented by physical systems or processes, preferably by use of geometrically safe configurations.

RESPONSE - The spent fuel storage racks have a geometrically safe configuration that provides spacing and poison sufficient to maintain a k_{eff} of less than 1.0 when flooded with unborated water. The boron concentration of 850 ppm in the pool is sufficient to limit K_{eff} to less than 0.95 under normal storage conditions. Although designed to preclude flooding, the dry new fuel storage racks have a geometrically safe configuration that provides spacing and poison sufficient to maintain K_{eff} less than 0.95 when fully flooded with unborated water.

References: Chapter 9, SEP Topic IX-1 and Technical Specifications Section 5.4, NRC correspondence from Robert G Schaaf to Thomas C. Bordine, Docket No. 50-255, October 28, 1997, Amendment No. 207 to the Palisades Operating License

5.1.7.4 Criterion 63 - Monitoring Fuel and Waste Storage

CRITERION - Appropriate systems shall be provided in fuel storage and radioactive waste systems and associated handling areas (1) to detect conditions that may result in loss of residual heat removal capability and excessive radiation levels and (2) to initiate appropriate safety actions.

RESPONSE - The process monitoring system is designed to monitor, indicate, record and alarm so that actions either automatic or manual can be taken to correct excessive radiation levels. All process systems which contribute to plant discharges are monitored prior to entering the various discharge systems. Each discharge system is also monitored, thus providing redundancy of radiation detection for plant effluents. Area monitoring radiation detectors are provided in the spent fuel area and radwaste storage and handling areas to monitor continuously and alarm radiation levels.

Loss of residual heat removal capability for various systems is provided by: temperature and pressure indicators on the Service Water System; temperature indicators, high-temperature alarms and low-pressure alarms on the Component Cooling Water System; high-temperature alarms and pressure indicator on the spent fuel pool cooling system.

References: Chapters 9 and 11, and SEP Topics VII-5 and IX-2

5.1.7.5 Criterion 64 - Monitoring Radioactivity Releases

CRITERION - Means shall be provided for monitoring the reactor containment atmosphere, spaces containing components for recirculation of Loss of Coolant Accident fluids, effluent discharge paths, and the plant environs, for radioactivity that may be released from normal operations, including anticipated operational occurrences and from postulated accidents.

RESPONSE - One gas monitor, four area monitors and two high-range area monitors provide continuous monitoring of the containment building atmosphere. Continuous monitoring is provided on the circulating water discharge and the Plant ventilation stack. In addition, a particulate monitor is provided on the stack. Radiation monitors, located adjacent to the main steam lines, monitor the activity present in the main steam lines. The main paths of waste to the Plant discharge points are continuously monitored. There are no monitors in the engineered safeguards pump rooms for monitoring radioactivity after a Loss of Coolant Accident; however, exhaust paths from the ESF room are monitored for airborne contamination. The two high-range area radiation monitors in containment are designed to operate following a Loss of Coolant Accident (LOCA). The four area radiation monitors are part of the engineered safeguards actuation system as channels for containment isolation upon high radiation. The East and South Radwaste Storage Buildings have a gaseous monitor and the East processing area exhaust has a sample collection system.

A preoperational radiological environmental survey was established to determine the naturally existing radioisotopes in the local environment by monitoring air, water and food chain samples. The operational program has additional sampling stations which are located outside of the influence of Plant operation and several local stations in the area of maximum influence. A comparison of the operational survey and preoperational survey identifies whether any changes in local environmental radiological activity are due to the operation of the facility and to assure that effluent releases are as low as reasonably achievable. The results of the operational survey are submitted in annual reports to the NRC in accordance with the Plant Technical Specifications.

References: Chapters 2 and 11

5.1.7.6 Conclusions

Based on the discussions in various parts of Subsection 5.1.7, it is concluded that the Palisades Plant is designed in conformance with the intent of Group VI criteria except for Criteria 64 in that there are no area monitors in the engineered safeguards pump rooms for monitoring radioactivity after a Loss of Coolant Accident.

5.1.8 OVERALL CONCLUSION

Based on the foregoing comparisons, we have concluded that there are no significant deviations between the Palisades Plant as it now exists and the intent of the NRC General Design Criteria (10 CFR 50, Appendix A) as it existed on July 7, 1971. The few minor deviations that do exist are due to Palisades Plant being designed and constructed prior to the issuance of later interpretations of 10 CFR 50, Appendix A, General Design Criteria.

5.2 CLASSIFICATION OF STRUCTURES, SYSTEMS AND COMPONENTS

5.2.1 BACKGROUND INFORMATION

Classification of structures, systems and components was a rapidly changing engineering practice in the nuclear industry during the late 1960s to middle 1970s. Between the issuance of the Palisades PSAR in 1966 and major Plant modifications (addition of the cooling towers and radwaste modification) in 1974, a series of industrial standards and NRC regulatory requirements were introduced which form the basis for current classification schemes. In addition, these criteria have been refined in the ensuing years and additional new criteria for design/service/post-accident classifications have been issued. Most of the current regulatory guides and industrial codes/standards were either not in existence or in draft form during the design phases of Palisades and preparation of the PSAR. Therefore, the Palisades DESIGN terminology in the FSAR Update may not be consistent with current terminology. The FSAR Update text retains the original design criteria and classification terminology, unless otherwise noted, for recent Plant modifications that were designed to current regulatory criteria and design codes.

5.2.1.1 Classification Overview

Subsequent to establishing Palisades design objectives in the PSAR, several types of classification began to emerge in the commercial nuclear industry. Principally, there were three categories dealing with (1) seismic considerations, (2) quality of construction/manufacturing and (3) a safety class based upon importance of structures, systems and components to function.

Safety aspects were introduced by development of ANSI N18.2-1973, "Nuclear Safety Criteria for the Design of PWR Plants." All components and structures are classified as Safety Class 1, 2 or 3 in accordance with their contribution to nuclear safety. Safety classification of electrical equipment was addressed in the initial issuance of IEEE 308-1971 and IEEE 379-1977. Seismic class components and structures (Seismic Category I) are those important to safety and are designed to withstand the effects of a safe shutdown earthquake (SSE) and remain functional. Those items were identified in the issuance of NRC Regulatory Guide 1.29. Quality Group Classification (A, B, C and D) for mechanical equipment applies to components important to safety containing water and steam. Those systems were identified in the issuance of Regulatory Guide 1.26 and 10 CFR 50.55a, and are governed by Section III of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel (B&PV) Code and addenda. Regulatory Guide 1.26 was used to establish piping system boundaries, but not for making safety-related determinations at that time.

Systematic Evaluation Program Topics III-1 and the "SEP Review of Safe Shutdown Systems" compared the safety and seismic design classifications at each SEP plant to existing NRC criteria at that time. The purpose of this review was not to backfit plants to the newer design classification criteria; rather, its purpose was to identify any safety significant weaknesses in original plant design to be considered for potential upgrade during the Integrated Plant Safety Assessment process. At Palisades, the result of this effort generally confirmed the adequacy of original plant design. The results of this review are summarized in Table 5.2-3.

In summary, some Palisades structures/systems/components have been re-evaluated for either design, service, maintainability or inspection criteria. Table 5.2-1 identifies the classification terminology used in the FSAR Update, its origin and application. Tables 5.2-2 and 5.2-3 provide specific classification of structures, systems and components, excluding electrical equipment. Electrical equipment and instrumentation and control components are identified on Tables 5.2-4 and 5.2-5.

5.2.1.2 <u>Original Palisades Design Review</u>

The general design classification terminology appearing in the FSAR Update is "Consumers Design Class" 1, 2 or 3, as defined in Section 5.2.2 and as originally defined in Appendix A of the 1980 FSAR. This original classification which primarily addressed structures and mechanical systems/components is unique and should not be confused with other classifications. Electrical equipment classification was later backfit to the Plant electrical systems (see Chapter 8). The definition of Consumers Design Classes appears in Subsection 5.2.2. The terminology was developed for the early design phases of Palisades and would today be considered a combination of Safety Class and Seismic Class.

The 1980 FSAR used the terms "Seismic Class 1" or "Class 1 System" when referring to system designs. Other subsequent licensing correspondence also utilized a variety of terms to identify the 1980 FSAR Appendix A classification. As a result, the FSAR Update has standardized on "Consumers Design Class." It must be emphasized that this information reflects how the Plant was originally designed and classified, and not necessarily how the operating Plant is classified for modifications and inspections pursuant to the Inservice Inspection Program (ISI).

5.2.2 CONSUMERS DESIGN CLASSIFICATIONS

5.2.2.1 Design - Class 1

As used in the FSAR Update, Consumers Design Class 1 structures, systems and components are defined as those whose failure could cause uncontrolled release of radioactivity or those essential for safe shutdown of the NSSS and long-term operation following a Loss of Coolant Accident. They are designed to withstand the appropriate seismic loads simultaneously with other applicable loads without loss of function. When a system, as a whole, is referred to as Class 1, portions not associated with loss of function of the system may be designated as Class 2 or 3, as appropriate.

1. Class 1 structures are listed on Table 5.2-2.

   Class 1 structures, except for the containment building and the auxiliary building addition, were designed for functional dependability following an earthquake by utilizing the load combinations described in Section 5.9.1. Load combinations for the containment buildings shell are found in Section 5.8.3. Load combinations for the containment interior structure are found in Section 5.9.2. Load combinations for the auxiliary building addition are found in Section 5.9.6. The design of equipment supports, rupture restraints, and jet impingement and missile barriers is discussed in Sections 5.9.1 and 5.9.2. The major design code utilized is also provided in Table 5.2-2. Class 1 structures are essentially equivalent to Seismic Category I structures as determined in SEP Topic III-1.

2. Class 1 systems and components are listed on Table 5.2-3.

   Class 1 systems and components were designed for functional dependability following an earthquake by using the load combinations in Section 5.10.1. Class 1 systems and components are always Seismic Category I equivalents in current design practice; however, they may be equivalent to ASME B&PV Class 1, 2 or 3. Class 1 systems could also be Safety Class 1, 2 or 3 per ANSI N18.2-1973. Table 5.2-3 identifies systems' classification and industrial design codes utilized.

3. Class 1 electrical equipment consisted of control boards, switchgear, load centers, batteries and cable runs serving Class 1 mechanical equipment and Class 1 electrical equipment supports.

5.2.2.2 Design - Class 2

Class 2 structures, systems and components are those whose limited damage would not prevent safe shutdown of the NSSS following the initiation of a trip or normal shutdown, whose failure may damage Class 1 components within the proximity, and whose failure would not cause uncontrolled release of radioactivity. They are designed to withstand the appropriate seismic load simultaneously with other applicable loads. Class 2 systems and components are listed on Table 5.2-3.

CPCo Design Class 2 systems were evaluated to the requirements of Subsection 5.10.2.1.

5.2.2.3 Design - Class 3

Class 3 structures, systems and components are those whose failure would not result in the release of radioactivity and would not prevent reactor shutdown but may interrupt power generation. This class includes all Plant structures, systems and components not listed as either Class 1 or Class 2.

Class 3 systems are shown on Table 5.2-3.

Class 3 systems are those systems not classified as Class 1 or 2. These systems, under current ASME terminology, would be considered nonsafety class, Quality Group D.

5.2.2.4 Design - Palisades Modifications

Design changes/modifications are performed pursuant to the Quality Program that is identified in Section 15.1. Administrative procedures provide specific instructions on performing work. These procedures require utilization of the "safety classifications" for modifications to safety-related systems and components under the QA program and identify the current classification status of each component. These procedures identify the person responsible for identifying the classification of equipment within a modification and delineate how documentation should be prepared. Equipment is to be "maintained" by the quality standards identified in the safety classifications; however, replacements can be made according to the original design code as allowed in administrative procedures.

The safety classifications were developed primarily as a tool to track the "quality maintenance" required on all Palisades mechanical, electrical and I&C components for safety-related equipment. Each component is identified as to its safety class, industrial code conformance (IEEE, ASME, etc) and other design attributes.

In addition, CPCo identifies system classifications per Regulatory Guide 1.26 and 10 CFR 50.55(a) via a procedurally controlled process.

Through parallel examination of the safety classifications and other plant documents, an overall picture of current system/component class can be obtained. Original code design/classification requires examination of the design/procurement specification.

5.2.2.5 Inservice Inspection

Upon the promulgation of 10 CFR 50.55a(g), all utilities with operating licenses were required to identify/classify systems and components pursuant to ASME Classes 1, 2 and 3 and thereafter inspect those systems/components to the ASME B&PV Code Section XI requirements. This was reflected in each licensee's ISI Program by reclassifying all mechanical components for inspection/repair according to the current ASME Code, regardless of design, although replacement to the original design code is permissible under certain circumstances.

Pursuant to 10 CFR 50.55a, Palisades has implemented an ISI Program as described in Section 6.9. Equipment classification within the ISI Program is compatible with the equipment safety classifications.

5.2.2.6 Service Quality Group Classification

Quality Group Classification applies to mechanical systems and mechanical components only, which are identified in Regulatory Guide 1.26, Revision 3, 1976. Table 5.2-3 shows the relationship between CPCo Class, Regulatory Guide 1.26 Class and ASME Class. ASME Section XI piping class boundaries are identified and maintained in accordance with 10 CFR 50.55(a) for Class 1 and Regulatory Guide 1.26 for Class 2 and 3.

5.2.2.7 Service - Electrical and Instrumentation and Controls Equipment Classification

Palisades construction was essentially complete when IEEE 279-1971, "Criteria for Protection Systems for Nuclear Power Generating Stations" and IEEE 308-1971, "Standard Criteria for Class 1E Electrical Systems for Nuclear Power Generating Stations" were issued. Although the 1980 FSAR Appendix A identified electrical equipment and its supports which were supporting Class 1 mechanical equipment as Class 1, a direct correlation to "Class 1E" was not readily identifiable. Subsequently, electrical equipment classification has been backfit to Palisades as a result of general industry upgrading, IE bulletins, NRC reviews, Systematic Evaluation Program, post TMI and fire protection upgrading following Brown's Ferry. The use of the term "Class 1E" in the FSAR and elsewhere is further discussed in Section 8.1. Chapters 7 and 8 have been extensively rewritten to reflect resultant equipment reclassification for both service and design.

"Environmentally Qualified" equipment was introduced in Memorandum and Order CLI-80-21 and formally adopted by 10 CFR 50.49. All "safety-related" electrical equipment exposed to harsh environments was to be either analyzed/approved for that service or replaced as defined in Regulatory Guide 1.89. Additionally, post-accident monitoring equipment as defined in Regulatory Guide 1.97 that supplies "information that is essential for the direct accomplishment of specified safety functions" following an accident are categorized as Type A instrumentation requiring enforcement of design/ service QA controls and environmental qualification. Refer to Chapter 8, Subsection 8.1.3 for details. Consumers has responded to both of these programs by establishing a comprehensive EEQ program. Tables 5.2-4 and 5.2-5 provide a listing of current "Class 1E" equipment at Palisades.

5.2.2.8 Safety-Related Classification

5.2.2.8.1 Safety-Related

Safety-related, as defined in 10CFR50.2, means those structures, systems and components that are relied upon to remain functional during and following design basis events to assure:

(1) The integrity of the reactor coolant pressure boundary

(2) The capability to shut down the reactor and maintain it in a safe shutdown condition; or

(3) The capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures set forth in 10CFR100.11.

5.2.2.8.2 Important to Safety

Important to safety, as used in 10CFR50 Appendix A to encompass the broad scope of equipment covered by the General Design Criteria (Reference 1), is defined as structures, systems and components that provide reasonable assurance that the facility can be operated without undue risk to the health and safety of the public. Important to Safety is also defined for specific applications in other sections of 10CFR50 and NRC Regulatory Guides. The use of these application specific definitions are identified in the applicable sections of the FSAR.

REFERENCES

1. Generic Letter 84-01, NRC Use of the Terms, "Important to Safety" and "Safety Related"

5.3 WIND AND TORNADO LOADINGS

5.3.1 WIND

5.3.1.1 Design Parameters

CP Co Design Class 1 structures were designed to withstand the external pressure resulting from a sustained wind velocity of 100 mi/h with a recurrence interval of 100 years. This velocity is considered to be conservative since the fastest mi/h wind, with a 100-year recurrence interval at the Palisades site, is 90 mi/h as determined from Figure 1(b) of Reference 1.

5.3.1.2 Forces on Structures

For all CP Co Design Class 1 structures, except the auxiliary building TSC/EER/HVAC addition, the forces due to wind were calculated in accordance with Reference 1. Applicable pressure and shape coefficients were used. The wind velocity was assumed not to vary with height and a gust factor was not used.

In 2003, Facility Change FC-976 upgraded the main hoist of the spent fuel crane in order to meet single failure criteria. The capacity of the main hoist was also increased to 110 tons. The steel frame structure over the Spent Fuel Pool in the Auxiliary Building was qualified for loads over 100 tons only when wind velocity is equal to or less than 90 mi/h.

For the auxiliary building TSC/EER/HVAC addition, the forces due to wind (see chart below) were calculated in accordance with Reference 2. The wind velocity was assumed to vary with height from the 100-year recurrence value of 100 mi/h at an elevation of 30 feet.

					Roof Suction Load (lb/ft^2) Roof Slope 0°20°	
		Dynamic	Wall Load (lb/ft^2)		Windward	Leeward
Height (ft)	Velocity (mi/h)	Pressure q (lb/ft^2)	Pressure (0.9 q)	Suction (0.5 q)	Slope (0.7 q)	Slope (0.6 q)
0-50	100	28	25	14	20	17
50-150	125	44	40	22	31	26
150-400	155	68	61	34	48	41
Over 400	185	96	86	48	67	58

NOTE: $q = 0.00256V^2 \times 1.1$

5.3.2 TORNADO

5.3.2.1 Design Parameters

The CP Co Design Class 1 enclosure over the spent fuel pool was not designed for tornado loads.

Other CP Co Design Class 1 structures, with the exception of the auxiliary building TSC/EER/HVAC addition, were designed to withstand the following tornado loads simultaneously:

1. External pressure resulting from a tornado funnel having a peripheral tangential velocity of 300 mi/h whose center is traveling at 60 mi/h

2. Differential pressure between the inside and the outside of enclosed areas of 3 psig (bursting)

The auxiliary building TSC/EER/HVAC addition was designed to withstand the following tornado loads simultaneously while remaining airtight (see Reference 2):

1. External pressure resulting from a tornado funnel having a maximum tangential wind velocity of 300 mi/h at a radius of maximum rotational speed of 150 feet, with its center moving at 60 mi/h

2. Differential pressure between the inside and the outside of enclosed areas of 3 psig (bursting)

3. Impact from missiles described in Subsection 5.5.1.1.4

Tornado missiles are discussed in Subsection 5.5.1.

5.3.2.2 Forces on Structures

The forces due to the 300 mi/h wind were calculated in accordance with Reference 1. Applicable pressure and shape coefficients were used. The tornado wind velocity was assumed not to vary with height. The resulting design pressures for CP Co Design Class 1 structures are shown in Table 5.3-1.

Blowout panels were not used as a design feature. Therefore, the structures were designed to resist the full differential pressure.

5.3.3 PLANT REEVALUATION

The capability of the CP Co Design Class 1 structures to resist the effects of tornado wind loads was evaluated in Topic III-2 of the Nuclear Regulatory Commission's (NRC) Systematic Evaluation Program (SEP) (Reference 3). This evaluation was performed prior to construction of the auxiliary building TSC/EER/HVAC addition. The CP Co Design Class 1 structures listed in Subsection 5.3.2 were determined to meet or exceed current NRC requirements. The following structures and components were determined not to have been designed to resist tornado wind loads:

1. Condensate storage tank

2. Intake and exhaust vents for the emergency diesel generators

3. Safety injection and refueling water (SIRW) tank

4. Steel framed enclosure over the spent fuel pool

However, the NRC concluded that any damage that might occur to these "unprotected" structures and components would not adversely affect the safe shutdown capability of the Plant.

REFERENCES

1. "Wind Forces on Structures," Task Committee on Wind Forces, ASCE Paper 3269, ASCE Transactions, Volume 126, Part II, Pages 1124-1198, 1961

2. Tornado and Extreme Wind Design Criteria for Nuclear Power Plants, Bechtel Topical Report, BC-TOP-3-A, Revision 3

3. NRC Systematic Evaluation Program Topic III-2, "Wind and Tornado Loadings," NUREG-0820, Section 4.6, October 1982.

5.4 WATER LEVEL DESIGN

The capability of the Consumers Design Class 1 structures to withstand the loads associated with the "probable maximum flood" and the "design basis flood," in accordance with NRC Standard Review Plans (SRP) 2.4.2, 2.4.3 and 2.4.5, was evaluated as part of the Nuclear Regulatory Commission's (NRC) Systematic Evaluation Program (SEP). The SEP also evaluated the protection of safety-related equipment from flooding due to these natural phenomena as well as flooding due to postulated failures of Nonclass 1 systems including the Circulating Water System and the Fire Protection System.

5.4.1 FLOODING FROM NATURAL SOURCES

5.4.1.1 Description of Events

The local "probable maximum flood" is based upon a "probable maximum precipitation" of 25.5 inches of rain in 6 hours. Rainfall was assumed to occur in the immediate Plant vicinity and the resulting runoff was determined to move overland toward Lake Michigan. It was assumed that one half of the peak runoff (555 ft^3/s) would pond on the east side of the service building to a depth of 5 feet (elevation 601 feet). For the remainder of the Plant area the depth would be less than 6 inches.

The "design basis flood" results from the addition of an on shore surge value (10.9 feet) to the maximum monthly mean lake level. The resulting flood level during SEP was determined to be 593.5 feet compared to a Plant grade of 589 feet along Lake Michigan. Subsequent to SEP, a new maximum monthly mean lake level for the period 1900 to present was recorded in October of 1986. This level of 583.2 feet MSL with the added on shore surge height of 10.9 feet yields a new "design basis flood" level of 594.1 feet. For additional detail see References 1, 2 and 5.

5.4.1.2 Effects on Consumers Design Class 1 Structures and Safety-Related Equipment

The NRC concluded that the design approach used for the Consumers Design Class 1 structures ensures that these structures are more than adequate for resisting the effects of the "design basis flood." Therefore, they are also adequate for the "probable maximum flood." In addition, it was concluded that the intake structure could withstand the dynamic effect of wave run-up of approximately 8 feet. Since the service building is not a Consumers Design Class 1 structure, the effect of the ponded water on its integrity was not evaluated.

The plant was originally evaluated for protection against flooding and determined to be able to safely shutdown for a flood level up to 594 feet 8 inches with the limiting component being the Service Water Pump Motor. Barriers of various types including marine-type watertight doors are utilized to protect safety-related equipment below this level (see Reference 6). Water that might enter the service building would not be deep enough to reach the auxiliary building addition via the doorway at elevation 607. Hence, the NRC concluded that safety-related equipment at Palisades is adequately protected from the potential flooding from natural sources. However, the basis for the Service Water Pump Motor windings being the limiting safety related component could not be verified to include consideration of potentially flooding the motor lower bearing lube oil reservoir at approximately elevation 594 feet 5½ inches. Therefore, the plant will be considered to be protected against a flood level up to an elevation of 594.4 feet.

5.4.2 FLOODING AND WETTING FROM PLANT SOURCES

SEP Topic VI-7.D considered the effects of flooding on safety-related equipment required for safe shutdown or accident mitigation due to postulated failures in Nonclass 1 systems. Ten types of equipment were determined to require protection. This equipment is listed in Table 5.4-1 along with its general location. The NRC's review of this subject considered the potential for flooding from both within and without the equipment compartments. Postulated failures in the Circulating Water System and the Fire Protection System represented the controlling cases. The NRC concluded that the auxiliary feedwater pumps and the diesel generators were inadequately protected. The evaluation of flooding in the auxiliary feedwater pump room concluded that the pumps were vulnerable to flooding from internal sources (see Reference 3). The subsequent addition of a third auxiliary feedwater pump in the west engineered safeguards room provides redundancy and satisfactorily resolves this issue. The evaluation of flooding in the diesel generator rooms concluded that the generators were vulnerable to flooding caused by a break in the fire protection line or other water carrying piping that passes through these rooms. However, additional study by Consumers indicated that at no time would the operability of both of the Diesel Generators be impaired due to flooding from plant sources (see Reference 4).

The remaining eight types of equipment listed in Table 5.4-1 were considered to be adequately protected.

The issue of protection of the service water pumps from postulated pipe breaks was reevaluated under the auspices of SEP Topic IX-3. This time, the NRC concluded that the pumps were vulnerable to both flooding and wetting from piping located within the intake structure. As a result of this finding modifications were made. Specifically, a 26-inch solid manhole cover was replaced with a perforated cover and deflector shields were installed over the service water pump motors. These actions satisfactorily resolved the NRC's concerns.

The rupture of the SIRW tank was evaluated separately. It was concluded that rupture of this tank will have no effect on other safety-related equipment. Flooding of the component cooling water pumps was also evaluated separately. It was concluded that they were adequately protected from flooding up to elevation 594 feet 8 inches.

Flooding as a result of postulated breaks in the Plant's steam heating system is discussed in Subsection 5.6.7.1(3). Flooding of the engineered safeguards rooms from postulated breaks in the service water system discharge piping is discussed in Subsection 9.1.3.3.

REFERENCES

1. "High Water Level Study, Palisades Plant, Lake Michigan," by R M Noble & Associates, dated March 12, 1982

2. Letter from T V Wambach (NRC) to D J VandeWalle (CPCo) dated October 7, 1982

3. Letter from DMCrutchfield (NRC) to DPHoffman (CPCo), "SEP Topic VI-7.D, Long Term Cooling - Passive Failures," dated April 30, 1981

4. Letter from GBSlade (CPCo) to NRC, "Generic Multi-Plant Issue B-11, 'Susceptibility of Safety Related Systems to Flooding Caused by the Failure of Non-Category I Systems,'" dated November 19, 1992

5. EA-FC-864-46, Appendix A, Attachment 5, "National Oceanic and Atmoshpheric Administration Report on Lake Michigan Lake Levels"

6. FSAR Amendment 18, Question 2.4

5.5 MISSILE PROTECTION

5.5.1 TORNADO MISSILES

5.5.1.1 Design Parameters

5.5.1.1.1 Containment Structure, Auxiliary Building, Turbine Building

The containment structure and the Consumers Design Class 1 portions of the auxiliary and turbine buildings were analyzed for the effects of the missiles shown below using Reference 1. The missile velocities were obtained based upon a uniform wind of 600 mi/h of infinite extent and an acceleration distance of 800 feet, which is the farthest extent of the graded Plant area from the Plant buildings. The sand dunes surrounding the Plant area on three sides will break up uniform winds from greater distances and it is not credible that a missile could originate from the lake.

Design Tornado Missiles

Item	Weight (lb)	Velocity (ft/s)	Impact Area End of Item (ft^2)
Pipe (3 in diam x 10 ft long)	76	620	0.095
Wood Plank (4 in x 12 in x 12 ft long)	104	760	0.29
Automobile	4,000	560	31.5
Flatcar	40,000	480	16
Boxcar	47,500	500	-
Locomotive	240,000	310	120

The equipment hatch was evaluated for the missile shown below. Due to the structural configuration around the hatch, this was determined to be the only credible missile capable of striking the hatch.

Item	Weight (lbs)	Velocity (ft/s)
Steel Rod (1" diam. x 3 ft. long)	8	317

5.5.1.1.2 Intake Structure

The Consumers Design Class 1 portion of the intake structure was analyzed for the effects of the missiles shown below using Reference 1. The missile velocities are one-half the values shown above.

Design Tornado Missiles

Item	Weight (lb)	Velocity (ft/s)	Impact Area End of Item (ft^2)
Pipe (3 in diam x 10 ft long)	76	310	0.095
Wood Plank (4 in x 12 in x 12 ft long)	104	380	0.29
Automobile	4,000	280	31.5

5.5.1.1.3 Auxiliary Building Radwaste Addition

This structure was analyzed for the effects of the missiles shown below using Reference 1. The missile velocities were based on a tornado with a maximum wind speed of 360 mi/h. The automobile was considered to fly no more than 25 feet off the ground.

Design Tornado Missiles

Item	Weight (lb)	Velocity (ft/s)	Impact Area End of Item (ft^2)
Wood Plank (4 in x 12 in x 12 ft long)	104	440	0.29
Automobile	4,000	74	31.5

5.5.1.1.4 Auxiliary Building TSC/EER/HVAC Addition

These structures were analyzed for the effects of the missiles shown below using References 2 and 3. The missile velocities were based upon a tornado wind with a maximum wind speed of 360 mi/h. Neither the automobile nor the utility pole need be considered for any portion of the structures which is 30 feet above the highest ground elevation within a one-half mile radius of the buildings.

Design Tornado Missiles

Item	Weight (lb)	Velocity (ft/s)	Impact Area End of Item (ft^2)
Steel Rod (1 in diam x 3 ft long)	8	317	0.005
Wood Plank (4 in x 12 in x 12 ft long)	108	440	0.29
Utility Pole (13-1/2 in diam x 35 ft long)	1,490	211	0.99
Automobile	4,000	106	20

5.5.1.1.5 Diesel Fuel Oil Storage Tank Housing

The analysis of this structure, which is located mostly below grade with a roof slab two feet above grade, considered the effects of the missiles shown below per Reference 13. Reference 14 was used in the analysis. Since the structure is essentially buried except for the roof slab, the missile velocities used are those applicable to vertical impact.

Design Tornado Missiles

Item	Weight (lb)	Velocity (ft/s)	Impact Area End of Item (ft^2)
Wood Plank	114.7	190.5	0.29
6" sch. 40 pipe	286.7	119.4	0.24
12" sch. 40 pipe	749.7	107.9	0.89
1" Steel Rod	8.82	167.3	0.0054
Utility Pole	1124.6	126.3	0.99
Automobile	3991.1	135.5	27.97

5.5.1.1.6 Condensate Storage Tank

The condensate storage tank provides water to the auxiliary feedwater system for decay heat removal during normal and accident conditions. As the condensate storage tank supply is depleted, backup water supplies can be aligned to the auxiliary feedwater system. A tornado missile strike near the bottom of the condensate storage tank could cause a loss of tank inventory before backup supplies can be aligned, resulting in auxiliary feedwater pump damage due to air ingestion. To prevent this, the lower

portions of the condensate storage tank are protected from tornado missiles by a concrete block barrier on the west side of the condensate storage tank (References 7 and 19). Adjacent plant components and structures along the other sides of the tank also provide protection. This protection provides assurance that a tornado missile strike could not empty the tank and damage the auxiliary feedwater pumps before a back-up water supply can be aligned.

Design Tornado Missiles (Ref. 19)

Item	Weight (lb)	Velocity (ft/s)
6" sch. 40 pipe	287	135
1" Steel Sphere	0.147	26
Automobile	4000	135

An additional steel plate barrier is erected for beyond-design-basis events around the entire perimeter of the tank that extends to the height of the tank (Reference 21).

5.5.1.2 Structural Considerations

The wall and roof thicknesses of Consumers Design Class 1 structures are shown in Table 5.5-1.

The only design missile that could both reach and penetrate the wall of the containment structure is the railroad flatcar. Some of the wires in 10 to 12 of the tendons would rupture at the point of impact but total tendon failure would not occur. The containment structure would not collapse, the liner plate would remain intact and the NSSS would not be affected.

The only design missile that could both reach and perforate the walls of the auxiliary building which house the control room, switchgear and diesel generators is the 3-inch diameter pipe. This missile will not disrupt the operation of safety-related systems because of the existence of redundant equipment.

The only design missile that could both reach and perforate the Class 1 portions of the turbine building and intake structure is the 3-inch diameter pipe. This missile will not disrupt the operation of safety-related systems because of the existence of redundant equipment.

5.5.1.3 Plant Reevaluation

5.5.1.3.1 Review Parameters

The capability of the Consumers Design Class 1 structures to resist the effects of tornado missiles was evaluated in Topic III-4.A of the Nuclear Regulatory Commission's (NRC) Systematic Evaluation Program (SEP). Additionally, nine Consumers Design Class 1 systems identified as "safe shutdown systems" by the SEP program were evaluated (Reference 4).

These evaluations were performed prior to construction of the auxiliary building TSC/EER/HVAC addition.

In accordance with Standard Review Plan (SRP) 3.3.2 (Reference 5), the following missiles were considered:

Review Tornado Missiles

Item	Weight (lb)	Horizontal Speed (ft/s)	Vertical Speed (ft/s)
Steel Rod (1 in diam x 3 ft long)	8	317	253
Utility Pole (13-1/2 in diam x 35 ft long)	1,490	211	169

The missile speeds shown above were based on a tornado with a maximum wind speed of 360 mi/h. These missiles were considered to be capable of striking in all directions with vertical speeds equal to 80% of the horizontal speeds.

5.5.1.3.2 Summary

The NRC concluded that the gross structural response of Consumers Design Class 1 structures to the two review missiles was enveloped by their response to the five design missiles.

The NRC review of the nine Consumers Design Class 1 systems determined that the following safety-related equipment was vulnerable to tornado missiles:

1. Condensate storage tank
2. Intake and exhaust vents for the emergency diesel generators
3. Safety injection and refueling water (SIRW) tank
4. Vent stacks for the atmospheric dump valves
5. Vent stacks for the main steam safety relief valves

In addition, the steel frame enclosure over the spent fuel pool, the emergency personnel access enclosure and the compressed air system supply for various Consumers Design Class 1 system valves were determined to be vulnerable to tornado missiles. The NRC concluded that any damage that might occur to the above-mentioned structures, systems and components would not adversely affect the safe shutdown capability of the Plant.

5.5.2 TURBINE MISSILES

5.5.2.1 Background

During the design effort for the steam and power conversion system, both the high-pressure turbine and the low-pressure turbine were reviewed for their potential to generate missiles. It was concluded that the state-of-the-art of rotor forgings and inspection techniques guaranteed, for all practical purpose, defect-free turbine rotors. Furthermore, it was concluded that Westinghouse conservative design practices would eliminate any harmful stress concentration points. These conclusions were substantiated by the record of Westinghouse turbine generators; no unit, to that date, had ever experienced a rotor or disk failure. Therefore, the only potential cause of a turbine generator failure, hence missile generation, would be excessive overspeed. The overspeed protection system is described in Section 7.5.

Since the design of this system, new information concerning steam turbines has been generated. Based on this information, the industry and the NRC have now identified potential low-pressure turbine missiles which can be generated during normal operating conditions.

5.5.2.2 High-Pressure Turbine Missiles

The original and current assessment of the missile generating capability of the high-pressure turbine is discussed below.

Due to the very large margin between the high-pressure turbine spindle bursting speed and the maximum speed at which the steam can drive the unit with all the admission valves fully open, the probability of spindle failure is practically zero. Therefore, no harmful missile is anticipated in case of turbine runaway.

Based on the admission steam thermodynamic properties and blade geometry, the maximum theoretical speed which the unit could reach is 205% of nominal.

Based on the stress analysis of the low-pressure turbine disks, the maximum actual speed at which the unit may run without failure is 180% of nominal.

The minimum bursting speed of the high-pressure turbine spindle, based on the minimum specified mechanical properties of the spindle material, is 260% of nominal.

Hence, the actual margin between the high-pressure turbine spindle bursting speed and the low-pressure turbine maximum running speed is of the order of 80% of nominal; ie, 260% less 180%.

No failure of the high-pressure turbine is anticipated as a consequence of a unit runaway and, therefore, no missiles are expected to be generated.

5.5.2.3 Low-Pressure Turbine Missiles

The low-pressure (LP) turbines are of a double flow design. Each element consists of a double flow rotor assembly, an outer cylinder, an inner cylinder, blade rings, and blade carriers. The rotor assembly consists of a shaft with 8 shrunk-on disks made of low alloy steel and two shrunk-on couplings. Steam enters at the top of each outer cylinder where it flows to the inlet chamber of the inner cylinders. In the inlet chamber, the steam is distributed equally to both halves of the rotor and flows through the blading to the condenser. Disks are numbered from the inlet chamber outward in two symmetric sets from 1 to 4.

The low pressure turbine rotors, inner cylinders, and stationary components were replaced per Reference 16. The replacement components were originally installed at the Connecticut Yankee Haddam Neck Nuclear Plant. These components were removed from the plant, inspected, and refurbished by Siemens Westinghouse. The inspections did not reveal any indications in the keyway or bore regions of the disks.

The NRC requires that the probability of missile generation for a turbine with an unfavorable orientation, such as that at Palisades, be less than 1E-05 per year. The probability of missile generation is determined by the product of the probabilities of a disk burst and a casing penetration.

Siemens Westinghouse performed a disk burst and casing penetration analysis of the rebuilt low pressure turbine (Reference 17). This analysis determines disk burst and casing penetration probabilities at speeds up to 120% of rated speed.

Table 5.5-2 provides the calculated disk burst probabilities for the individual rotors and for the entire unit. These probabilities were calculated assuming a 3 mm default bore flaw. Siemens Westinghouse recommends that disk burst probability should not exceed 1E-04 per year. Based on the unit operating 8000 hours per year, this table indicates that the disk burst probabilities for the individual rotors and the entire unit will be 6.50E-05, 2.54E-05, and 9.04E-05, respectively, in the fall of 2014, or after 120,000 operating hours.

The Siemens Westinghouse analysis determined that casing penetration probability is less than 1E-06 at 180% of rated speed. This low probability is due to the inner cylinders that were installed under Reference 16. These inner cylinders incorporated "crash rings" which are designed to prevent external missiles at speeds up to 120% of rated speed.

Based on the low probabilities of a disk burst and a casing penetration as determined in the Siemens Westinghouse analysis, the probability of a missile generated by the low pressure turbine is extremely low and well below the NRC requirement of 1E-05.

5.5.2.4 Turbine Overspeed Protection System

Inspection of the overspeed protection system, as described below, provides reasonable assurance that the turbine will never operate at a speed high enough to cause a rapid, massive failure of the turbine disks.

At every turbine overhaul and at each refueling outage, the following two tests are performed:

1. The turbine is intentionally overspeeded to the trip set point in order to close the stop and governor valves.

2. As the turbine is brought up to operating speed, the stop and governor valves are tested.

Also, on a periodic basis, while the Plant is operating, each of the four main stop and governor valve combinations is exercised. The frequency at which the turbine valves are tested is determined to ensure that the probability of a turbine missile ejection is within the NRC criteria established in Reference 10. The probability of turbine missile ejection is within the NRC criteria with a turbine valve testing interval of 18 months (Reference 20).

5.5.3 INTERNALLY GENERATED MISSILES

5.5.3.1 Containment Missiles

High-pressure Primary Coolant System components, which could become a source of missiles, were reviewed. The following types of missiles were identified for Loss of Coolant Accidents with break sizes up to and including a double-ended guillotine break of the main coolant pipe: valve stems, valve bonnets, instrument thimbles and various types and sizes of nuts and bolts. For analytical purposes, these missiles were classified according to size, shape and kinetic energy. Their velocities were calculated considering both fluid and mechanical driving forces which could act during missile generation.

Missiles which could only be generated by a massive, rapid failure of the Primary Coolant System components (reactor vessel, steam generator, pressurizer, primary coolant pump casings and driver) were not considered credible. This position was justified for each component based upon material characteristics, inspections, quality control during fabrication and a conservative design.

Missile protection is provided to comply with the following criteria:

The containment liner plate and the engineered safeguards systems shall be protected from loss of function due to damage inflicted by the missiles identified in this subsection.

Protection against loss of function is provided in the form of missile barriers. These barriers were designed for both static and impact loads using state-of-the-art missile penetration data.

High-pressure Primary Coolant System components, which are the source of the missiles identified in this subsection, are suitably screened from the structures and systems mentioned above by one or more of the following missile barriers:

1. Reinforced concrete primary shield wall which surrounds the reactor vessel.

2. Reinforced concrete steam generator compartment walls which enclose the primary coolant loops.

3. Reinforced concrete operating floor.

4. Special reinforced concrete barriers. (Example: Removable slab over the control rod drive mechanisms to block any missiles generated by a fracture of these mechanisms.)

5.5.3.2 <u>Plant Reevaluation</u>

The issue of protection from internally generated missiles was addressed in Topic III-4.C of the Nuclear Regulatory Commission's (NRC) Systematic Evaluation Program (SEP) (Reference 6). Twenty-seven systems were identified as needed to perform safety functions (safe Plant shutdown or accident mitigation) or whose failure could result in a significant release of radioactivity. These systems were located both inside and outside the containment building.

In general, sources of potential missiles were identified during a walk down of the 27 systems. Next, these sources were evaluated to determine the likelihood that they would generate missiles such as valve bonnets, hardware retaining bolts, relief valve parts, instrumentation wells and pump impellers. Finally, the effect of the generated missiles on the identified systems was evaluated. Based upon these evaluations, the NRC concluded that the 27 systems were adequately protected from missiles such that any damage they might incur would not affect the Plant's overall ability to perform safety functions or would not release a significant amount of radioactivity.

5.5.4 SITE PROXIMITY MISSILES

The capability of Palisades' safety-related structures, systems and components to resist the effects of site proximity missiles was evaluated in Topic III-4.D of the Nuclear Regulatory Commission's (NRC) Systematic Evaluation Program (SEP).

The potential for hazardous activities in the vicinity of the Palisades Plant has been addressed in Subsection 2.1.3, Nearby Industrial, Transportation and Military Facilities. As indicated therein, little industrial activity is located near the Plant. Transportation facilities, including highways, railroads, pipelines and lake shipping lanes, are sufficiently far away so as not to present a credible missile hazard. No military facilities or military activities that could create a missile hazard exist near the Plant.

Aircraft operation is the only activity in the vicinity of the Palisades Plant that presents a potential missile hazard. South Haven Regional Airport is a general aviation facility located about three miles from the Plant. Other airports in the area will not have a significant effect on the safety of the Plant due to the nature of their operations and their distance from the Plant.

South Haven Regional Airport is used primarily by light single engine aircraft engaged in general aviation activities such as business and pleasure flying and agricultural spraying operations. The facility includes one paved runway and one turf runway. The paved runway, designated 4-22 and oriented in a northeast-southwest direction, is 4,300 feet long and 75 feet wide. The airport currently experiences 22,000 operations per year.

Airport activities will be monitored to determine if substantial increases in the annual number of operations or substantial use by heavier aircraft is imminent, since such changes will require a reassessment of the risk of an aircraft accident at the Plant and the damage it could inflict.

Nuclear power plant structures that are designed to withstand tornado missile loads simultaneously with other design loads can withstand collision forces imposed by light general aviation aircraft without adverse consequences. However, safety-related equipment located outside such protective structures is vulnerable to a light airplane crash. The overall probability of a light aircraft striking such equipment at the Palisades Plant is within the acceptance criteria of SRP 2.2.3, "Evaluation of Potential Accidents" (Reference 9), based on calculations employing the analytical model given in NRC Standard Review Plan (SRP) 3.5.1.6, "Aircraft Hazards" (Reference 8). Conservative assumptions used in the calculation include the following:

1. All operations at the South Haven Airport, an operation being either a takeoff or landing, involve aircraft which pass over the Plant area.

2. All relevant Plant targets are considered vulnerable to aircraft crashes from any direction, even when these targets are shadowed by other Plant buildings.

Five pieces of safety-related equipment at the Palisades Plant are potentially vulnerable to light aircraft impacts. All five either have a backup available or are not required to achieve a safe shutdown under normal Plant operations. The affected equipment is:

1. Atmospheric dump valves (via incapacitation of roof vents)

2. Condensate storage tank

3. Diesel generators (via incapacitation of intake and exhaust vents)

4. Safety injection and refueling water (SIRW) tank

5. Station transformers

The combined probability of an aircraft disabling one of these pieces of equipment and the simultaneous loss of normal Plant operations leading to a demand for that equipment is well within the acceptance criteria of SRP 2.2.3.

The spent fuel pool is also vulnerable. It is covered by a structural steel framework finished with thin metal panels. Assuming that this cover is not present, the probability of a light aircraft striking the pool is about 2.5×10^{-8} per year. However, the structural steel framework will provide substantial resistance to aircraft impacts. Therefore, the probability of an aircraft entering the spent fuel pool and damaging a sufficient number of fuel assemblies such that 10 CFR Part 100 dose guidelines would be exceeded is very low, well within the acceptance criteria of SRP 2.2.3.

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3. "Evaluation of Tornado Missile Impact Effects on Structures," J V Rotz; presented at the Tornado Symposium entitled "Assessment of Knowledge and Implications for Man," June 1976

4. NRC Systematic Evaluation Program, Topic III-4.A, "Tornado Missiles," NUREG-0820, Section 4.8, October 1982

5. NRC Standard Review Plan, NUREG-0800, Section 3.3.2, "Tornado Loadings," Revision 2, July 1981

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11. "Probabilistic Evaluation of Reduction in Turbine Valve Test Frequency" Westinghouse document WCAP-11525

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14. ASCE Publication on Impactive and Impulsive Loads, 2nd ASCE Conference, Vol. V, Knoxville, Tennessee, September, 1980

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17. Siemens Westinghouse Report, "Analysis of LP Disk Burst and Casing Penetration Probabilities," dated July 15, 1999

18. Engineering Assistance Request EAR-2000-0013, “Replacement of the High Pressure Turbine Rotor”

19. Calculation EA-EC7237-01, “Missile Barrier Calculation for Enhanced Protection of the Condensate Storage Tank (CST) [T-2].”

20. PLP-RPT-10-00060, “Palisades Turbine Overspeed Protection System Testing Frequency,” Rev. 0.

21. EA-EC8187-03, "FLEX Condensate Storage Tank (CST) T-2 and Primary Makeup Storage Tank T-81 Tornado Missile Shield Shell Design", Rev. 0.

5.6 DYNAMIC EFFECTS OF PIPE RUPTURE

5.6.1 DEFINITIONS

The following definitions are applicable to the high and moderate energy pipe break analyses:

1. High-energy systems are those systems where either of the following conditions are met during normal Plant conditions:

    a. The maximum normal temperature exceeds 200°F.

    b. The maximum normal pressure exceeds 275 psig.

2. Moderate-energy systems are those fluid systems which are "high energy" less than 2% of the time the system is in operation or are "high energy" less than 1% of the total Plant operating time, and piping systems which are pressurized above atmospheric pressure during normal plant conditions, but which are not identified as high-energy piping.

3. Circumferential break refers to a complete severance of a high-energy pipe, perpendicular to the pipe axis, resulting in an instantaneous release of mechanical internal pipe forces across the break. The resulting dynamic forces are assumed to separate the piping axially with at least a one diameter lateral displacement of the ruptured piping sections, unless the movement is limited by piping restraints, structural members or piping stiffness. The effective cross-sectional flow area of the pipe is used in the jet discharge evaluation.

4. Longitudinal break refers to a break postulated to occur in a high-energy pipe wall parallel to the pipe axis without pipe severance and oriented at any point around the pipe circumference. The longitudinal break is postulated to be a rectangular slot having a length of two inside pipe diameters and a cross-sectional area equivalent to the internal cross-sectional area of pipe upstream of the break location.

5. Critical crack refers to a through-wall opening in a moderate-energy pipe assumed as a circular orifice of cross-sectional piping flow area equal to one-half the pipe inside diameter times one-half the pipe wall thickness. The crack may occur at any orientation about the circumference of the pipe.

6. Terminal ends are extremities of piping runs that connect to structures, components, or anchors that act as essentially rigid constraints to piping rotation and translation; eg, connections to vessels, pumps and fittings that are rigidly anchored to structures. In piping systems which are pressurized during normal Plant conditions for only a portion of the run (ie, up to the first normally closed valve) a terminal end of such runs is the piping connection to this closed valve.

7. Piping runs, defined as the piping between terminal ends, traverse all branch connections on complex piping systems.

8. S_h is the allowable stress at maximum (hot) temperature as defined in Article NC-3600 of ASME Code Section III for nuclear Class 2 and 3 piping.

9. S_A is the allowable stress range for expansion stresses as defined in Article NC-3600 of ASME Code Section III for nuclear Class 2 and 3 piping.

10. S_m is the design stress intensity as defined in Article NB-3600 of ASME Code Section III for nuclear Class 1 piping.

5.6.2 DESIGN BASES

5.6.2.1 <u>Systems in Which Design Basis Failures Occur</u>

Plant piping systems or portions of systems that are pressurized above atmospheric pressure during normal Plant conditions are classified as either high- or moderate-energy piping. For piping interaction and Plant analysis, a single, nonmechanistic piping failure is postulated to occur in any piping system or portion thereof that contains high- or moderate-energy fluid during normal Plant conditions; ie, start-up, operation in the design power range and shutdown.

5.6.2.2 <u>Identification of Essential Systems and Components</u>

The systems and components necessary to achieve and maintain cold shutdown conditions and to mitigate the consequences of a postulated piping failure, without offsite power are as follows:

1. Auxiliary Feedwater

2. Chemical and Volume Control (charging path only)

3. Component Cooling

4. Containment Air Cooling Units

5. Containment Isolation Valves

6. Containment Liner and Penetrations

7. Containment Spray

8. Critical Service Water

9. Emergency Core Cooling

10. Emergency Power System

11. HP Safety Injection

12. Main Feedwater (main FW check valves to steam generator only)

13. Main Steam

14. Pressurizer

15. Primary System Charging

16. Shutdown Cooling

17. Steam Generator Blowdown (steam generator to containment isolation only)

These systems and components have been protected as necessary against pipe whip, jet impingement and environmental effects for high-energy pipe breaks. The components and system are similarly protected against water spray, flooding and environmental conditions for moderate-energy pipe breaks.

5.6.2.3 Limiting Conditions

The failure of piping containing high-energy fluid may lead to damage of surrounding systems, structures and equipment. The effects of such a postulated failure including pipe whip, fluid jet impingement, steam and/or water flooding, compartment pressurization and environmental effects have been analyzed to assure the following:

1. The ability to safely shut down the reactor and maintain it in a safe shutdown condition.

2. A high-energy pipe break which does not constitute a loss of reactor coolant must not cause an unisolable loss of reactor coolant.

3. A steam or feedwater line break must not cause a reactor coolant system pipe break or vice versa (ie, there shall be no simultaneous primary system and secondary system loss of fluid pressure boundary).

4. A main steam line, auxiliary steam line, main feedwater line, or auxiliary feedwater line break in one steam generator system must not cause a main steam line, auxiliary steam line, main feedwater line, or auxiliary feedwater line break in the other steam generator system.

5. Seismic Category I structures necessary to safely shut down the reactor and maintain it in a safe shutdown condition or to mitigate the consequences of postulated piping failures must be adequately designed to withstand (or be protected against) pipe whip and environmental effects of the rupture of high-energy piping located in the near vicinity.

6. Resultant doses are below the guideline values of 10 CFR 100.

5.6.2.4 Safety Evaluation

In order to meet the basic design criteria for pipe ruptures, the following assumptions are made:

1. Circumferential breaks are considered in high-energy piping runs exceeding a nominal pipe size of 1 inch.

2. Longitudinal breaks are considered in high-energy piping runs and branch runs with a nominal pipe size of 4 inches or larger.

3. A single active component failure may occur anywhere within the combined systems required to achieve cold shutdown following a postulated high-energy pipe break. No passive failures will be postulated to occur simultaneously during or following a postulated high-energy pipe break, except where a secondary pipe break or other passive failure is directly caused by pipe whip or fluid jet impingement resulting from the initial pipe break.

4. No DBA or seismic occurrence is assumed to occur concurrently with a non-LOCA pipe break. All structures, systems and components are available for use unless their function has been impaired by the postulated break or electric power is unavailable.

5. Only Seismic Category I safety-related equipment is assumed operable during the DBA event.

6. For any break which results in a reactor trip or turbine trip, offsite power will be assumed to be unavailable.

7. Operating conditions prior to the pipe break are considered normal steady-state, anticipated normal transients, or hot standby.

5.6.3 CRITERIA USED TO DEFINE BREAKS

5.6.3.1 ASME Section III, Class 1 Piping

For ASME Section III Codes Class 1 piping, pipe breaks are postulated to occur at terminal ends and at all intermediate locations throughout a piping system where the following criteria are met:

1. The stress intensity range, S, as calculated by Equation 10 of Paragraph NB-3653 exceeds 2.4 S_m, or

2. The cumulative usage factor, U, exceeds 0.1.

where

S = primary-plus-secondary stress-intensity range associated with the normal and upset Plant condition loadings, as calculated from Equations 10, 12 and 13 in Subarticle NB-3600 of the ASME Boiler and Pressure Vessel Code, Section III.

U = cumulative usage factor, as calculated in accordance with Subarticle NB-3600.

Where the stresses calculated for a piping run between terminal ends are everywhere less than the stress limits stated above, a minimum of two break locations are postulated based on highest stress. Generic Letter 87-11 may be utilized to eliminate the requirement for postulating intermediate breaks for locations where all piping stresses between terminal ends are less than the stress limits stated above. Where Generic Letter 87-11 is utilized, Table 5.6-4 must be updated to include any piping or systems which have used these relaxed criteria. When the piping system is modeled and analyzed as a whole and the stresses are maintained below the pipe break allowables as presented above, breaks are not postulated at the branch connections unless they are one of the two highest intermediate stress points in the piping run.

At each postulated break location, circumferential breaks are assumed to occur in pipes larger than 1 inch and longitudinal breaks are assumed to occur in pipes 4 inches and larger except:

1. Longitudinal breaks are not postulated at terminal ends if the pipe does not have a longitudinal weld at that location.

2. Longitudinal breaks are not postulated where the criterion for a minimum number of breaks must be satisfied. For seamed pipe, longitudinal breaks are oriented along the pipe seam. In instances where the seam orientation is not known, slot breaks at any point around the pipe are assumed.

5.6.3.2 ASME Section III Class 2 and 3 Piping (other than between containment isolation valves)

Breaks are postulated to occur at the following locations in ASME Section III Class 2 and 3 piping:

1. At the terminal ends of the pressurized portions of the run.

2. At intermediate locations selected by either one of the following methods:

   a. At all locations where the stress, S, exceeds 0.8 ($1.2S_h + S_A$)

   where

   S = stresses under the combination of loadings associated with the normal and upset Plant condition loadings, as calculated from the sum of Equations 9 and 10 in Subarticle NC-3600 of the ASME Boiler and Pressure Vessel Code, Section III.

   b. At each location of potential high stress, such as pipe fittings (elbows, tees, reducers, etc), valves, flanges and welded attachments.

3. If there are not at least two intermediate locations where the stress, S, exceeds 0.8 ($1.2S_h + S_A$), a minimum of two locations is chosen based on highest stress. Generic Letter 87-11 may be utilized to eliminate the requirement for postulating intermediate breaks for locations where all piping stresses between terminal ends are less than the stress limits stated above. Where Generic Letter 87-11 is utilized, Table 5.6-4 shall be updated to include any piping or systems which have used these relaxed criteria. When the piping system is modeled and analyzed as a whole and the stresses are maintained below the pipe break allowables as presented above, breaks are not postulated at the branch connections unless they are one of the two highest intermediate stress points in the piping run.

At each postulated break location, circumferential breaks are assumed to occur in pipes larger than 1 inch and longitudinal breaks are assumed to occur in pipes 4 inches and larger except:

a. Longitudinal breaks are not postulated at terminal ends if the pipe does not have a longitudinal weld at that location.

b. Longitudinal breaks are not postulated where the criterion for a minimum number of breaks must be satisfied.

For piping systems, or portions of systems, within enclosures, breaks are postulated in accordance with Items 1, 2 and 3 above, and it is demonstrated by analysis that such enclosure is adequately designed to prevent any damage to essential structures and equipment from the effects of pipe whip, jet impingement, pressurization of the enclosure compartment, environmental conditions and flooding associated with the escape of the contained fluid. Piping restraints within the enclosure may be accounted for in limiting the effects of the postulated pipe rupture.

5.6.3.3 Non-nuclear Class Piping

Breaks are postulated to occur at the locations as specified for ASME Section III Class 2 and 3 piping, if the nonnuclear piping is analyzed, hung and supported to withstand the full SSE loadings. For non-nuclear class piping systems where stress information is not available, breaks are postulated to occur at the following locations:

1. The terminal ends.

2. Each intermediate location of potential high stress or fatigue, such as pipe fittings (elbows, tees, reducers, etc), valves, flanges and welded attachments.

5.6.3.4 Piping Penetrating Containment

Pipe breaks are not postulated in portions of high-energy piping extending from the containment penetration to the first piping restraint beyond the first isolation valve outside containment, providing the following requirements are met:

1. The following design stress and fatigue limits are not exceeded:

   For ASME Code Section III Class 1 Piping

   There is no ASME Section III Class 1 piping penetrating the containment; therefore, no break criteria are provided.

   For ASME Code Section III Class 2 Piping

   The maximum stress ranges, as calculated by Equations 9 and 10 in Paragraph NC-3652, ASME Code Section III, considering normal and upset Plant conditions do not exceed 0.8 ($1.2S_h + S_A$).

2. The piping is anchored reasonably close to the first isolation valve such that occurrence of a pipe break outside containment beyond these restraints shall neither impair operability of the valves nor the integrity of the containment penetration. Terminal ends of the piping runs extending beyond these portions of high-energy piping are considered to originate at a point adjacent to these anchors.

3. Welded pipe support attachments to those portions of piping penetrating containment are avoided to eliminate stress concentrations.

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

5. The extent of piping run is reduced to the minimum length practicable.

6. The design at points of pipe fixity (eg, pipe anchors or welded connections at containment penetrations) does not require welding directly to the outer surface of the piping (eg, fluid integrally forged pipe fittings are acceptable designs).

5.6.4 PROTECTIVE MEASURES

Protective measures employed are separation, protective enclosures and piping restraints.

1. Separation - The Plant arrangement provides separation to the extent practicable between redundant safety systems in order to prevent loss of safety function as a result of hazards different from those for which a system is required to function as well as for the specific event for which the system is required to be functional. Separation between redundant safety systems with their related auxiliary supporting features, therefore, is the basic protective measure which is incorporated in the design to protect against the dynamic effects of postulated pipe break events.

2. Barriers, Shields and Enclosures - In many cases, protection requirements may be met due to walls, floors, columns, abutments and foundations. Where adequate protection does not already exist due to separation, additional barriers, deflectors or shields are provided as necessary to meet the functional protection requirements. Where compartments, barriers and structures are required to provide the necessary protection, they are designed to withstand the following direct effects:

    a. Pressure rise effects

    b. Jet impingement forces due to pipe rupture

    c. Impact due to pipe rupture

    Protection against the environmental effects of water spray and flooding, due to the failure of piping containing moderate-energy fluid, is also provided by barriers or shields as necessary to assure the operation of components required for safe shutdown or required to mitigate the consequences of the failure.

3. Piping Restraint Protection - Where adequate protection does not already exist due to separation, barriers or shields, piping restraints are provided as necessary to meet the functional protection requirements. Restraints are not provided when it can be shown that the broken pipe will not cause unacceptable damage to essential systems or components.

   Where restraints are not feasible, protection of critical systems is provided by one of the following means:

   a. Sections of Increase Pipe Strength

      Rupture of the main steam and feedwater lines adjacent to their containment penetrations is precluded by the use of thickened pipe sections in conjunction with special quality control tests and nondestructive testing, both conducted during Plant construction and by an augmented inservice inspection program.

   b. Concrete Wall or Slab Barriers

      The pressurizer top head and the feedwater lines are protected by concrete slab barriers from a main steam line rupture. Local yielding is permitted as long as there is no general failure.

5.6.5 JET IMPINGEMENT

Jet impingement forces are assumed to act adjacent to postulated circumferential or longitudinal breaks of high-energy piping systems. Essential structures and components could be damaged as a direct or indirect result of jet impingement.

The analysis of jet impingement is based on the procedures contained in ANSI-58.2-1980. In brief, the forces acting on components and targets in the jet stream are calculated as follows:

$$F = P\ A\ \text{Sin}\ \emptyset$$

where

- F = Thrust force on a target (lb)
- P = Pressure on a target at distance X from the pipe (psi)
- A = Target area (in^2)
- $\emptyset$ = Angle of incidence between jet axis and target surface

In addition to the thrust force on a target, consideration should be given to the effects of jet impingement temperature and moisture.

5.6.6 PLANT MODIFICATION LINE-BREAK ANALYSIS

5.6.6.1 Plant Modifications Involving High- or Moderate-Energy Piping

Modification to an existing high- or moderate-energy system shall be reviewed to determine if additional line-break analysis is required. The same criteria as for the original analysis apply.

5.6.6.2 Plant Modifications Involving Essential Systems and Components

Modification to an essential system or component shall be reviewed to determine if there is interaction with any postulated high-energy line breaks. The procedure to accomplish this task is as follows:

1. Determine the exact location of proposed system or component.

2. Review Reference 2 for break outside containment or Topic III-5.A, Phase IV (Reference 6) for breaks inside containment to determine whether the breaks interact with the proposed system or component.

3. Determine whether the interaction remains resolved or additional target analysis is required.

The criteria contained in ANSI 58.2 (Reference 7) shall be followed for required target analysis.

5.6.7 HISTORY OF PALISADES HIGH-ENERGY LINE-BREAK ANALYSIS

5.6.7.1 High-Energy Line Breaks Outside Containment

In December 1972, the NRC initially raised the concern for the dynamic effects of pipe ruptures (Reference 1). In response to this letter, Consumers Power Company had an analysis done for postulated high-energy line breaks outside of containment. This report is entitled "Special Report No 6 - Analysis of Postulated High-Energy Line Breaks Outside of Containment" (SR-6), Revision 3A (Reference 2).

The subject of High-Energy Line Breaks (HELBs) outside of containment was further evaluated in 1981 and 1982 as part of the Nuclear Regulatory Commission's (NRC) Systematic Evaluation Program (SEP) discussed in Subsection 1.8 as Topic III-5.B. This evaluation compared the analysis and criteria used in SR-6, modifications (mainly to pipe supports) made from 1979 through 1981 pursuant to IE Bulletin 79-14 (Reference 3) and the following supplemental evaluation information against present-day methods and criteria. The result was that Consumers Power Company's criteria for postulating pipe breaks and its method for evaluating postulated breaks in high-energy piping outside containment are, in general, in accordance with currently accepted standards (Reference 4).

NRC Generic Letter 87-11 was invoked in 1991 to relax the break location criteria in Section 5.6.3. The relaxed criteria allow the analyst to omit intermediate pipe breaks from the HELB analysis if pipe stresses are less than the limits given in Section 5.6.3, i.e. if stresses are within the given limits, the HELB analysis may only consider breaks at terminal ends.

The safety objective of Systematic Evaluation Program (SEP) Topic III-5.B, "Pipe Break Outside Containment," was to ensure that pipe breaks would not cause the loss of needed functions of safety-related systems, structures and components and to ensure that the Plant can be safely shut down in the event of such breaks. The needed functions of safety-related systems are those functions required to mitigate the effects of the pipe break and permit safe shutdown.

In support of the SEP evaluation, the following additional information was provided:

1. Analytical Results Recorded in SR-6

   The report defines safety systems and evaluates the results of high-energy line breaks outside containment with respect to Plant safety. The report concluded that modifications to the Plant were desirable, and these modifications were incorporated in 1975 time frame.

   In general, the list of safety equipment required for safe shutdown in SR-6 (Table 4-1) is still valid; ie, no significant modifications have been made to the Plant in this area since 1975.

2. Criteria

   The criteria used in SR-6 is the same as Standard Review Plan (SRP) 3.6.1 except as follows:

   a. SR-6 considered that temperature must exceed 200°F and pressure must exceed 275 psig if a line is to be classified as high energy. SRP 3.6.1 defines a line as high energy if either the pressure or the temperature values are exceeded. The only high-energy system involved is the primary coolant letdown system. While the effects of a line break in this 2-inch line were not evaluated because it was not considered high energy, the break would not have been considered significant by the size and/or location criteria per Page 7-1 of SR-6.

A reanalysis has verified this statement. Review of the electrical drawings shows no electrical cable trays in the penetration area. However, where the 2-inch letdown line is located, conduits running to several containment isolation valves for power and position indicators enter the area. The isolation valves of interest are solenoid-operated control valves which close on loss of power and loss of air. Loss of the circuits or air supply lines is, therefore, in the fail-safe direction.

The piping and containment penetrations for the isolation valves are also located in this area (602-foot evaluation pipe way). The valves and piping from these adjacent systems might be a target for an interaction with the letdown pipe should it fail. The letdown system outside containment contains less energy than the primary system since this portion of the line is downstream of the letdown orifices and the letdown heat exchanger. Any highly improbable failures of the piping associated with adjacent systems due to interactions with the letdown system would not be significant and would not inhibit the Plant's ability to shut down or maintain a shutdown condition. The adjacent systems in the area are the drain line from the primary system drain tank, primary coolant controlled bleedoff, inlet to waste gas surge tank, discharge line from degasifier pumps and the discharge receiver tank lines.

b. Breaks in portions of the Auxiliary Feedwater System were deemed not credible because of low usage (see Page 7-3 of SR-6) whereas Footnote 6, Page 14 of SRP 3.6.2 specifically notes that the Auxiliary Feedwater System is a high-energy system. Significant modifications have been made to this system in response to NUREG-0737 commitments. These modifications resolve HELB issues.

c. Breaks Selected on Stress Points

Certain high-energy line breaks were postulated by SR-6 based on calculated stresses. Breaks were assumed in a pipe run at two intermediate locations of highest combined stresses. Breaks were also assumed when calculated stresses exceeded 0.8 (S_h + S_A) or the expansion stresses exceeded 0.8 S_A. Subsequent to completion of SR-6, large bore safety piping at Palisades was reanalyzed (Reference 3) based on "as-built" data collected in 1979 and 1980. As a result of this reanalysis, some points of highest combined stresses changed from those points considered by SR-6. The high stress point relocations have been reviewed on a sample basis. The relocations are small and are not significant with respect to installed restraints; ie, the relocations do not invalidate SR-6 results.

3. Plant Heating System HELB Evaluation

The Plant heating system is designed for a maximum steam service condition of 15 psig and 250°F and is, therefore, a high-energy fluid system according to the criteria of SRP 3.6.1. A low-pressure, forced draft boiler with a capacity of 23,830 lb/h of steam at 212°F supplies Plant heating steam to the auxiliary building, turbine building and containment as well as process heat to safety injection and refueling water (SIRW) heat exchanger, condensate tank heat exchanger, primary makeup storage tank heat exchanger and domestic water storage tank heat exchanger. The boiler is equipped with operating and combustion safety controls to turn the boiler on and off in response to steam pressure, water temperature or water level and modulate firing rate in response to heat demand. An additional source of heating steam is LP turbine steam extraction which supplements the boiler. Piping carrying the heating steam is designed and fabricated in accordance with USAS B31.1.0, 1967, Power Piping Code.

An "effects oriented" approach was utilized to determine susceptibility to Plant heating system line breaks. Break effects considered were compartment pressurization, jet impingement, flooding and environmental conditions of temperature, pressure and humidity in safety-related areas only.

Heating in the switchgear area (elevation 625 feet, 0 inch) in the auxiliary building is provided by Heating Coil VHX-33, located in an adjacent turbine building fan room. A 4-inch Plant heating steam line supplies steam to VHX-33. This line does not run through the switchgear area and a full circumferential double-ended break would not create adverse pressurization in the switchgear or cable spreading rooms. Jet impingement impact on the safety-related equipment is not considered credible because of their physical separation and location.

Flooding in the area would be controlled by existing drainage facilities. In case of failure of Ventilating Fan V-33, due to pipe-break effects, Ventilating Fan V-47 could be used to provide ventilation to enable Plant shutdown.

A 6-inch heating steam line is currently routed to provide steam to Heating Coils VHX-17 and VHX-34 (elevation 639 feet, 0 inch) for control room heating; these heating coils are no longer used. HVAC modifications were completed in 1983 eliminating the use of the existing Air Handling Units V-17 and V-34 and the heating coils. The two new control room HVAC units are located in a safety-related area and do not utilize steam for heating.

Air Handling Units V-17 and V-34 and their heating coils are not located in the control room, and a heating steam-line break would not directly affect the control room because the units are no longer used.

The fuel handling area (elevation 649 feet, 0 inch) is heated by Heating Coils VHX-7A and VHX-7B (elevation 651 feet, 0 inch) extracting steam through a 4-inch Plant heating steam line. A break in the heating line would not produce significant pressurization or short-term temperature rise because of the large room volume combined with low steam-line pressure. The line is physically separated from other equipment and structure and jet impingement impact due to a low-pressure heating steam-line break would not affect the function of any safety-related equipment. Flooding in the area would be controlled by existing drainage facilities.

The Containment Purge Supply Temperature Coil VHX-5 and Air Room Purge Supply Heating Coil VHX-48 extract steam from a 4-inch Plant heating steam line. Also, the safety injection and refueling water (SIRW) tank heat exchanger draws steam from another 4-inch Plant heating steam-line branch. These are in a safety-related area for which main steam-line break analyses have been provided in SR-6. Any break of the low-pressure heating steam lines in the area would not create conditions as severe as those already analyzed in SR-6 for main steam or feedwater line breaks. Thus, the integrity of safety equipment would not be threatened by Plant heating steam-line breaks.

5.6.7.2 <u>High-Energy Line Breaks Inside Containment</u>

The subject of High-Energy Line Breaks (HELBs) inside of containment was initially raised as Systematic Evaluation Program (SEP) Topic III-5.A discussed in Subsection 1.8. In response to this topic concern, Consumers Power Company had an analysis done to evaluate the postulated effects of HELBs inside of containment (References 5 and 6).

Of the 1,246 potential interactions identified, only one remained unresolved. The remaining postulated condition was in the 3-inch pressurizer spray-line pipe-whip interaction with a cable tray containing power cables to Hydrogen Recombiner M69B. Based on the following argument, the NRC accepted Consumers Power Company's position that Plant modification is not justified for this break.

The postulated break of the pressurizer spray line was assumed as a double-ended guillotine rupture that results in the whipping pipe striking the cable tray approximately 20 feet from the pipe. Break fracture mechanics analysis showed that a stable 90° crack with a leak rate of approximately 0.1 gpm was possible without immediate pipe failure. A leak rate of this magnitude was probably too small to be readily detectable by present in-Plant leak detection equipment. However, the leak would be detected in the daily calculation of the primary coolant inventory. Based upon the preliminary deterministic fracture mechanics analysis, Consumers Power Company believed that the possibility of the 3-inch pressurizer spray-line break resulting in a pipe whip was unlikely. Therefore, the need to modify the Plant to add whip restraints or a barrier to protect the target cable tray was unwarranted. Furthermore, a modification to add local leak detection to monitor one weld was also not warranted. The weld associated with the break location was examined to ASME B&PV Code, Section XI, Class 1 requirements.

The postulated break was listed as unresolved in the enclosed evaluation due to the severance of the power cables to the hydrogen recombiner coupled with a single failure of the other diesel generator supplying power to the redundant hydrogen recombiner. Because the recombiner was not an active component, a single failure of it solely did not need to be considered.

A change to 10 CFR 50.44 for combustible gas control (Reference 9) eliminated the hydrogen release associated with a design basis LOCA and the associated requirements for the hydrogen recombiners. The hydrogen recombiners were subsequently removed from containment.

Subsequent to the SEP review of HELBs within containment, concerns were raised regarding the vulnerability of CCW piping within containment to failure caused by an HELB. Failure of this CCW piping, coupled with the CCW supply valve to containment (CV-0910) failing open due to a loss of air, could cause a complete loss of CCW inventory. An evaluation of this issue concluded that this CCW piping within containment is not vulnerable to failure caused by an HELB (Reference 8).

NRC Generic Letter 87-11 was invoked in 1991 to relax the break location criteria in Section 5.6.3. The relaxed criteria allow the analyst to omit intermediate pipe breaks from the HELB analysis if pipe stresses are less than the limits given in Section 5.6.3, i.e. if stresses are within the given limits, the HELB analysis may only consider breaks at terminal ends.

5.6.7.3 Moderate-Energy System Pipe-Break Evaluation

There are two systems which are moderate energy Class 1 systems. They are the Component Cooling Water (CCW) and Service Water System (SWS). The systems have piping runs in several areas, some areas of which are common to both systems.

Either safeguards rooms (east or west) could be flooded above the level where equipment could become inoperable, but redundant equipment would remain available in the adjacent safeguards room which is sealed with a watertight door. An indication of flooding or excessive drainage to the room sumps would be provided by the sump high-level alarm. Moderate-energy line breaks in the CCW room are considered less severe than a feedwater line break. Therefore, the flooding and wetting effects in the CCW room were not analyzed for these moderate-energy line breaks.

Breaks of the SWS in the intake structure will not result in equipment flooding. However, NRC Branch Technical Position (BTP) MEB 3-1 requires assuming all unprotected components are wetted within the compartment. This could affect all three service water pumps. The SWS pumps initially had open drip-proof enclosures that could not protect the motors from direct water spray. Spray shields have since been installed on the SWS pump motors to protect them from wetting. Wetting of equipment in other areas will not cause failure of redundant equipment.

Because of large flow rates for the SWS and CCW pumps, it is unlikely the cooling functions of either system would be affected significantly by a critical crack unless the postulated break occurs on a branch connection supplying specific equipment. For example, a break in a 2-inch service waterline to the control room air-conditioning unit may cause some loss of cooling (break flow rates would be about 35 gpm). The second control room cooler, however, is supplied by the redundant SWS line; therefore, the cooling function is not lost. Wetting of both control room air-conditioning units is again possible, but loss of function of both units will not inhibit safe shutdown of the Plant.

REFERENCES

1. Letter from A Giambusso (AEC) to R C Youngdahl (CP Co), PW721215A, Attachment 1, "General Information Required of the Effects of a Piping System Break Outside Containment."

2. "Consumers Power Company, Palisades Plant, Special Report No 6: Analysis of Postulated High Energy Line Breaks Outside Containment," Revision 3A, July 2007. Prepared by Bechtel Associates Professional Corporation. Transmitted to the NRC via PW810825A (Cart\Frame No 2642\2121).

3. NRC IE Bulletin 79-14, "Seismic Analysis for As-Built Safety Related Piping Systems," Revision 0 through Revision 1, Supplement 2, dated July 2, July 18, August 15 and September 7, 1982, respectively. Transmitted to CP Co via PR790702B, PR790720A, PR790815A and PR790907B, respectively.

4. NRC Systematic Evaluation Program, Topic III-5.B, "Pipe Break Outside Containment," letter from T V Wambach (NRC) to D J VandeWalle (CP Co), PW820219B.

5. NRC Systematic Evaluation Program, Topic III-5.A Report, "...High Energy Line Breaks Inside Containment, Palisades Nuclear Station, Summary Report," July 1981. Prepared by EDS Nuclear. Transmitted to the NRC via PW810813A (Cart\Frame No 2642\1760).

6. NRC Systematic Evaluation Program, Topic III-5.A Report, "...High Energy Line Breaks Inside Containment, Palisades Nuclear Station, Phase IV Summary of Results," Revision 0, November 1982. Prepared by EDS Nuclear, Melville, New York. Transmitted to the NRC via PW821209A (Cart\Frame No 2707\2004).

7. ANSI/ANS-58.2-1980, "Design Basis for Protection of Light Water Nuclear Power Plants Against Effects of Postulated Pipe Rupture."

8. Deviation Report No D-PAL-89-061, "Post Accident Operation of CCW System," initiated March 23, 1989.

9. 10 CFR 50.44 Combustible Gas Control for Nuclear Power Reactors, effective October 16, 2003.

5.7 SEISMIC DESIGN

This section deals with the seismic analysis, testing and design of CP Co Design Class 1 structures, systems and components at the Palisades Nuclear Power Plant. The term "CP Co Design Class 1 Structure," as used herein, is equivalent to the term "Category I Structure," which is current practice. Computer programs used in the original design analysis of CP Co Design Class 1 structures, systems and components are listed in Table 5.7-1.

Subsection 5.7.1 deals with the seismic input of all the original CP Co Design Class 1 structures, systems and components. Revised seismic input criteria for CP Co Design Class 1 piping is discussed in Subsection 5.7.4. The seismic input discussed in Subsection 5.7.4 apply to all reevaluations and modifications to CP Co Design Class 1 piping being reviewed after July 1986.

5.7.1 SEISMIC INPUT

5.7.1.1 Design Bases

Based on the conclusions described in Subsection 2.4.4 on seismicity, the CP Co Design Class 1 structures, systems and components have been designed to resist two seismic events:

1. Design Earthquake (E) = 0.1 g. This event is equivalent to the Operating Basis Earthquake (OBE). The Plant must remain operational with no loss of function up to this level.

2. Maximum Credible Earthquake (E') = 0.2 g. This event is also known as the Hypothetical Earthquake and it is equivalent to the Safe Shutdown Earthquake (SSE). All structures, systems and components required to achieve and maintain safe shutdown of the Plant are required to remain operational with no loss of function up to this level.

The vertical ground acceleration for each event is taken as two-thirds (2/3) of the corresponding horizontal ground acceleration. This assumption has been supported as being conservative by Dr George W Housner of the California Institute of Technology, a noted expert in this area.

The current terminology, OBE and SSE, will be used throughout the remainder of this section and elsewhere in the FSAR Update.

5.7.1.2 Ground Design Response Spectra

The ground design spectrum used in the analysis of ground-supported structures is shown in Figure 5.7-1, normalized to a maximum ground acceleration of 0.1 g (OBE). The horizontal ground design spectrum for the SSE is obtained by multiplying values from the OBE spectrum by a factor of two. This spectrum was generated by averaging many acceleration spectra from actual earthquake records, normalized to the same maximum ground acceleration, principally El Centro 1934 and 1940, Olympia 1949 and Taft 1952. The average response spectrum thus obtained covers a variety of foundation conditions ranging from rock to deep alluvium. This spectrum is commonly referred to as the "Housner" spectrum. Since the ground surface acceleration values used in the design are twice those recommended in Subsection 2.4.4, it is felt that any unconservatism resulting from the spectrum-averaging process has been adequately covered.

5.7.1.3 Floor Design Response Spectra

Floor design response spectra for the analysis of structurally-supported systems and components were developed from a modal time history analysis of the containment building, the auxiliary buildings and other CP Co Design Class 1 structures for which spectra were produced, using the Taft 1952 earthquake acceleration record normalized to the OBE. The time history record was digitized to 0.01 second intervals and had a duration of 24 seconds. The building models used in the analysis are described in Subsections 5.7.2 and 5.7.3.

Comparisons of the Taft 1952 and Palisades smooth ground design spectra are shown in Figures 5.7-2 and 5.7-3 for 4% and 7-1/2% of critical damping, respectively. Floor response spectra were produced only for the OBE case and doubled for SSE analyses. This is considered conservative since structural damping is higher for the SSE level. Where no vertical structural dynamic analyses were performed, the ground design spectra, normalized to the OBE, were used for vertical analyses of in-structure systems and components.

The modal time history analysis produced acceleration time histories at dynamic degrees of freedom of the building model. These time histories were then filtered through a family of single degree of freedom systems with various natural frequencies and damping values to produce rough floor response spectra. These rough plots were then smoothed using straight-line segments which generally envelope the data. The spectra converge to the maximum floor accelerations from the building response spectrum analyses at 33 Hz.

Floor response spectra for the containment building, the auxiliary building and the auxiliary building radwaste addition were produced using Bechtel Computer Code CE611. Spectra were only produced for horizontal directions. They were computed for frequencies ranging from 0.1 to 33.0 Hz, and peaks at building natural frequencies were broadened ± 10% to account for variations in soil and structural material properties. Containment building and auxiliary building and floor spectra were generally produced for 0.5%, 2.0% and 5.0% of critical damping. Auxiliary building radwaste addition spectra were produced for 0.5% of critical damping.

Floor response spectra for the auxiliary building TSC/EER addition were produced using Bechtel Computer Codes CE800 (BSAP) and CE802 (SPECTRA). Spectra were generated in accordance with the recommendations of USNRC Regulatory Guide 1.122 (Reference 1) with respect to frequency intervals (0.2 to 34.0 Hz), peak broadening (± 15%) and combination of three directions of earthquake motion (SRSS). The spectra were made to envelope the ground design spectra in all cases. Spectra were produced for 0.5%, 1.0%, 2.0%, 3.0%, 4.0%, 5.0%, 7.0% and 10.0% of critical damping.

Floor response spectra are shown in Reference 21.

As an outgrowth of the USI A-46 and IPEEE work, new spectra were developed for Palisades using the methodology delineated in RG 1.60 and conforming to the guidance of the Standard Review Plan (References 22 & 23). This spectra uses modern methods for modeling soil springs and soil-structure interaction. This spectra can be considered a "conservative design" spectra. This new spectra will be used in resolving outliers by demonstrating compliance, either by evaluation or enhancement, with the methods of the Generic Implementation Procedure. The use of this spectra for resolving outliers was proposed in the USI A-46 SQUG Assessment Summary Report submitted to the NRC on May 23, 1995, and was found to be acceptable in an NRC SER dated September 25, 1998 (Reference 24).

5.7.1.4 Damping Values

The damping values expressed as a percent of critical for various materials and types of construction are shown in Table 5.7-2 for the OBE and SSE. The damping values for various CP Co Design Class 1 structures are shown in Table 5.7-3. The analysis of composite structures, such as the containment and auxiliary buildings, used the various techniques described in Subsections 5.7.2 and 5.7.3 to incorporate different damping values for soil, concrete and steel.

5.7.2 SEISMIC ANALYSIS OF MAJOR CP CO DESIGN CLASS 1 STRUCTURES

The horizontal dynamic analysis of major CP Co Design Class 1 structures was accomplished in a series of steps as follows:

1. A mathematical model of the structure was constructed in terms of lumped masses, interconnected by massless beam elements. At appropriate locations within the structure, such as floor levels, points were chosen to lump the weights of the structure and major equipment. Between these locations, values were calculated for beam element moments of inertia, cross-sectional areas, effective shear areas and lengths. The member properties were used in Bechtel Computer Program CE309 (Stress) to obtain flexibility coefficients referenced to the lumped mass locations in the mathematical model. Since all major structures are separated by a minimum of two inches above grade and one inch below grade, they were analyzed separately with no dynamic coupling.

2. The natural frequencies and mode shapes of the structure were obtained using Bechtel Computer Program CE617. The flexibility coefficients were formulated into a matrix and inverted to form a stiffness matrix. The lumped weights were formulated into a diagonal mass matrix. The program used the technique of diagonalization by successive rotations to obtain eigenvalues (natural frequencies) and eigenvectors (mode shapes).

3. The response of the structure to the earthquake was obtained using Bechtel Computer Code CE641 which utilizes the modal response spectrum analysis technique. The ground design response spectra described in Subsection 5.7.1.2 were used in the analysis. Using a weighting technique, the modal damping values were established based upon the mode shapes and the damping values presented in Subsection 5.7.1.4.

   For each mode, based on the natural frequency and the damping value, a spectral acceleration was obtained from the ground design spectrum. This acceleration was multiplied by the modal participation factor, mode shape vector and lumped mass to obtain inertial forces at the lumped mass points. Shears and moments were then computed from the inertial forces. Mass point displacements were computed by multiplying the spectral acceleration by the participation factor and mode shape vector and dividing by the square of the circular (radians) natural frequency. All modes with natural frequencies less than 33 Hz were combined by the square-root-of-the-sum-of-the-squares (SRSS) method to obtain inertial forces, shears, moments and displacements at lumped mass points throughout the structure.

4. The results from each horizontal analysis were combined separately with the vertical results. The worst case from these combinations was used in the structural analysis and design.

The soil-supported structures were checked for dynamic stability against overturning and sliding, and compared with the minimum allowable factors of safety which are 1.5 for the OBE and 1.1 for the SSE.

5.7.2.1 Containment Building

Three separate horizontal dynamic analyses of the containment building have been performed.

In early 1967 a fixed-base, single-stick, dynamic model of the containment shell was analyzed for the OBE only, using 2% damping in all modes. The modal responses were combined by the sum-of-absolute-values (SAV) technique. The forces generated by this analysis were used in the structural design.

Later, a second dynamic model incorporating soil-structure interaction was analyzed for both the OBE and SSE. The soil springs were based on formulas by P A Parmelee (Reference 2) for translation (swaying) and rotation (rocking). The rotational soil spring was replaced by two equipollent vertical springs. The mass of the internal structure was lumped at the base of the containment along with the basemat mass. Local rotational inertia was neglected; however, the overall rotational degree of freedom at the basemat elevation was included. The analysis used 4% damping for all modes in the OBE analysis and 7.5% damping for all modes in the SSE analysis. The modal responses were combined by the SRSS method.

The final dynamic model was analyzed in June 1969 and incorporated both soil-structure interaction and the coupling effect between containment shell and internal structures. The internal structures were modeled as a separate stick with zero offset from the containment stick and coupled at the basemat elevation only. The dynamic model is shown in Figure 5.7-4. Damping was determined for each mode based on a weighting technique which considered the mode shapes and material damping values shown in Table 5.7-3. Damping in the OBE analysis was 5% in the first and second modes and 2% in the third and fourth modes. Damping in the SSE analysis was 7.5% for all 4 modes. See Reference 16 for additional information.

The modal responses for the first four modes were combined by the SRSS method. This final dynamic model was also used in the generation of OBE floor design response spectra as described in Subsection 5.7.1.3.

A comparison of containment shell shears and moments obtained from the first and final (third) dynamic analyses is shown in Figure 5.7-5. The values from the original dynamic analysis are consistently higher than those from the final analysis and since the original values were used in the structural design, the design loads were conservatively estimated. See References 16 and 17 for additional information.

None of the dynamic analyses considered torsional behavior. This approach is considered reasonable due to the axisymmetric nature of the structure.

The factors of safety against overturning were computed as 4.9 for the OBE and 2.2 for the SSE, and against sliding as 4.3 for the OBE and 1.9 for the SSE. These values meet the criteria set forth in Subsection 5.7.2 for CP Co Design Class 1 structures on soil.

5.7.2.2 Auxiliary Building

Separate horizontal seismic dynamic analyses of the CP Co Design Class 1 portion of the auxiliary building were performed for the north-south and east-west directions. The mathematical models used in this analysis are shown in Figures 5.7-6 and 5.7-7, and consist of basically three parts:

1. Above elevation 649 feet 0 inch, a lumped mass and beam representation of the structural steel framing.

2. Between elevations 589 feet 0 inch and 649 feet 0 inch, a lumped mass and beam representation of the reinforced concrete walls and floors.

3. Below elevation 589 feet 0 inch, translational and equipollent offset vertical springs to represent soil-structure interaction. The soil springs were computed using formulas for rectangular basemats assuming an effective area of 100 by 158 feet.

For mode shapes and frequencies see Figure 5.7-8.

Damping was determined for each mode based on a weighting technique which considered the mode shapes and material damping values shown in Table 5.7-3. Although modal damping values were computed for both OBE and SSE analyses, only an OBE analysis was performed. The results were doubled for the SSE which is conservative due to the higher SSE damping. The OBE damping values for the north-south analysis were 0.5% in the first mode and 5% in the second and third modes. The OBE damping values for the east-west analysis were 0.5% in the first and third modes, and 5% in the second and fourth modes.

The modal responses for each analysis were combined by the SRSS method and the two sets of results were enveloped to form a single set of results for design. The results of the combined analyses are shown in Reference 17 for the OBE. The SSE results are twice the OBE values. Torsional effects, which arise from the asymmetry of the building, were considered in the design by distributing the horizontal loadings obtained from the decoupled analyses in accordance with the actual shear wall rigidity distribution. No dynamic coupling was considered between the auxiliary and turbine buildings. Typical floor response spectra are shown in Reference 21.

The factors of safety against overturning were computed as 8.5 for the OBE and 3.8 for the SSE, and against sliding as 2.7 for the OBE and 1.2 for the SSE. These values meet the criteria set forth in Subsection 5.7.2 for CP Co Design Class 1 structures on soil.

5.7.3 SEISMIC ANALYSIS OF OTHER CP CO DESIGN CLASS 1 STRUCTURES

5.7.3.1 Turbine Building

Only those portions of the turbine building listed in Section 5.2 are designated CP Co Design Class 1. The remainder of the turbine building is CP Co Design Class 3.

5.7.3.1.1 CP Co Design Class 1 Portion

The auxiliary feedwater pump room, which is completely below ground level, was analyzed for seismic accelerations equal to the ground accelerations.

Separate horizontal dynamic analyses of the electrical penetration enclosure were performed for the north-south and east-west directions. The basic methodology was the same as for the major CP Co Design Class 1 structures as described in Subsection 5.7.2. The effects of soil-structure interaction were not considered in the analyses, and fixed-base, single-stick models were used. Only OBE analyses were performed with 5% damping assumed in all modes. Only one mode in each direction has a natural frequency less than 33 Hz. The dynamic models, natural frequencies and shear envelopes for both analyses are shown in Figure 5.7-8.

5.7.3.1.2 CP Co Design Class 3 Portion

The CP Co Design Class 3 portion of the turbine building was initially connected to the auxiliary building. Also, it is adjacent to the CP Co Design Class 1 portion of the intake structure. Therefore, a dynamic analysis of the turbine building was performed to assess the effect of the seismic activity of this structure upon the adjacent structures. This analysis was performed for the SSE using a model of the less rigid east-west direction. The basic methodology described in Subsection 5.7.2 was used. The effects of soil-structural interaction were not included in the analysis.

Three cases were analyzed:

1. The turbine building frame was considered to be restrained laterally by its ties to the auxiliary building (original configuration). This occurred along Column Row J between Column Rows 16 and 22. In this area the roof girders of the turbine building auxiliary bay (roof elevation 625 feet 0 inch) were connected to secondary columns of the frame which are encased by the auxiliary building wall. The fundamental frequency was found to be 1.17 Hz.

   It was concluded that this connection will not cause failure of the auxiliary building wall or the roof over the turbine building auxiliary bay. However, it was also concluded that the auxiliary building floor slab at elevation 625 feet 0 inch was overstressed. This overstress condition was eliminated prior to the completion of construction by providing a 3-inch gap between the turbine building girders and the auxiliary building wall. Vertical supports with sliding surfaces were attached to this wall. In the east-west direction, flexible slotted bolt connections with a ±3-inch range were employed.

2. The turbine building was considered to be a rigid frame, supported at the ground floor level and unrestrained at the operating floor level (elevation 625 feet 0 inch). The fundamental frequency was found to be 0.66 Hz. The maximum frame deflection at this elevation was calculated to be 3.44 inches. This deflection will close the 0.75-inch gap between the turbine generator pedestal and the operating floor, thus causing the pedestal to act as a restraint. This gap closure necessitated the Case 3 analysis.

3. The turbine generator pedestal was treated as a restraint to the building frame at the operating floor level. A dynamic analysis of this final case was not performed. Instead, the accelerations from Case 2 were applied directly to the Case 3 model to produce nodal forces, and then a static analysis was performed. This approach is quite conservative since the accelerations for the restrained case should be considerably lower. The maximum deflection at elevation 625 feet 0 inch was reduced to 1.32 inches. It was concluded that the resulting lateral force would not affect the turbine pedestal.

The dynamic models for the three cases are shown together in Figure 5.7-9. Fundamental mode shapes for the first two cases are shown in Figure 5.7-10. Acceleration and displacement responses for all three cases are shown in Figure 5.7-11.

In all three cases covered above, the crane was assumed unloaded and located at any bay of the turbine building. The mode shapes indicate that the crane support columns move in the same direction. Based on the uniform column movement and the fact that column stresses were determined to be within allowable limits, it was concluded that the turbine building will not collapse and the crane bridge and trolley will remain in place during a seismic event.

The maximum seismic deflection of the turbine building roof at elevation 676 feet 9 inches ($\pm$ 2.75 inches) is more than sufficient to close the gaps along Column Row M of the auxiliary building and along the intake structure. In the auxiliary building, these additional forces will be carried by concrete shear walls whose overall stress level will not exceed 85% of yield. Because the calculated deflection at the roof level of the intake structure is less, the additional forces will be less and the intake structure walls will be stressed to less than 85% of yield.

5.7.3.2 Intake Structure

Only that portion of the intake structure listed in Section 5.2 is designated CP Co Design Class 1. The remainder of the intake structure is CP Co Design Class 3. The intake structure is mainly below ground and was analyzed for seismic accelerations equal to the ground accelerations.

5.7.3.3 Auxiliary Building Radwaste Addition

A structure was added adjacent to the north end of the auxiliary building in 1972 to house a radwaste addition and to extend the fuel handling crane into this area. The structure is isolated by expansion joints from the auxiliary building to the south, service building to the north, and technical support center addition to the west, and it is supported by its own basemat. The radwaste addition consists of a reinforced concrete structure from its base at elevation 590 feet to elevation 665 feet, and a steel braced frame from elevation 665 feet to elevation 696 feet where a steel roof truss supports a lightweight roof. The fuel handling crane rails extend into this structure at elevation 676 feet.

Separate horizontal dynamic models were developed for the north-south and east-west directions. The effects of soil-structure interaction were incorporated using translational and equipollent offset vertical springs computed using formulas for rectangular basemats, assuming an effective area of 38.5 feet by 125 feet. The mass of the steel superstructure and crane was lumped at the top stick model mass point (elevation 665 feet). The dynamic model is shown in Figure 5.7-12.

A modal analysis was performed as described in Subsection 5.7.2 for the north-south and east-west directions. The fundamental natural frequencies from this analysis were 3.4 Hz for the north-south direction and 5.3 Hz for the east-west direction.

A response spectrum analysis was performed as described in Subsection 5.7.2 for the north-south and east-west directions. The 0.1 g OBE Palisades ground design spectrum with 5% structural damping was used to obtain acceleration, shear and moment responses.

Floor response spectra were generated for the north-south and east-west directions as described in Subsection 5.7.1.3. Floor response spectra are shown in Reference 21.

SSE values were obtained by doubling OBE values.

5.7.3.4 Auxiliary Building TSC/EER Addition

The technical support center (TSC) and electrical equipment room (EER) were added to the auxiliary building in 1983. A major portion of this reinforced concrete addition was built on top of the existing waste gas decay tank room (WGDTR) between elevations 607 feet 6 inches and 639 feet. The combined structure, consisting of WGDTR, EER and TSC, is isolated by expansion joints from the auxiliary building radwaste addition to the east and the remainder of the TSC/EER addition to the south.

Two separate three-dimensional dynamic stick models were used to analyze the structure. The first analysis assumed that the torsional stiffness of the stick model elements was the sum of the individual rectangular wall section torsional stiffnesses. The second analysis assumed that the floor slabs were rigid, causing the individual wall sections to act together, providing increased stiffness.

Flexible vertical elements in the dynamic model were located at the centers of horizontal stiffness of the wall systems between floors and were connected by rigid horizontal elements at floor elevations to lumped masses located at their respective centers of gravity. Additional rigid horizontal members were included at each floor level to obtain responses at the corners of the structure. The effects of vertical floor flexibility on vertical response were included in the dynamic model through the addition of single-degree-of-freedom vertical spring-mass systems at each lumped mass point (except the basemat). The effective mass and stiffness were based on the floor's vertical (transverse) first mode of vibration. The effective mass was subtracted from the total vertical floor lumped mass. The effects of soil-structure interaction were simulated using translational and rotational soil springs for a rectangular basemat. The dynamic model is shown in Figure 5.7-13.

For each model a fixed base, modal free vibration analysis, neglecting soil-structure interaction, was first performed to obtain structural mode shapes and frequencies. Soil spring and radiation damping values were computed and input along with the modal data into the DAMPSI (CE207) module of Bechtel Computer Program BSAP-DYNAM to compute composite modal damping, using a method described in Bechtel Topical BC-TOP-4-A (Reference 3). Structural (reinforced concrete) hysteretic damping was input as 5% of critical damping, and soil material damping was input as 3% of critical damping. In no case, however, was the composite modal damping for any mode allowed to exceed 10% of critical damping in the analysis. A flexible base, modal free vibration analysis, including soil-structure interaction springs was then performed, for each model, to obtain system natural frequencies, mode shapes and participation factors. The results of this second modal free vibration analysis are summarized in Table 5.7-4. Mode shapes were plotted using Bechtel Computer Program BSAP-POST (CE201).

An OBE modal response spectrum analysis was performed for both torsional models using the Palisades ground design spectra normalized to 0.10 g horizontal and 0.067 g vertical maximum ground accelerations. Bechtel Computer Program BSAP-DYNAM was used to compute accelerations and displacements and element stresses. The modal responses were combined by the USNRC Regulatory Guide 1.92 "Grouping" method (Reference 4), and the results for each direction of motion were combined by the SRSS method. The horizontal displacements were computed at the four corners of each floor and the results were enveloped. The results of the response spectrum analysis are summarized in Figure 5.7-14.

Floor response spectra were produced as described in Subsection 5.7.1.3. The horizontal floor response spectra from both torsional models and for two directions were enveloped to form a single plot for each floor elevation. The vertical floor spectra included the effects of vertical floor flexibility. Floor response spectra are shown in Reference 21.

In all cases, SSE values were obtained by doubling the OBE values.

The response spectrum and time history analyses used a sufficient number of modes to include 99.9% of the modal mass and all the modes with natural frequencies less than 33 Hz.

The factors of safety against sliding were computed as 2.85 for the OBE and 2.20 for the SSE. The factors of safety against overturning were computed using an energy method described in Bechtel Topical Report BC-TOP-4-A (Reference 3). The minimum factors of safety for tipping about the east edge were 88.8 for the OBE and 11.2 for the SSE. Soil pressure under the building basemat was checked against bearing capacity using Bechtel Computer Program CE705.

5.7.3.5 Other Auxiliary Building Additions

A reinforced concrete structure housing HVAC area was added in the north-west corner of the auxiliary building in 1983. This structure rests on top of the enclosure for the diesel generator exhaust mufflers at elevation 629 feet 2 inches and extends vertically to elevation 659 feet 2 inches.

A portion of the reinforced concrete auxiliary building TSC/EER addition was built on top of the existing baler room between elevations 607 feet 6 inches and 639 feet. This portion is isolated from the remainder of the TSC/EER addition by expansion joints.

The effect of these additions on the seismic response of the auxiliary building was considered to be negligible due to their relatively small mass.

5.7.3.6 CP Co Design Class 1 Tank Foundations

Foundations for the condensate storage tank, primary system makeup tank and utility water storage tank were designed for Dead, Snow, Wind, OBE and SSE loads. In addition, sloshing of liquids inside the tanks was considered. An equivalent static analysis was used to determine required reinforcing. The design of the condensate storage tank valve pit was based on 0.1 g horizontal and 0.07 g vertical accelerations acting on the condensate storage tank foundation.

The foundation for the fuel oil storage tank T-10A was designed for Dead, Snow, Wind, Tornado, OBE, SSE and Blast Loads. The foundation is a below grade housing which serves to protect the fuel oil storage tank.

5.7.3.7 Miscellaneous Frames and Trusses

As part of the Nuclear Regulatory Commission's Systematic Evaluation Program, the containment dome trusses which support the safety injection tanks were evaluated for adequacy during an SSE. The stress resultants from the EDS analysis (Reference 5) of the safety injection tanks for GRAVITY + SSE were applied as loads on the trusses in a static analysis. The results for the vertical and two horizontal directions of earthquake were combined simultaneously by the SRSS method. All combined stresses were found to be within ASME Boiler and Pressure Vessel Code allowables.

5.7.4 SEISMIC ANALYSIS OF CP CO DESIGN CLASS 1 PIPING

CP Co Design Class 1 piping, except the primary coolant piping, was originally analyzed by one of three possible methods:

1. Piping with a fundamental natural frequency below 20 Hz was classified as flexible and a three-dimensional response spectrum OBE analysis was performed using either EDS Corporation Computer Program PISOL or Bechtel Computer Program ME632 or ME101. This was the method generally used for large pipes, 3 inches and over. The piping system was modeled with lumped masses located at valves, pipe supports, elbows, tees and other appropriate locations connected by straight and curved prismatic elastic members. The model was bounded by anchors and equipment. Insulation and content weight were included in the analysis. The effect of flexible equipment to which the piping was attached was included in the analysis. The effect of small equipment, where equipment-piping interaction was significant, was also considered. The three-dimensional stiffness matrix was determined by the direct stiffness method. Axial, shear, flexural and torsional deformations of each member were included. For curved members, a decreased stiffness was used in accordance with the USA Standard Code for Pressure Piping, Power Piping, USAS B31.1.0 (1967). A diagonal lumped mass matrix was also computed.

   The modified Jacobi method was used to compute mode shapes and frequencies. The appropriate OBE floor response spectra for 0.5% damping were used to determine the system displacements in each mode. Piping systems spanning two or more elevations used the spectrum curve for the elevation closest to and higher than the center of mass of the piping system. The inertial forces at each mass point for each mode were obtained by multiplying the stiffness matrix by the displacement vector for each mode. The member forces in each mode were obtained by transforming the inertial force vector for each mode from the global to local coordinate system of each member.

   The computer programs allowed for two different options in combining the modal responses. Either all modes with a natural frequency below 20 Hz or a maximum of 10 modes were combined. At 20 Hz, the rigid range of the floor response spectra is essentially attained and 10 modes are usually quite adequate to determine the response of the piping system. The modal responses were combined by the SRSS method.

   Stresses were computed in accordance with USAS B31.1.0 (1967). Piping systems were analyzed for each horizontal direction combined simultaneously with the vertical direction. The maximum stress at any point in the pipe was the larger number obtained from either of these analyses.

2. Piping with a fundamental natural frequency above 20 Hz was classified as rigid and analyzed statically for maximum floor accelerations. This method was generally used only for small pipe, 2-1/2 inches and under. The rigidity requirement was achieved by limiting the piping spans. These span values were obtained assuming a single-span, simply-supported, straight pipe with rigid supports. The stresses in piping designed by this method were relatively low.

3. Piping not classified as rigid or flexible was analyzed statically using the response spectra peak values.

The SSE results were obtained by doubling the OBE results.

Some piping systems contain shock suppressors (snubbers) to minimize the possibility of unrestrained pipe motion as might occur during an earthquake or severe transient. Table 5.7-9 lists Safety-Related Hydraulic Shock Suppressors and Table 5.7-10 lists Safety-Related Mechanical Shock Suppressors in the Palisades Plant.

The effect of appendages such as valve operators on piping were considered statically using the accelerations from the piping analysis or building accelerations as appropriate.

In some cases, CP Co Design Class 1 piping is directly connected to CP Co Design Class 2 piping. Where the size of the CP Co Design Class 2 piping is not small in comparison to that of the CP Co Design Class 1 piping, such as for the main steam line, the entire piping network was analyzed as a CP Co Design Class 1 system. Where the size of the CP Co Design Class 2 piping is small in comparison to that of the CP Co Design Class 1 piping, the systems were analyzed separately. For this case, the effect of the CP Co Design Class 2 piping on the CP Co Design Class 1 piping was accounted for by lumping a portion of the CP Co Design Class 2 piping mass at its point of attachment to the CP Co Design Class 1 piping.

As part of Consumers Power response to the Nuclear Regulatory Commission's Inspection and Enforcement Bulletin 79-14, seismic reanalysis was performed on all lines greater than 2-1/2 inches, with the exception of the main primary coolant piping. As a result of this reanalysis, a number of supports were added, removed or modified (see Subsection 5.10.3.1.3).

In July 1986, the seismic input and certain analysis procedures for CP Co Design Class 1 piping were revised. (See Subsection 5.10.3.1.3.) The basis for the revision was discussed in Reference 8. The USNRC responded to the Reference 8 letter in Reference 9.

The revised seismic input and analysis criteria are to be employed for all new CP Co Design Class 1 piping systems. In addition, the revised seismic input may be used to evaluate any existing systems as an alternate analysis and design criteria.

The revised seismic input and analysis criteria is intended to reflect more recent knowledge of piping system design and seismic response. The change was introduced in order to accommodate the greater allowable seismic damping values proposed for piping in ASME Section III Code Case N-411. In order to employ these higher damping values, it was necessary to include the special considerations for that code case as detailed in USNRC Regulatory Guide 1.84 Rev 24 (Reference 10). The special considerations which CP Co has agreed to abide by (Reference 10) are:

1. Use of a broad-band seismic input spectra for the piping. For the case of the Palisades Plant, the response spectra of NUREG/CR-1833 (Reference 11) were agreed upon as suitable to meet that requirement.

2. Use of USNRC Regulatory Guide 1.92 (Reference 4) for combining by SRSS the colinear responses over the three directions of input and for grouping closely-spaced modes by the 10% rule.

3. Use of USNRC Regulatory Guide 1.61 (Reference 12) for damping values for equipment which is included as part of the piping model.

4. Use of a zero-period-acceleration correction factor to account for frequency response due to frequencies greater than 33 Hz which were not included directly in the modal response.

The zero-period-acceleration correction factor will be that incorporated into computer programs used to perform pipe stress analysis. In the revised seismic analysis criteria, modal analysis is done up to 33 Hz rather than the 20 Hz which was typical in original analyses. Correction for the missing mass is made as a single mode to be combined with the others by square root sum of squares.

The response spectra for a given building elevation and loading direction drawn from the seismic response spectra of NUREG/CR-1833 and Code Case N-411 damping values are shown in Reference 16.

5.7.5 SEISMIC ANALYSIS OF MAJOR CP CO DESIGN CLASS 1 SYSTEM AND COMPONENTS

The methods of analysis for major CP Co Design Class 1 systems and components are described in Subsections 5.7.5.1 and 5.7.5.2. Each horizontal and vertical acceleration was applied simultaneously in separate load cases (ie, N-S + vertical and E-W + vertical). The worst case was used for design.

5.7.5.1 Primary Coolant System

The analysis of the main loop of the Primary Coolant System, performed by Combustion Engineering, Inc (vendor), was based on an equivalent static load method. The seismic accelerations were established by multiplying the peak value of the ground design spectrum by the ratio of the maximum floor and maximum ground accelerations. Damping was 0.5% for piping and 2.0% for components. A flexibility model of the entire main loop, including piping, components and supports, was created. Inertia loads were applied at the center of gravity of the component and as a distributed load along the piping. The model was analyzed using the MEC21 computer code developed by the Mare Island naval shipyard. The calculated force and moment reactions at particular points of interest, such as component nozzles and component supports, were tabulated and included in the appropriate component specifications.

In order to assess the suitability of this method of analysis, Bechtel Power Corp (A/E) subsequently performed simplified dynamic analyses of the individual components to obtain their dynamic characteristics. The components were modeled dynamically as single degree of freedom systems in each of the three directions with masses concentrated at the center of gravity. Due to the massiveness of the concrete internal structures, dynamic coupling between structures and components was ignored. Fundamental natural frequencies for each principal direction were estimated using the static deflection approach. The lowest computed natural frequency was used to determine an acceleration from the appropriate floor response spectrum. In all cases, the natural frequencies were greater than the frequencies at the spectrum peaks, and the resulting accelerations were slightly higher than those used in the equipment specifications. A subsequent review of the frequency calculations using less conservative assumptions, resulted in higher frequencies and lower accelerations. These revised accelerations were lower than the equipment specification values.

Pipe rupture loads and seismic loads were considered simultaneously in the design of supports for the Primary Coolant System components (reactor vessel, steam generators and primary coolant pumps). The rupture loads were much larger than the seismic loads. Design of these supports is discussed in Subsection 5.9.2.3.

5.7.5.1.1 Reactor Vessel Assembly

The reactor vessel is supported by an extremely rigid structure whose acceleration was estimated to be virtually the same as the concrete mass from which it is supported. The reactor vessel was designed for OBE accelerations of 0.26 g horizontally and 0.17 g vertically, and SSE accelerations of 0.468 g horizontally and 0.312 g vertically. The reactor vessel nozzles were designed for the same vertical accelerations as the vessel, but the horizontal accelerations were 1.0 g and 1.8 g for the OBE and SSE, respectively.

The control rod drive mechanism was designed based on horizontal accelerations of 0.76 g for the OBE and 1.35 g for the SSE assuming 0.5% damping and a lateral seismic support. The control rod drive mechanism housings and incore instrumentation housings were designed for OBE accelerations of 0.975 g horizontally and 0.104 g vertically, and the SSE accelerations of 1.72 g horizontally and 0.21 g vertically. The control rod shrouds were designed for a 1.4 g horizontal SSE acceleration.

5.7.5.1.2 Steam Generators

The fundamental natural frequency of the steam generator was computed assuming it was supported only at the bottom as a cantilever. This was a very conservative approach since the upper horizontal support provides restraint during an earthquake. The actual frequency was estimated to be at least 30% higher than the computed value; the horizontal and vertical accelerations corresponding to this frequency were lower than those specified for the design. The steam generators were designed for horizontal and vertical OBE accelerations of 0.2 g.

5.7.5.1.3 Primary Coolant Pumps

The primary coolant pumps are supported directly on very rigid frames from the floor beneath them. It was assumed that they would experience the same acceleration as the supporting floor.

The acceleration of this floor is lower than the specified SSE horizontal and vertical accelerations of 0.55 g.

5.7.5.1.4 Pressurizer

The pressurizer was designed for horizontal and vertical OBE accelerations of 0.2 g, and horizontal and vertical SSE accelerations of 0.4 g.

5.7.5.1.5 Primary Coolant System Piping

The fundamental natural frequencies for each principal direction of the most flexible main coolant piping run, even without the pump support, were found to be much higher than the floor response spectra peak value frequencies. Since the pump support increases the piping system stiffness, the piping natural frequencies would increase, resulting in lower spectral accelerations.

5.7.5.1.6 Pressurizer Quench Tank

The pressurizer quench tank was designed for OBE accelerations of 0.35 g horizontally and 0.24 g vertically, and SSE accelerations of 0.50 g horizontally and 0.33 g vertically.

5.7.5.1.7 Pressurizer Safety and Power-Operated Relief Valves

The pressurizer safety valves and the power-operated relief valves (PORVs) have been qualified by test as part of the NUREG-0737, Item II.D.1 requirements. However, as discussed in Subsection 4.3.9, the PORVs have been blocked out of service.

5.7.5.2 Other Major CP Co Design Class 1 Systems and Components

Major systems and components are identified in Chapters 4, 6, 9 and 10. Seismic design accelerations for selected originally purchased components are summarized in Table 5.7-6.

Components were classified according to fundamental natural frequency and dynamic degrees of freedom, and an analysis was performed as discussed below. Most components were analyzed using Method 1.

1. A component with a fundamental natural frequency greater than 33 Hz was classified as rigid, and a static analysis was performed using the high-frequency asymptote of the floor response spectrum.

2. A component with a fundamental natural frequency less than 33 Hz and a single significant dynamic degree of freedom was analyzed statically using the spectral acceleration at the equipment frequency for the appropriate equipment damping.

3. A component with a fundamental natural frequency less than 33 Hz and multiple degrees of freedom was analyzed by the modal response spectrum method.

4. If the fundamental frequency of the component was unknown and the building analysis was available, the peak value from the floor response spectrum was used in a static analysis. However, if the building analysis was not available, a conservative acceleration was assumed for use in a static analysis.

5.7.6 SEISMIC ANALYSIS OF SPENT FUEL STORAGE RACKS

5.7.6.1 Region 1 Racks

In 1977 the spent fuel storage capacity was increased by replacing the original racks with higher density racks which are designated Seismic Category I per NRC Regulatory Guide 1.29 (see Subsection 9.11.3.2). The analysis of these new racks is presented in this subsection; it satisfies the requirements of NRC Standard Review Plan (SRP), Section 3.8.4.

A three-dimensional beam and plate finite element model was developed for the analysis of the spent fuel storage rack under the load combinations presented in Reference 6. Because it was assumed that thermal expansion closed the gaps between adjacent racks, it was necessary to model all racks along one fuel pool direction. The long pool direction was chosen and the forces resulting from this analysis were assumed to be applicable in the short pool direction. A typical 8-foot by 8-foot "C" rack was modeled in detail using 1/2 symmetry while the adjacent racks were modeled as longitudinal grid beams and concentrated weights. The model assumed support at the walls and rack feet. For the typical rack, the stiffness of the fuel assemblies was neglected; however, the mass of the fuel assemblies and an effective mass of water were considered to be uniformly distributed along the fuel cans.

A modal response spectrum analysis of the fuel racks was performed using the STARDYNE computer program. This dynamic analysis yielded a fundamental lateral frequency of 12 Hz, participation factor of 1.8 and an SSE spectral acceleration of 0.567 g. The associated mode shape is characterized by compression of the grids and bending of the cans. Higher modes were not considered due to small participation. Both the OBE and SSE analyses assumed 4% modal damping based upon 2% for steel-framed structures plus 2% for the surrounding water. The fundamental vertical frequency was 36.8 Hz, which is in the rigid spectrum range; therefore, a static analysis was performed with a 0.133 g SSE spectral acceleration.

During a seismic event, the fuel assembly will move inside the can due to the existence of a 1/8-inch gap between the fuel assembly and the can. The effect of this motion, termed fuel/can interaction, was considered. A non-linear dynamic analysis of a single can and fuel assembly was performed using the ANSYS computer program. The can and the fuel assembly were modeled by beam finite elements, separated by nonlinear gap elements. The can was restrained at the upper and lower grid elevations. The fuel, which was assumed to be pinned at its base, was given an initial velocity relative to the can, and impact loads were determined as a function of time. Maximum shear forces and bending moments were then computed at critical sections of the can.

For each horizontal direction, stresses computed using the dynamic analysis results were reduced by the ratio of rack mass without fuel to the rack mass with fuel and combined with the corresponding fuel/can interaction stresses by the SRSS method. This combination method was considered appropriate since the two phenomena are statistically independent. The resulting stresses for the two horizontal directions (same) and the vertical direction were then combined by the SRSS method. All critical section, SSE load combination stresses were within allowables with margins of safety ranging from 1.02 to 1.19. OBE combination stresses were not calculated since OBE impact loads would be approximately 1/4 to 1/3 as large as SSE loads. In addition, the maximum can wall buckling stress (no fuel/can interaction) was 14,170 psi versus 20,380 psi allowable for a 1.44 margin.

In 2013, the existing Region 1 racks in the Main Spent Fuel Pool that contained Carborundum® were removed and replaced with new Region 1 racks that utilize Metamic™ as the neutron absorber material. The Region 1 Metamic™ racks have the same number of storage cells per rack as their predecessors. Thus the total storage capacity of the Spent Fuel Pool was unchanged. The Region 1 Carborundum® rack in the North Tilt Pit was not replaced.

The seismic response of Metamic™ Region 1 racks was analyzed using the computer code DYNARACK, which is a non-linear time history simulation code. In DYNARACK, each Region 1 rack was individually modeled as a freestanding body having 12 degrees of freedom, capable of rocking, sliding, and twisting under the seismic forces. The spent fuel assemblies stored inside each Region 1 rack were collectively modeled as five lumped masses equally spaced along the height of the rack cell region. Each lumped fuel mass had 2 horizontal degrees of freedom, and it interacted with the rack through an arrangement of fuel-to-rack impact springs. The fluid coupling effects between the lumped fuel masses and the storage cell, as well as between adjacent Region 1 racks, were also accounted for in the DYNARACK model. The calculated stresses in the Region 1 racks at the critical cross sections were compared against the stress limits specified in Appendix D of SRP Section 3.8.4.

5.7.6.2 Region 2 Racks (See Section 9.11)

In 1987 the spent fuel storage capacity was increased from 798 to 892 fuel assemblies (Reference 15). The newly designated Region 2 racks are designed to Seismic Category I requirements and are classified as ANS Safety Class 3 and ASME Code Class 3 Component Support Structures. The racks are designed in accordance with the NRC "OT Position for Review and Acceptance of Spent Fuel Storage and Handling Applications," (Reference 14) and Standard Review Plan, Section 3.8.4. The structural evaluation and seismic analyses were performed using the loads and load combinations specified in Section IV-4 of the OT Position Paper.

The seismic and stress analysis of the spent fuel rack modules considered the various conditions of full, partially filled, and empty fuel assembly loadings. The racks (Region 2) were evaluated for both Operating Basis Earthquake (OBE) and Safe Shutdown Earthquake (SSE) conditions and meet Seismic Category I requirements. A detailed stress analysis was performed to verify the acceptability of the critical load components and paths under normal and faulted conditions. The racks rest freely on the pool floor and were evaluated to determine that under all loading conditions, they do not impact each other, nor do they impact the pool walls or the existing Region 1 racks. Additional analysis was performed to determine if modification to the Region 1 racks was required to prevent their impacting the Region 2 racks and resulted in the determination that no modification was necessary.

The dynamic response of the fuel rack assembly during a seismic event is the condition which produces the governing loads and stresses on the structure. The seismic analysis of a free-standing fuel rack is a time history analysis performed on a nonlinear model. The time history was performed on a single cell nonlinear model with the effective properties of an average cell within the rack module. The effective single cell properties were obtained from a structural model of the rack modules. The structural model is a finite element representation of the rack assembly consisting of beam elements interconnected at a finite number of nodal points, and general mass matrix elements.

The nonlinear model was run with simultaneous inputs of the vertical and the most limiting horizontal acceleration time history values. The damping values used in the seismic analysis are 2% damping for OBE and SSE. In addition, the model was run for a range of friction coefficients (0.2 and 0.8) to obtain the maximum values. The results from these runs were fuel-to-cell impact loads, support pad loads, support pad liftoff, rack sliding, and fuel rack structure internal loads and moments. Maximum values were obtained by utilizing the full time histories. The internal loads and stresses from the seismic model were adjusted by peaking factors from the structural model to account for the stress gradients through the rack module. Consequently, the maximum loaded rack components of each type was analyzed. Such an analysis enveloped the other areas of the rack assembly. The maximum stresses from each of the three seismic events were combined by the SRSS

method. In addition, the results were used to determine the rack response for full, partially filled, and empty rack module loading conditions.

5.7.7 SEISMIC ANALYSIS AND TESTING OF OTHER CP CO DESIGN CLASS 1 COMPONENTS

Components procured prior to January 1979 were either analyzed as described in Subsection 5.7.5.2 or tested for qualification acceleration levels exceeding the specification levels. Most components were either proven or assumed to be seismically rigid.

Components procured after January 1979 were analyzed or tested in accordance with one of several Bechtel specifications. A comparison of the requirements imposed by each specification is shown in Table 5.7-5. These specifications all contain the following requirements in common:

1. Only the first three analytical methods described in Subsection 5.7.5.2, which require identification of component fundamental natural frequency, were used. A component with a natural frequency exceeding 33 Hz was considered to be rigid.

2. The minimum variation in natural frequency due to uncertainties in material and geometric properties was ± 10%. The worst spectral acceleration for this range was used.

3. Combination of modes for flexible multidegree of freedom systems was in accordance with the NRC Regulatory Guide 1.92 "Grouping" method (Reference 4).

4. Testing was performed in accordance with IEEE Standard 344-1975 (Reference 7) using the required response spectrum provided with the specification. In all cases, the Test Response Spectrum (TRS) was required to envelope the Required Response Spectrum (RRS).

5. Floor response spectra were provided in the specification for the OBE and doubled to obtain SSE values.

After 1988, all modifications and replacements of Seismic Category I equipment may be procured and qualified in accordance with the following standards which are applicable to the type of equipment.

1. IEEE Standard 344-1975 and RG 1.100, Rev 1.

2. IEEE Standard 382-1980 for qualification of valve actuators.

3. ANSI C16.41-1983 for qualification of rigid valve actuators and valve assemblies.

4. ANSI C37.98-1978 for qualification of relays, starters and similar equipment.

5. IEEE Standard 649-1980 for qualification of motor control centers.

6. Material damping ratios for analysis are in accordance with Table 5.7-2 of this chapter.

7. Load, load combinations and allowable stress for analysis are in accordance with Subsection 5.10.1.3.

8. Modal response summation using the response spectrum analysis method is in accordance with RG 1.92, Rev 1.

9. Equipment response due to three components of earthquake motion in a dynamic analysis are summed using the RG 1.92, Rev 1 methods; or, the larger response of each horizontal analysis is combined separately with the vertical analysis response in accordance with Section 5.7.2.

10. Modal testing methods such as the normal mode method or the transfer function method may be used to determine the resonant frequencies, mode shapes, modal damping, etc, of the dynamic characteristics of a complex structure.

11. The use of power spectral density (PSD) analysis to evaluate the response of a piece of equipment at a certain location may be used if determined applicable and approved by CP Co.

12. In lieu of the above-mentioned methods, verification of equipment ruggedness may be developed using seismic experience, in accordance with the methodology developed by the Seismic Qualification Utility Group (SQUG) and as validated by the NRC Safety Evaluation Report for the SQUG Generic Implementation Procedure (GIP) Revision 2. NRC approval of the use of the GIP was granted in References 19 and 20.

13. Input motion is determined in accordance with the ground response spectra delineated in Subsection 5.7.1.2. The floor response spectra is developed using the ground response spectra as described in Subsection 5.7.1.3.

5.7.7.1 Electrical Equipment and Instrumentation

Class 1E electrical equipment and instrumentation purchase specifications seismic acceleration levels are shown in Table 5.7-7. The definition of Class 1E electrical equipment and instrumentation is provided in Subsection 8.1.1.

A sufficient amount of conservatism was incorporated into some equipment specifications to preclude malfunctions due to seismic loads. For example, the Reactor Protective System was qualified for a 0.8 g horizontal seismic acceleration. This is well above the specification level.

As part of the Nuclear Regulatory Commission's (NRC) Systematic Evaluation Program (SEP), the anchorage and support of safety-related electrical components were investigated in 1980 for adequacy during an SSE event and modifications were subsequently made as necessary (see Subsection 5.10.3.3).

5.7.7.2 Tanks

Tanks associated with major CP Co Design Class 1 systems are mentioned in Subsections 5.7.5.1.6 and 5.7.5.2.

Original purchase specification design acceleration levels for tanks associated with other CP Co Design Class 1 systems are given in Table 5.7-8.

The diesel generator fuel oil day tank was evaluated as part of SEP, and it was determined that modifications were required. The principal changes consisted of grouting between each tank and the concrete walls on three sides, and adding stiffening beams on the fourth side to increase ability of the tank walls to resist seismic loads.

5.7.7.3 Appendages to CP Co Design Class 1 Components

Appendages to CP Co Design Class 1 piping are discussed in Subsection 5.7.4.

Appendages (small masses elastically attached to large masses) to CP Co Design Class 1 components were analyzed dynamically using the response spectrum curves for the point of attachment. Their weight was included in the analysis of the large mass to which they are attached.

5.7.7.4 Overhead Cranes

For the containment polar crane and the spent fuel pool crane, horizontal motion is restrained by the flanged wheels on both the bridge and trolley. For the spent fuel pool crane, vertical upward acceleration is insufficient to displacement the bridge and the trolley (References 26 and 27). For the containment polar crane, vertical upward movement is not restrained by any positive means. However, Reference 25 concluded that the “uplifters” on the containment polar crane are not required.

5.7.7.5 Containment Air Locks

The personnel air lock and the escape air lock were considered as rigid bodies and designed for OBE accelerations of 0.25 g horizontally and 0.067 g vertically. These values equal or exceed the OBE accelerations of the containment shell at the lock locations. The resulting stresses were combined with those for dead load, and internal or external pressure as applicable, with the total stress being kept within the allowables of the ASME Boiler and Pressure Vessel Code, Section III, Subsection B. The air lock deformations were checked for twice the OBE values to ensure that SSE accelerations would not cause a loss of leak tightness.

5.7.8 SEISMIC ANALYSIS OF BURIED STRUCTURES AND COMPONENTS

There are no CP Co Design Class 1 buried tunnels at the Palisades Plant, except the containment access gallery which is rigidly connected throughout its length to the containment base slab and is separated from other structures by soil backfill.

Electrical cables routed underground are pulled in plastic conduits which are encased in concrete. Although the concrete might crack during a seismic event, the cables would not be damaged due to the flexibility of the conduit and cable. In addition, the conduits are not fully filled with cable which permits the cable to move without being damaged.

At penetrations, CP Co Design Class 1 buried piping has been designed to accommodate horizontal differential movement between the building and the soil.

5.7.9 SEISMIC INSTRUMENTATION

A triaxial strong-motion accelerograph has been installed at the Plant site to record any significant seismic events. It is located in the Training Building and measures ground surface motion in the free field.

REFERENCES

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2. "Building Interaction Effects," P A Parmelee, Engineering Mechanics Specialty Conference (ASCE), October 12, 1966

3. "Seismic Analysis of Structures and Equipment for Nuclear Power Plants," Bechtel Topical Report BC-TOP-4-A, Revision 4, August 1980

4. NRC, Regulatory Guide 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," Revision 1, February 1976

5. "Truss Modification Analysis," EDS Report 02-0660-1087, Addendum 2

6. Spent Fuel Pool Modifications, letter from D A Bixel (CP Co) to A Schwencer (NRC), dated February 8, 1977, response to Question S6, (2510/0390)

7. "IEEE Recommended Practices for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations," IEEE Standard 344-1975

8. Letter from J L Kuemin (CP Co) to Director, Nuclear Reactor Regulation, "Modification of the Seismic Design Criteria To Incorporate ASME Code Case N-411," July 28, 1986

9. Letter from A C Thadani (NRC) to K W Berry (CP Co), "Modification of the Palisades Seismic Design Criteria To Incorporate ASME Code Case N-411," October 20, 1986

10. USNRC Regulatory Guide 1.84, "Design and Fabrication Code Case Acceptability - ASME Section III Division 1," Revision 24, June 1986

11. NUREG/CR-1833, "Seismic Review of the Palisades Nuclear Power Plant Unit 1 as Part of the Systematic Evaluation Program," January 1981

12. USNRC Regulatory Guide 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," October 1973

13. ASME Boiler and Pressure Vessel Code Case N-411, "Alternative Damping Values for Seismic Analysis of Classes 1, 2 and 3 Piping Systems, Section III, Division 1"

14. "OT Position for Review and Acceptance of Spent Fuel Storage and Handling Applications," USNRC, January 18, 1979

15. Facility Change, FC-680, Rerack of Spent Fuel Pool, (Cartridges 9576 - 9579)

16. Palisades Specification M-195(Q), "Requirements for the Design and Analysis of Palisades Plant Safety Related Piping and Instrument Tubing"

17. Letter from Bechtel to W E Garrity (CP Co), "Subject: Civil Calculations," April 29, 1991

18. Deleted

19. Letter from A Masciantonio, NRC, to G B Slade, CP Co, "Safety Evaluation of the Palisades Plant 120-Day Response to Generic Letter 87-02, Supplement 1", dated November 23, 1992

20. Letter from J G Partlow, NRC, to A-46 Plant Licensees who are SQUG members, "Supplement No. 1 to Generic Letter (GL) 87-02 that Transmits Supplemental Safety Evaluation Report No. 2 (SSER No. 2) on SQUG Generic Implementation Procedure, Revision 2, as Corrected on February 14, 1992 (GIP-2)", dated May 22, 1992

21. Palisades Specification C-175(Q), "Requirements for Seismic Evaluation of Electrical and Mechanical Equipment"

22. EA-POC0007899-RG160SSI, "Palisades RG 1.60 Floor and Piping Response Spectra with Soil Structure Interaction"

23. EA-POCOO23368-RG160SAM, "Seismic Anchor Movements for New Palisades RG 1.60 Spectra with SSI"

24. Letter from RGSchaaf, NRC, to NLHaskell, Consumers Energy, "Palisades Plant - Resolution of Unresolved Safety Issue (USI) A-46, Verification of Seismic Qualification of Equipment in Operating Plants," dated September 25, 1998

25. EA-EAR-99-0347-01, ”Assess Need For Reactor Building Polar Crane L-1 Seismic Keepers”

26. EA-FC-976-09, “Evaluation of the L-3 Crane Upgrade and Installation”

27. EA-FC-976-03, “Bridge Wheel Loads Calculation”

5.8 CONTAINMENT STRUCTURE

5.8.1 DESIGN BASIS

The containment structure completely encloses the Primary Coolant System in order to minimize the release of radioactive material to the environment should a serious failure of the Primary Coolant System occur. Therefore, the containment structure is designated a CP Co Design Class 1 structure. The structure provides adequate biological shielding during both normal operation and accident situations. The containment structure is designed to ensure that leakage will not exceed 0.1% per day by weight at a design pressure of 55 psig and a design temperature of 283°F.

The principal design basis for the structure is that it be capable of withstanding the internal pressure resulting from the Design Basis Accident (DBA) with no loss of integrity. The DBA assumes that the total energy contained in the water of the Primary Coolant System is released into the containment through a double-ended break of the largest primary coolant pipe coincident with a loss of normal and standby electrical power. Subsequent pressure behavior is determined by the Engineered Safeguards Systems and the combined influence of energy sources and heat sinks.

Energy is available for release into the containment structure from the following sources:

NSSS Stored Heat
Reactor Core Decay Heat
Metal-Water Reactions
Hydrogen Combustion

Transients resulting from the DBA and other lesser accidents are presented in Chapter 14 and serve as the basis for the containment design pressure of 55 psig and the design temperature of 283°F. Although the containment response analyses in Section 14.18 predict peak transient temperatures that exceed the 283°F design temperature, these peaks have a negligible effect on the containment structure (Reference 28). The containment liner has been analyzed for liner temperatures up to 410°F for a design basis accident (Reference 49).

The external design pressure of the containment shell is 3 psig. This value is approximately 0.5 psig beyond the maximum external pressure that could be developed if the containment were sealed during a period of low barometric pressure and high temperature and, subsequently, the containment atmosphere were cooled with a concurrent rise in barometric pressure. Vacuum breakers are not provided.

The design of the Engineered Safeguards Systems and their operation are discussed more fully in Chapter 6; only their relation to the basis of containment design is discussed below. Four Engineered Safeguards Systems are provided to limit the consequences of the DBA. Their energy removal capabilities limit the internal pressure after the initial peak so that containment design limits are not exceeded and the potential for release of fission products is minimized.

The Safety Injection System injects borated water into the reactor vessel to remove core decay heat and to minimize metal-water reaction and the associated release of heat and fission products. Flashed primary coolant, NSSS sensible heat and core decay heat transferred to containment are removed by two Engineered Safeguards Systems: the Containment Spray and/or the containment air recirculation cooling systems.

The Containment Spray System removes heat directly from the containment by cold water quenching of the containment steam atmosphere and subsequent heat removal by recirculation of the containment sump water through the shutdown cooling heat exchangers.

The Containment Air Cooling System removes heat directly from the containment atmosphere to the Service Water System with recirculating fans and cooling coils.

5.8.2 GENERAL DESCRIPTION

The containment structure is a CP Co Design Class 1 structure. The containment structure consists of a post-tensioned, reinforced concrete cylinder and dome connected to and supported by a massive, reinforced concrete foundation slab as shown in Figure 5.8-1. The entire interior surface of the containment structure is lined with 1/4-inch-thick welded ASTM A-442 steel plate to ensure a high degree of leak tightness. Numerous mechanical and electrical systems penetrate the containment wall through steel penetrations which are welded to the containment liner plate (see Figures 5.8-2 and 5.8-3). For the steam generator replacement construction opening description see Section 5.8.9.

Principal dimensions are as follows:

Inside Diameter	116 Feet
Inside Height (Including Dome)	189 Feet
Vertical Wall Thickness	3-1/2 Feet
Dome Thickness	3 Feet
Foundation Slab Thickness	8-1/2 to 13-1/2 Feet
Liner Plate Thickness	1/4 Inch
Internal Free Volume	1,640,000 ft^3

In a post-tensioned containment, the internal pressure load on the shell is approximately balanced by the prestressing forces which act in a manner similar to an external pressure load. At the Palisades Plant sufficient prestressing forces are applied to the cylinder and dome to more than balance the internal design pressure of 55 psig. The internal pressure loads on the base slab are resisted by both the external soil pressure and the strength of the reinforced concrete slab. Therefore, there is no need for prestressing tendons in the base slab.

The concrete used in the structure is made with crushed dolomitic limestone aggregate obtained from Drummond Island in northern Lake Huron. Such aggregate produces an excellent high strength, dense, sound concrete. The design strengths are 5,000 psi at 28 days for the shell and 4,000 psi at 90 days for the base slab.

ASTM A-432 reinforcing steel, mechanically spliced using T-series CADWELDS, is used throughout the base slab and around the equipment hatch penetration to resist membrane and flexural forces. A-432 steel is also used to resist base slab shear forces. Throughout the shell, ASTM A-15 reinforcing steel is provided to control cracking. At major geometric discontinuities, where analysis indicated that additional reinforcement was required for membrane and flexural forces, A-432 steel is generally used to provide additional elastic strain capability. Elsewhere, this additional reinforcement is A-15 steel. A-15 steel is also used throughout the shell to resist shear forces.

The post-tensioning system consists of:

1. Three groups of 55 dome tendons oriented at 120 degrees to each other for a total of 165 tendons anchored at the vertical face of the dome ring girder.

2. 178 vertical tendons anchored at the top surface of the ring girder and at the bottom of the base slab.

3. Six groups of hoop tendons enclosing 120 degrees of arc for a total of 502 tendons anchored at the 6 vertical buttresses.

There are two empty vertical tendon sheaths and twenty empty hoop tendon sheaths. These sheaths did not have tendons installed during initial construction due to either tendon sheath blockage or the original containment design did not require them to be installed. Empty tendon sheaths are: V142, V208, AC4, AC5, AC6, BD3, BD4, BD5, FB4, FB5, FB6, CE2, CE5, CE6, DF4, DF5, DF6, EA3, EA4, EA6, EA20, and EA78 (Ref D-Pal-90-050).

The general tendon arrangement in the cylinder and dome is shown in Figure 5.8-1. The deflection of tendons in the vicinity of the equipment hatch is illustrated in Figure 5.8-4.

The prestressing system employed is the BBRV system as furnished by the Inland-Ryerson Construction Products Company. Each tendon consists of 90 ASTM A-421, 1/4-inch-diameter, buttonheaded wires. The tendons are housed in spirally wound, corrugated, thin wall sheathing which is attached to mild steel "trumplates" (trumpet plus bearing plate) and capped at each anchorage by a pressure-tight sheathing filler cap. The prestressing load is maintained by using split tube shims under all anchor heads. Some of the prestressing system hardware (sheathing, trumplates, anchor heads, shims) is shown in Figure 5.8-5.

After fabrication, the tendon was shop dipped in a petrolatum corrosion protection material, bagged and shipped. After installation, the tendon sheathing and caps were filled with a corrosion preventative grease.

The 1/4-inch-thick liner plate is stitch welded to a gridwork of structural steel angles embedded in the concrete. The details of the anchoring system are provided in Figure 5.8-1. The anchoring system is designed to prevent significant distortion of the liner plate during accident conditions and to ensure that the liner maintains its leak-tight integrity. The liner plate has been coated on the inside with 4-1/2 mils of inorganic zinc paint for corrosion protection. There is no paint on the side in contact with the concrete.

Personnel and equipment access to the structure is provided by a personnel air lock with 2 - 3-foot 6-inch x 6-foot 8-inch doors, an escape air lock with 2 - 30-inch-diameter doors, and an equipment hatch with a single door that provides total access to the 12-foot-diameter passageway (see Figures 5.8-6 through 5.8-8). The air locks and hatch were fabricated from ASTM A-516, Grade 70, firebox quality steel, made to the requirements of SA-300, Charpy V-notch tested at a temperature of 0°F, and conforming to the requirements of the ASME Boiler and Pressure Vessel Code, Section III.

The structural brackets provided for the containment crane runway and for the dome liner erection trusses were fabricated of ASTM A-36 steel shapes and ASTM A-516, Grade 70, insert reinforcing plates (see Figure 5.8-1). All structural brackets and reinforcing plates were shop fabricated and stress relieved as completed assemblies and then shipped to the job site for welding into the 1/4-inch liner plate.

Upon completion of initial testing of the containment, the anchorages at the ring girder and the buttresses were enclosed by corrugated aluminum siding chosen to coordinate the architectural appearance of the containment with the balance of the Plant. This siding is removable to permit access to the tendons for the surveillance program discussed in Subsection 5.8.8.3. No siding was provided for tendons that terminate within adjacent buildings or within the access gallery under the containment wall.

The original Plant design included a 3-part covering for the containment dome. First the dome was covered by 1/2 inch to 3/4 inch of sprayed-on urethane foam. Then the foam was covered by 1 coat of neoprene followed by 3 coats of hypalon. Together, these 2 parts formed a membrane approximately 20 mils thick. During 1981 this covering was removed because the foam had become impregnated with water. Currently, the dome has an elastomeric coating.

An impressed current cathodic protection system utilizing close-coupled anodes was provided in the original design of the Plant to protect the liner plate, reinforcing bars and tendon sheathing in the base slab. Subsequent to Plant start-up, it was determined that this system is not required and can be disconnected.

The following statements support this conclusion:

1. Since all the steel elements mentioned above are embedded in concrete, they have no contact with the soil.

2. The Plant is situated on a uniformly graded dune sand which extends to a depth of 25 feet to 30 feet below the top of the containment base slab. This sand is inherently noncorrosive. Other Consumers Power Company Plants are sited on similar soil and they have experienced no corrosion problems even though they do not have a cathodic protection system.

3. Recordings of the currents taken by the soil over the years indicate that the soil has a very low conductivity.

4. Buried metals at the Plant have very little dissimilarity.

5.8.3 LOADS AND LOAD COMBINATIONS

5.8.3.1 Containment Structure Concrete

5.8.3.1.1 Construction Condition

During the construction period, prior to the post-tensioning operation, the containment structure was designed for dead load, live load (including construction loads), and a reduced wind load.

In addition to the above loads, the containment structure was designed for loads developed due to the post-tensioning operation via the load combination $D + F_i$ (F_i is defined in Subsection 5.8.5.3.1).

5.8.3.1.2 Working Stress Condition

The containment structure was designed for the following loads and load combinations using the working stress criteria presented in Subsection 5.8.5.2.3.

1. $D + F_f + L + T_o$

2. $D + F_f + L + P + T_a$

3. $D + F_f + L + P'$

Where:

D = dead loads

L = live loads

F_f = final prestress loads (defined in Subsection 5.8.5.3.1)

P = DBA pressure load

T_o = thermal loads due to operating temperature

T_a = thermal loads due to DBA temperature

P' = test pressure (1.15 x DBA pressure)

5.8.3.1.3 Yield Strength Condition

The containment structure was designed for the loads and load combinations shown below using the yield strength design criteria presented in Subsection 5.8.5.2.4. Design loads were increased by load factors to reflect built-in conservatism for loads subject to large variations and to reflect the probability that each load would occur simultaneously with certain other loads in specified load combinations. In some instances, this resulted in a minimum emphasis on the fixed gravity loads and a maximum emphasis on the accident, earthquake and wind loads. The load factor approach, together with the design requirements of Subsection 5.8.5.2.4, assures a structure with a low strain elastic response.

The final design of the containment structure satisfied the following equations:

1. $Y = 1/\Phi\ (1.05D + 1.5P + 1.0T_a + 1.0F_f)$

2. $Y = 1/\Phi\ (1.05D + 1.25P + 1.0T_a + 1.25H + 1.25E + 1.0F_f)$

3. $Y = 1/\Phi\ (1.05D + 1.25H + 1.0R + 1.0F_f + 1.25E + 1.0T_o)$

4. $Y = 1/\Phi\ (1.05D + 1.0F_f + 1.25H + 1.25W' + 1.0T_o)$

5. $Y = 1/\Phi\ (1.0D + 1.0P + 1.0T_a + 1.0H + 1.0E' + 1.0F_f)$

6. $Y = 1/\Phi\ (1.0D + 1.0H + 1.0R + 1.0E' + 1.0F_f + 1.0T_o)$

Where:

Y = required yield strength of the structure (see discussion in Subsection 5.8.5.2.4).

Φ = yield capacity reduction factor (see discussion in Subsection 5.8.5.2.4).

Φ = 0.90 for reinforced concrete in flexure.

Φ = 0.85 for shear (diagonal tension), bond and anchorage in reinforced concrete.

Φ = 0.75 for spirally reinforced concrete compression members.

Φ = 0.70 for tied reinforced concrete compression members.

Φ = 0.90 for fabricated structural steel embedded in concrete.

Φ = 0.90 for reinforcing steel in direct tension.

Φ = 0.90 for welded or mechanical splices of reinforcing steel.

Φ = 0.85 for lap splices of reinforcing steel.

Φ = 0.95 for prestressing tendons in direct tension.

D = dead load of structure and equipment plus any other permanent loads contributing stress, such as soil or hydrostatic. In addition, a portion of the "live load" is added when it includes items such as piping, cable and trays, suspended from floors. An allowance is also made for future permanent loads.

P = DBA pressure load.

F_f = final prestress loads (defined in Subsection 5.8.5.3.1).

R = force and/or pressure on structure due to rupture of any one high-energy line. The following rupture loads are included, as appropriate: pipe reactions, jet impingement and pipe whip.

H = force on structure due to thermal expansion of pipes under design conditions.

T_o = thermal loads due to operating temperature.

T_a = thermal loads due to DBA temperature.

E = OBE loads resulting from a ground surface acceleration of 0.1 g.

E' = SSE loads resulting from a ground surface acceleration of 0.2 g.

W'= tornado loads (wind pressure and differential pressure).

Equation 1 assures that the containment structure has the capacity to withstand a pressure load at least 50% greater than the DBA pressure load.

Equation 2 assures that the containment structure has the capacity to withstand a pressure load at least 25% greater than the DBA pressure load coincident with seismic loads 25% greater than those calculated for the OBE.

Equation 3 assures that the containment structure has the capacity to withstand seismic loads 25% greater than those calculated for the OBE coincident with the rupture of any attached piping due to this earthquake.

Equation 4 assures that the containment structure has the capacity to withstand tornado loads 25% greater than those calculated.

Equation 5 assures that the containment structure has the capacity to withstand the DBA pressure and DBA temperature coincident with the SSE loads.

Equation 6 assures that the containment structure has the capacity to withstand the SSE loads coincident with the rupture of any attached piping due to this earthquake.

5.8.3.2 Liner Plate System

5.8.3.2.1 Liner Plate

The liner plate was designed for strain compatibility with the shell for applicable containment structure loads. Also, the following fatigue loads were considered in the design of the liner plate:

1. Thermal cycling due to annual outdoor temperature variations. The number of cycles for this loading was 40 cycles for the Plant life of 40 years. However, the impact of outdoor temperature variations upon liner plate stresses, insulated by the 3'6" concrete containment wall, is negligible in comparison with the stresses caused by the design basis accident temperature. The annual outdoor temperature variations are not the controlling design consideration because the design loads related to accidents result in higher stress conditions (see also Section 5.8.4.3.1).

2. Thermal cycling due to containment interior temperature varying during the start-up and shutdown of the reactor system. The number of cycles for this loading was assumed to be 500 cycles.

3. Thermal cycling due to the DBA was assumed to be one cycle.

5.8.3.2.2 Liner Plate Anchors

The following loads were considered in the design of the anchorage system:

1. Dead load
2. Prestress
3. Creep and shrinkage of concrete
4. Thermal gradients (operating and DBA)
5. DBA pressure
6. Vacuum
7. OBE or SSE
8. Wind or tornado

5.8.3.3 Penetrations

Loads and load combinations for penetrations are presented in Subsection 5.8.6.3.

5.8.4 ANALYSIS

5.8.4.1 Containment Structure Concrete

5.8.4.1.1 General

The overall containment structure was analyzed for both axisymmetric and nonaxisymmetric loads. The analysis for the axisymmetric loads (dead load, live load, prestress, pressure and temperature), which employed an axisymmetric finite element model, is described in Subsection 5.8.4.1.2. The analyses for the nonaxisymmetric loads (lateral loads due to earthquake and wind) are addressed in Subsection 5.8.4.1.3.

Local regions were analyzed for the effects of tendon anchorages, penetrations and missile impact. In addition, a specialized analysis was required for the liner plate anchors. These topics are discussed in Subsections 5.8.4.2 through 5.8.4.4 with the exception of missile impact which is discussed in Section 5.5.

Buttresses

On the basis of the following observations, it was concluded that the behavior of the containment structure could be adequately modeled without the inclusion of the buttresses.

1. At each buttress, hoop tendons are either continuous or anchored. The anchored tendons are normally arranged in pairs with one tendon anchored on each face of the buttress. Each pair effectively produces a single "spliced" tendon. The resulting pattern is continuous tendons alternating with "spliced" tendons. In the buttress region associated with the "spliced" tendon, the compressive force in the concrete is approximately twice that in the region associated with the continuous tendon. Ignoring losses, the net effect is that the average compressive force in the buttress is approximately 1.5 times the prestressing force between buttresses. But the cross-sectional area of the buttress is approximately 1.5 times that of the typical wall. Therefore, the hoop stresses, as well as the hoop strains and radial displacements, can be considered nearly constant all around the structure.

2. The vertical stresses and strains caused by the vertical post-tensioning become constant at a short distance away from the anchorages because of the stiffness of the cylindrical shell. Since the stresses and strains remain nearly axisymmetric despite the presence of the buttresses, the effect of the buttresses on the analysis of the overall containment structure is negligible when the structure is under dead load or prestressing loads.

3. When an internal pressure acts upon the containment structure combined with a thermal gradient, such as during a DBA, the concrete on the outside face of the structure may crack. If cracking occurs, the effective cross section (rebar, tendons and uncracked concrete) at both the buttresses and elsewhere will be reduced, and the difference in shell stiffness between the buttress zones and the typical wall zones will be reduced. Hence, the structure will act more axisymmetric. This effect will be more pronounced at "yield strength conditions" because the pressure load is factored.

5.8.4.1.2 Axisymmetric Loads

The analysis for dead load, live load, prestress, pressure and temperature is discussed in this subsection.

The finite element technique is a general method of structural analysis in which the continuous structure is replaced by a system of elements (members) connected at a finite number of nodal points (joints). Conventional analysis of frames and trusses can be considered to be examples of the finite element method. In the application of the method to an axisymmetric solid (eg, a concrete containment structure), the continuous structure is replaced by a system of rings with a rectangular or triangular cross section. These rings are interconnected along circumferential joints to form the desired configuration. Based on energy principles, work equilibrium equations are formed in which the radial and axial displacements at the circumferential joints are unknowns of the system. The results of the solution of this set of equations are the deformation of the structure under the given loading conditions. Stresses are computed knowing the strain and stiffness of each element.

The finite element mesh used to describe the containment structure is shown in Figure 5.8-9. Because of the small scale of this figure, the liner plate appears as just a heavy line. The quadrilateral elements that form the cross section, including the liner plate, were subdivided by the computer program into four triangular constant-stress, constant-strain elements. These elements were formulated using a three-dimensional stress-strain relationship. The upper portion and lower portion of the containment structure were analyzed independently to permit a greater number of elements to be used for those areas of the structure of major interest, such as the ring girder area and the base of the cylinder. Horizontal rollers were assumed as supports for the top part and located sufficiently far below the ring girder that the stresses at the match line were undisturbed by the rollers. Static equilibrium was checked for the top part of the structure. The roller loads were then applied to the bottom part (which rests on soil) and static equilibrium was checked. At the match line for the two parts, the loads and stresses were found to agree for the various load combinations. The finite element mesh of the base slab was extended down into the foundation material to take into consideration the elastic nature of the foundation material and its effect upon the behavior of the base slab; however, this soil mesh has not been included in Figure 5.8-9. The liner plate was modeled as an integral part of the structure, but with different material properties than the concrete, and not as a mechanism which would act as an outside source to produce loading only on the concrete portion of the structure.

The major benefit of the finite element program was its capability to predict shears and moments due to internal restraint and due to the interaction of the foundation slab with the soil. The structure was analyzed assuming an uncracked homogeneous material. This is conservative because the decreased relative stiffness of a cracked section would result in smaller secondary shears and moments. If necessary, the modulus of elasticity was corrected for those elements which were stressed beyond their proportional limit to account for the nonlinear stress-strain relationship at high compression and the stresses were recomputed. The computer output included displacements at each nodal point, direct stresses and principal stresses.

The use of the finite element computer program permitted an accurate estimate of the stress pattern at various locations of the structure. The following material properties were used in the program for the various loading conditions:

$E_{\text{foundation concrete}}$ (psi)	5.0×10^6
$E_{\text{shell concrete}}$ (psi)	
Instantaneous loads (P)	5.5×10^6
Sustained loads (D, F_i, F_f, T_o, T_a)	2.7×10^6
ν concrete (Poisson's Ratio)	0.17
α concrete (coefficient of expansion) (in/in per °F)	0.5×10^{-5}
E_{soil} (psi)	0.1×10^6
E_{liner} (psi)	30×10^6
fy_{liner} (psi)	32,000(a)

(a) The actual yield stress of the liner material is 30 Ksi. However, use of the larger value is conservative in this application.

The above values were design assumptions and some material variations were both expected and permissible. In arriving at the tabulated values of E_{concrete}, the effect of creep was included using the following equation for long-term loads such as dead load, prestress and operating thermal load:

$$E_{cs} = E_{ci}\, \varepsilon_i/(\varepsilon_s + \varepsilon_i)$$

Where:

E_{cs} = sustained modulus of elasticity of concrete (psi)

E_{ci} = instantaneous modulus of elasticity of concrete (psi)

ε_i = instantaneous strain (in/in per psi)

ε_s = creep strain (in/in per psi)

No modification was made of ν for instantaneous or sustained loadings. Under DBA conditions (load combinations including P and T_a) or yield strength conditions (factored load combinations per Subsection 5.8.3.1.3), cracking of the concrete at the outside face was expected. For these conditions, the value of the sustained modulus of elasticity of concrete, E_{cs}, was used in conjunction with ACI 505-54 to find the stresses in the concrete, reinforcing steel and liner plate.

The thermal gradients used for design are shown in Figure 5.8-10. The gradients for both the design accident condition and the factored load condition are based on the temperature associated with the containment pressure. The design pressure and temperature of 55 psig and 283°F became 82.5 psig and 310°F at factored conditions, the latter being the temperature associated with the saturated steam pressure at factored conditions. A maximum calculated liner plate temperature of 266°F was used for both cases (see Curve T1).

Based upon the above data, temperature values were conservatively determined for every node point in the cross section. These values were entered into the finite element program. The resulting thermal stresses were calculated at the center of each element.

The isostress plots of the homogeneous uncracked concrete structure indicate the general stress pattern for the overall structure under various axisymmetric loading conditions. These plots (see Figures 5.8-11 and 5.8-12) show the three principal stresses and their corresponding directions. One principal stress direction always coincides with the hoop direction. The remaining two principal stress directions, which depend upon the structure's deformed shape, are normal to the hoop direction. The principal stresses provide valuable information about the behavior of the structure under these loading conditions and they were a valuable aid for the final design.

The plots were prepared by a cathode-ray tube plotter. The data for plotting were taken from the stress output of the finite element computer program for the following load combinations:

1. $D + F_i$
2. $D + F_f + 1.15P$
3. $D + F_f + 1.5P + T_a$
4. $D + F_f + T_a$

The above axisymmetric loading conditions have been found to be governing in the axisymmetric design since they produced the highest stresses at various locations of the structure. Load Combination 4 was critical for concrete stresses and occurred after depressurization of the containment. Load Combination 3 was critical for the reinforcing stresses and it occurred when pressure and thermal loads were combined; caused cracking at the outside face.

Containment structure stress resultant profiles for D, F_f, P and T_a are found in Figure 5.8-13.

5.8.4.1.3 Nonaxisymmetric Loads

Nonaxisymmetric loads include those due to seismic, wind, tornado and missiles. The seismic analysis of the containment structure is presented in Section 5.7. Wind and tornado loads are presented in Section 5.3. Credible missiles and their effect on the containment structure are discussed in Section 5.5.

5.8.4.2 Prestressing System

The overall containment structure was analyzed for prestressing loads as discussed in Subsection 5.8.4.1.2. The tendon anchorage zones, which have areas of stress concentrations created by the anchorage loads, were independently analyzed. The overall analysis was performed in two separate stages using slightly different methods. The results of the first analysis determined that additional vertical reinforcing steel was needed in the buttresses to control bursting stresses. The results of the second analysis indicated that the reinforcing steel that controls bursting stresses had acceptable stress levels. The following discussion addresses the two analyses separately.

5.8.4.2.1 Tendon Anchorage Zones

First Analysis

The analysis of the anchorage zone stresses at the buttresses was determined to be the most critical of all the various types of anchorage areas of the shell. The local stress distribution in the immediate vicinity of the bearing plates has been derived by the following three analysis procedures:

1. The Guyon equivalent prism method: This method is based both on experimental photoelastic results as well as on equilibrium considerations of homogeneous and continuous media. It should be noted that the relative bearing plate dimensions are considered.

2. In order to include biaxial stress effects, use has been made of the experimental test results presented by S J Taylor at the March 1967 London Conference of the Institution of Civil Engineers (Group H, Paper 49). This paper compares test results with most of the currently used approaches (such as Guyon's equivalent prism method). It also investigates the effect of the rigid trumpet welded to the bearing plate.

3. Stresses were predicted using a two-dimensional plane strain finite element model which represented the concrete as a homogeneous elastic material. The model consisted of a 60-degree horizontal segment of the containment shell with a buttress in the center. In the vertical direction the model was one element thick. A portion of this model is shown in Figure 5.8-14. The effects of the prestressing loads were applied as concentrated forces at the anchorage and as uniformly distributed loads that varied according to the curvature of the tendons. Isostress plots are shown in Figure 5.8-15.

The Guyon method yields the following results for a loading ratio (a'/a) = 0.9, where (a'/a) is the ratio of width of bearing plate to width of concrete under bearing plate. Maximum compressive stress under the bearing plate:

σ_c = -2,400 psi

Maximum tensile stress in spalling zone:

σ spalling . +2,400 psi = $-\sigma_c$

Maximum tensile stress in bursting zone:

σ maximum bursting = 0.04P = +95 psi

S J Taylor's experimental results indicate that the anchor plate will give rise to a similar stress distribution pattern as Guyon's method; the main difference lies in the fact that the central bursting zone has a tensile stress peak of twice Guyon's value:

σ maximum bursting = +190 psi

By finite element analysis, the symmetric buttress loading yields a tensile peak stress in the bursting zone very close to S J Taylor's value:

σ maximum bursting = +220 psi

A state of biaxial tension exists on the outside face of the concrete for the load combination $1.05D + 1.5P + 1.0T_a + 1.0F_f$. For this loading condition, the averaged vertical (meridional) stress component is:

$f_a \approx +400$ psi

The compressive bearing plate stress at 10-inch depth below the bearing plate is:

$f_c \approx -1{,}500$ psi

NOTE: The steel trumpet carries 7.2% of the prestress force.

Thus, the two values introduced in the biaxial stress envelopes proposed in S J Taylor's article:

f_c/f'_c = 1,500/5,000 = 0.3

f_a/f'_c = 400/5,000 = 0.08

show that failure could occur if vertical reinforcing were not provided. In fact, the maximum allowable vertical averaged tensile stress according to Taylor's interaction curve is f_a/f'_c = 0.03; therefore, f_a = +150 psi.

Based upon this analysis, it was determined that additional vertical reinforcing steel was needed in the buttresses to control bursting stresses. This conclusion was also supported by:

1. The results of the second analysis

2. Full-scale load tests of the anchorage on the same concrete mix used in the structure and review of prior uses of the anchorage

3. The post-tensioning supplier's recommendations of anchorage reinforcing requirements

4. Review of the final details of the combined reinforcing by the consulting firm of T Y Lin, Kulka, Yang, and Associate

<u>Second Analysis</u>

The second analysis was a continuation and refinement of the previous analysis. Its purpose was to assess the adequacy of the reinforcing steel that controls the bursting stresses.

The analysis calculated the contact pressure (f_{cp} per ACI 318-63) assuming a uniform pressure distribution over the area of the bearing plate. The resulting contact pressures for the dome buttress and top vertical tendon anchorages were 3,090 psi. The contact pressure for the bottom vertical tendon anchorage was 3,145 psi.

Adoption of the ACI 318-63 criteria for contact pressure requires that adequate reinforcement be provided in the anchorage zones. The analytical methods used in evaluating the reinforcement and their results are as follows:

1. Buttress strains were predicted using a two-dimensional plane stress finite element model. The model consisted of a 1-inch thick vertical slice parallel to the outside surface of the buttress taken across the anchorage of the hoop tendons. A compressive displacement of 0.0154 inch, which corresponds to the summation of about 470 micro-strain in the vertical direction, was applied as the upper boundary condition for the analytical model of the buttress portion. This input strain was derived from the finite element analysis discussed in Subsection 5.8.4.1.2 for the load combination $D + F_f + 1.5P + T_a$.

   The anchorage forces were applied to the bearing plate. Successive analytical cycles were made to allow the redistribution of stresses resulting from concrete cracking.

   The maximum strain in the bursting zone was then used to determine the level of reinforcing steel stress reported in Table 5.8-2 for the buttress vertical reinforcement.

2. Bursting zone reinforcement stresses were calculated using Leonhardt's formula (Reference 23) for the following areas:

   a. Hoop tendon anchorages (radial direction only)

   b. Vertical tendon anchorages (top and bottom)

   c. Dome tendon anchorages

   Leonhardt's formula is:

   $$Z = 0.3V(1 - a/d)$$

   Where:

   Z = total splitting force

   V = prestressing force

   a = anchor plate diameter

   d = effective prism width

   The results were checked by comparisons with data from S J Taylor's experiments (Reference 24).

Stresses due to the loading conditions defined in Table 5.8-2 were added to the bursting zone stresses. These ($D + F_f + 1.5P + T_f$ and $D + F_f + T_a + E^I$) stresses were derived using the finite element model discussed in Subsection 5.8.4.1.2.

3. A direct tension evaluation of the base slab was performed for the loads imposed by the tendon anchorages. Both the horizontal and vertical components were considered in the finite element computer solution. The finite element analysis did not indicate that there was a tendency for the base slab to crack. However, the capability of the reinforcing to resist the horizontal force vector was reevaluated. For an anchorage transfer load of 750 KIPs, the horizontal component was approximately 150 KIPs. The hoop steel was considered to have no effect and the base slab horizontal steel near the anchor plate was considered to resist the entire horizontal load. The area of steel in a 2-foot distance was considered to be 12 square inches. The resultant stress was approximately 12.5 Ksi (150/12). Therefore, propagation of a crack was considered unlikely.

5.8.4.3 <u>Liner Plate System</u>

5.8.4.3.1 Liner Plate

The liner plate was analyzed for strain compatibility with the shell.

The liner plate was also analyzed for the fatigue loadings listed in Subsection 5.8.3.2.1. Using Figure N-415(a) of the ASME Boiler and Pressure Vessel Code, Section III, Article 4, 1965. Since this figure does not extend below ten cycles, ten cycles were used conservatively for the DBA instead of one cycle as indicated in Subsection 5.8.3.2.1. The resulting stresses were insignificant.

5.8.4.3.2 Liner Plate Anchors

When the liner plate moves inward radially as shown in Figure 5.8-16, Sheet 1, the sections will develop membrane stress due to the fact that the anchors have moved closer together. Due to initial inward curvature, the section between 1 and 4 will deflect inward giving a longer length than adjacent sections and some relaxation of membrane stress will occur. It should be noted here that Section 1-4 cannot reach an unstable condition due to the manner in which it is loaded.

The first part of the solution for the liner plate and anchorage system is to calculate the amount of relaxation that occurs in Section 1-4, since this value is also the force across Anchor 1 if it is infinitely stiff. This solution was obtained by solving the general differential equation for beams and the use of calculus to simulate relaxation or the lengthening of Section 1-4.
Figure 5.8-16, Sheet 1, shows the symbols for the forces that result from the first step in the solution.

Using the model shown in Figure 5.8-16, Sheet 2, and evaluating the necessary spring constants, the anchor was allowed to displace.

The solution yielded a force and displacement at Anchor 1, but the force in Section 1-2 was (N)-$K_{R(Plate)}S_1$ and Anchor 2 was no longer in force equilibrium.

The model shown in Figure 5.8-16, Sheet 2, was used to allow Anchor 2 to displace and then to evaluate the effects on Anchor 1.

The displacement of Anchor 1 was $S_1 + S_1$ and the force on Anchor 1 was $K_c(S_1 + S_1)$. Then Anchor 3 is not in force equilibrium and the solution continued to the next anchor.

After the solution was found for displacing Anchor 2 and Anchor 3, the pattern was established with respect to the effect on Anchor 1 and, by inspection, the solution considering an infinite amount of anchors was obtained in the form of a series solution.

The preceding solution yielded all necessary results. The most important results were the displacement and force on Anchor 1.

Various patterns of welds attaching the angle anchors to the liner plate have been tested for ductility and strength when subjected to a transverse shear load such as ΔN and are shown in Figure 5.8-17.

Using the results from these tests together with data from tests made for the Fort St Vrain PSAR, Amendment 2, and Oldbury vessels (Reference 1), a range of possible spring constants was evaluated for the Palisades liner. By using the solution previously obtained, together with a chosen spring constant, the amount of energy required to be absorbed by the anchor was evaluated.

5.8.4.4 Penetrations

The analysis of penetrations is discussed in Subsection 5.8.6.4.

5.8.5 DESIGN

5.8.5.1 Design Basis

The containment structure shall be designed in such a manner that its integrity, both structural integrity and leak tightness, will be maintained for the various load combinations presented in Subsection 5.8.3. In order to accomplish this objective, the following criteria are imposed:

1. The structure shall have a low-strain elastic response such that its behavior will be predictable under all design loadings, and

2. The integrity of the liner plate shall be guaranteed under all loading conditions.

The containment structure was designed in accordance with the design criteria presented in Subsections 5.8.3 and 5.8.5. These criteria are based upon the Building Code Requirements for Reinforced Concrete, ACI 318-63 and the ASME Boiler and Pressure Vessel Code (ASME B&PV Code), Sections III, VIII and IX. The use of these codes ensures that the two criteria mentioned above are satisfied. Where departures or additions from these codes have been made, they have been done in the following manner:

1. The environmental conditions of severity of load, load cycling, weather, corrosion conditions, maintenance and inspection for this structure have been compared and evaluated with those for other code designed structures to determine the appropriateness of the modifications.

2. The consultant firm of T Y Lin, Kulka, Yang and Associate was retained to assist in the development of the design criteria, to update the criteria and to review the analysis, the design and the construction drawings to ensure that the criteria were being implemented as intended.

3. Upon completion of the PSAR review by the AEC Division of Reactor Licensing Staff, it was agreed to pursue the most recent research on the combined effects of shear force, axial force and moment on concrete structures. Dr Alan H Mattock of the University of Washington was retained by Bechtel to assist in developing the proper design approach and to interpret or modify the formulas of ACI 318-63 for design of the containment structure.

4. All criteria, specifications and details relating to liner plate and penetrations, cathodic protection and corrosion protection have been referred to Bechtel's Metallurgy and Quality Control Department. This department advised the corporation on problems of welding, quality control, metallurgy, cathodic protection and corrosion protection.

5. The design of the Palisades containment structure was continually reviewed as the criteria were improved for successive license applications to ensure that the structure met the latest design criteria.

The following observations concerning the design of the containment structure provide confidence in its ability to withstand the load combinations of Subsection 5.8.3 without any loss of integrity:

1. The primary membrane integrity of the structure is provided by the unbonded post-tensioned tendons, each one of which has been stressed to 80% of ultimate strength during installation and performs at approximately 60% to 65% during the life of the structure. Thus, each tendon has been effectively proof-tested prior to operation of the Plant.

2. Any three adjacent tendons in any of the three tendon groups can fail without significantly affecting the strength of the containment structure because of the load redistribution capabilities of the shell. The bonded reinforcing steel provided on the outside face of the shell ensures that this redistribution capability exists.

3. The unbonded tendons are continuous from anchorage to anchorage, being deflected around the penetrations as required. Because the tendons are unbonded, they are effectively isolated from the effects of local stress concentrations. Thus, the membrane integrity of the shell can be ensured even where high localized stresses exist.

4. Over the life of the containment structure, the tendons are subjected to an ever decreasing stress due to relaxation of the tendons and creep of the concrete. During containment pressurization resulting from the DBA, the tendons are subjected to a small stress increase (2% to 3% of the ultimate tendon strength). Thus, the tendons are never subjected to large changes in stress.

5. The concrete portion of the structure is subjected to the highest compressive stresses during the post-tensioning operation. During pressurization resulting from the DBA, the concrete is subjected to a large change in stress, but this change is, in general, a decrease in stress; the large membrane concrete compressive stresses due to the prestressing forces are reduced by the pressure induced stresses, while the tensile stress in the tendons increases slightly.

6. The deformations of the containment structure during normal Plant operation, or during accident conditions, are relatively minor due to the low strain behavior of the shell. The largest deformations occur during or shortly after the post-tensioning operation. This low strain behavior together with the inherent strength of the structure permits the shell to be used as an anchor point for all piping that passes through it (see Subsection 5.8.6). This behavior eliminates the need for expansion bellows and significantly reduces the likelihood of leaks developing at these containment penetrations.

5.8.5.2 Containment Structure Concrete

5.8.5.2.1 General Criteria

In order to ensure the safety of the containment structure, it was designed with adequate strength to resist various load combinations which represented both working stress and yielding situations. Further, to ensure the proper performance of the structure, the amount of cracking, the magnitude of deformations and the extent of corrosion were considered. The structure was designed for the following loading conditions:

1. Construction condition

2. Working stress condition

3. Yield strength condition

The design methods and allowable stresses of ACI 318-63 were used for the concrete, reinforcing steel and prestressing tendons, unless otherwise noted in Subsection 5.8.5.2.

Stresses from the finite element computer output for axisymmetric loads and from analytical solutions for nonaxisymmetric loads were based on homogeneous materials; therefore, some adjustment was necessary to evaluate the true stress-strain conditions when cracks developed in the tensile zone of the concrete.

For design purposes, the above-mentioned stresses were converted to stress resultants for the particular cross section being designed. These stress resultants were combined in accordance with the appropriate load combinations to produce total moment, total axial force and total shear. For the combination of moment and axial force or moment alone, the neutral axis of the cracked section (proposed reinforcement included) can be determined from the conditions of equilibrium using a compressive stress block based upon the straight line, elastic stress profile. The location of the neutral axis can be expressed as a function of the modular ratio, the amount of reinforcement and the axial force.

Concrete cracking will reduce the calculated thermal moment. However, the existence of a net compressive stress over the cross section precludes this self-relieving action (eg, $D + F_f + T_a$). Whenever a cracked section approach was utilized and the thermal moment was in the same direction as the moment associated with the other loads (bending moment), a reduced thermal moment was added to the bending moment (eg, $D + F_f + 1.5P + T_a$). Nonthermal moments at cracked sections were not reduced.

Throughout the shell, with the exception of the ring beam, a minimum of 0.25% (approximate) ASTM A-15 reinforcing steel is provided in each of two perpendicular directions on the exterior face to accommodate the tensile stresses due to shrinkage and temperature (including DBA temperature). In general, there are no tensile thermal stresses on the inside face of the shell; therefore, no "minimum" steel is provided on this face.

Approximately 0.5% reinforcing steel, rather than the 0.25% required by the detailed analysis, is used for the dowels at the cylinder/slab junction. This increase places the reinforcement percentage within the lower limits of Dr Mattock's test data (approximately 0.3%), thereby ensuring that sufficient steel is provided to prevent cracking, caused by flexural stresses, from adversely affecting the shear capacity of the section.

For the construction condition and the working stress condition, the computed stresses for both the concrete and the reinforcing steel were allowed to exceed their associated allowables provided that the yield strength criteria of Subsection 5.8.5.2.4 was satisfied.

5.8.5.2.2 Construction Condition

For loads encountered prior to the post-tensioning operation (dead, live and wind), the containment structure was designed as a conventionally reinforced concrete structure in accordance with the working stress provisions of ACI 318-63.

In addition to the above-mentioned loads, the containment shell was designed for the loads associated with the post-tensioning operation. For this condition the shell was designed in accordance with the provisions of ACI 318-63, with the following exceptions:

1. For load combinations that include initial prestress (F_i), ACI 318-63, Chapter 26, allows a concrete compressive stress of 0.60 f_{ci}'. In order to limit creep deformations, the membrane compressive stress was limited to 0.30 f_{ci}'. However, for the combination of membrane compressive stress and flexural compressive stress, the maximum allowable stress was limited to the ACI 318-63 value of 0.60 f_{ci}'.

2. For local compressive stress concentrations, which are predicted by the finite element analyses, the maximum allowable concrete compressive stress was limited to 0.75 f_{ci}', as long as reinforcing steel was provided to control the associated local strains. These areas of high local stresses are present in every structure but they are seldom identified because of simplifications made in the analysis. These high stresses were allowed because they occurred in a very small percentage of the cross section, were confined by material at lower stress, and would have to be considerably greater than the values allowed before significant local plastic yielding would occur.

3. Membrane tensile stress and flexural tensile stress were permitted during the post-tensioning sequence provided they did not jeopardize the integrity of the liner plate. Membrane tensile stress, which occurs adjacent to the Region being posttensioned, was limited to 1.0 / f_{ci}'. If this limit was exceeded, additional reinforcement was provided. When there was a moment but no tensile force, the section was designed in accordance with Section 2605(a) of the ACI Code. The stress in the liner plate due to the combination of membrane tensile stress and flexural tensile stress was limited to 0.5 f_y.

4. Reinforcing steel was provided for radial shear (out-of-plane shear) in accordance with the shear criteria of ACI 318-63, Chapter 26, with the exceptions noted in Subsection 5.8.5.2.4.

5. Because the post-tensioning of the containment structure was undertaken after the concrete had already achieved design strength, the term for initial strength, f_{ci}', was numerically equal to $f\dot{c}$.

5.8.5.2.3 Working Stress Condition

For the loads and load combinations presented in Subsection 5.8.3.1.2, the containment shell was designed in accordance with the provisions of ACI 318-63, with the following exceptions:

1. For load combinations that include final prestress (F_f), the concrete compressive stress was limited to 0.30 f'_c and 0.60 f'_c as described in Subsection 5.8.5.2.2. The concrete compressive stresses due to final prestress were less severe than those due to initial prestress because of concrete creep. For local compressive stress concentrations, the allowable concrete compressive stress was limited to 0.75 f'_c, as long as reinforcing steel was provided to control the associated strains.

2. If the average membrane compressive stress was less than 100 psi, this stress was neglected, a cracked section was assumed and reinforcing steel was provided to carry the tension created by the moment alone.

3. When the maximum tensile stress (membrane stress plus flexural stress) did not exceed $6 / f'_c$ and the extent of the tension zone was not more than one-third the depth of the section, reinforcing steel was provided to carry the entire tension in the tension block. If either or both of these criteria were exceeded, a cracked section was assumed and the reinforcing steel was provided in the following manner:

   a. Bending Moment Only

      Reinforcing steel was provided to resist the moment.

   b. Bending Moment Plus Thermal Moment

      When the bending moment was in the opposite direction to the thermal moment, the thermal moment was ignored. Reinforcing steel was provided to resist the bending moment.

      When the bending moment was in the same direction as the thermal moment, the tensile stress in the reinforcing steel due to the thermal gradient was computed in accordance with the method of ACI 505. For this situation, the allowable tensile stress in the reinforcement, due to the bending moment alone, was limited to 0.5 f_y minus the thermal stress.

4. Reinforcing steel was provided for radial shear (out-of-plane shear) in accordance with the shear criteria of ACI 318-63, Chapter 26, with the exceptions noted in Subsection 5.8.5.2.4.

For the loads and load combinations presented in Subsection 5.8.3.1.2, the containment base slab was designed in accordance with the working stress provisions of ACI 318-63.

5.8.5.2.4 Yield Strength Condition

The containment structure was designed for the loads and load combinations presented in Subsection 5.8.3.1.3 using the methods and allowable stresses discussed below.

Theory

In general, the containment structure was designed to maintain an elastic behavior under all load combinations. The upper limit of elastic behavior was considered to be the yield strength of the effective load carrying structural materials. For steels (structural steel, reinforcing steel, prestressing tendons), this limit was taken to be the guaranteed minimum yield given in the appropriate ASTM specification. For concrete, this limit was the ultimate values of shear (as a measure of diagonal tension), bond per ACI 318-63 and the 28-day ultimate compressive strength (f'_c).

The ACI ultimate strength stress distribution was not used. Instead, a straight line, elastic profile was employed. The maximum concrete strain was limited as follows:

Type of Stress	Allowable Strain
Membrane stress only	$0.85\ f'_c/E_c$
Membrane stress plus flexural stress from secondary moments and local loads	f'_c/E_c
Membrane stress plus flexural stress from secondary moments, local loads and thermal loads	0.003 in/in

Tensile reinforcing steel was allowed to yield provided that the above-mentioned strains were not exceeded; during yielding the steel stress was assumed to remain at f_y. The prestressing tendons were not allowed to yield under any circumstances.

Shell Reinforcement

The membrane tensile force was combined with the membrane shear (in-plane shear) to determine the principal tensile stress resultant. The associated tensile stress in the concrete was limited to $3 / f'_c$. If this limit was exceeded, additional reinforcement was provided. (Note: Sufficient prestressing forces were provided in the cylinder and dome to eliminate any membrane tensile force for the load combinations associated with the "Working Stress Condition"; therefore, for this condition, the principal stress resultant was not critical.)

The maximum tensile stress (principal tensile stress plus flexural stress) was limited to $6 / f'_c$. When the maximum tensile stress exceeded this limit, a cracked section was assumed and the reinforcing steel was provided in the following manner:

1. Thermal Moment Only

   Reinforcing steel was provided in accordance with the method of ACI 505. The minimum area of steel provided was approximately 0.25% in each direction.

2. Bending Moment Plus Thermal Moment

   When the bending moment was in the opposite direction to the thermal moment, the thermal moment was ignored. Reinforcing steel was provided to resist the bending moment.

When the bending moment was in the same direction as the thermal moment, the tensile stress in the reinforcing steel due to the thermal gradient was computed in accordance with the method of ACI 505. For this situation, the allowable tensile stress in the reinforcement, due to the bending moment alone, was limited to 1.0 f_y minus the thermal stress.

Reinforcing steel was provided for radial (out-of-plane) shear in accordance with the shear criteria of ACI 318-63, Chapter 26, with the exceptions noted below. These exceptions were recommended by Dr Alan H Mattock. All notation, except as noted below, is in accordance with Chapter 26 of the ACI Code.

1. Formula 26-12 of the code was replaced by:

$$V_{ci} = Kb'd \,/\, f'_c + M_{cr} \left(\frac{V}{M'}\right) + V_i$$

Where:

$$K = \left[1.75 - \frac{0.036}{np'} + 4.0\,np'\right]$$

but not less than 0.6 for $p' \geq 0.003$.
For $p' < 0.003$, the value of K was zero.

$$M_{cr} = \frac{I}{Y}\left[6 \,/\, f'_c + f_{pe} + f_n - f_i\right]$$

f_{pe} = compressive stress in concrete, including the stress due to any secondary moment, due to the prestress loads applied normal to the cross section after all losses. This stress is computed at the extreme fiber of the section at which tensile stresses are caused by the live loads. f_{pe} shall be positive for a compressive stress.

f_n = stress due to axial applied loads (ie, loads other than initial loads). f_n shall be negative for a tensile stress and positive for a compressive stress.

f_i = stress due to initial loads (ie, dead load and other permanent loads, prestress loads excluded), including the stress due to any secondary moment. This stress is computed at the extreme fiber of a section at which tensile stresses are caused by the applied loads. f_i shall be negative for a tensile stress and positive for a compressive stress.

n = $\frac{505}{f'_c}$

V = shear at the section under consideration due to the applied loads.

M' = moment at a distance d/2 from the section under consideration, measured in the direction of decreasing moment due to applied loads.

V_i = shear due to initial loads. V_i shall be positive when it is in the same direction as the shear due to the applied loads.

The lower limit placed by ACI 318-63 on V_{ci}, 1.7b'd f'$_c$, was not applied.

2. Formula 26-13 of the ACI Code was replaced by:

$$v_{cw} = 3.5\,b'\,d\,\sqrt{f'_c}\left(1 + \frac{f_{pc} + f_n}{3.5\sqrt{f'_c}}\right)^{1/2}$$

The term f_n is as defined above. All other notations are in accordance with Chapter 26, ACI 318-63.

Base Slab Reinforcement

The containment base slab was designed in accordance with the ultimate strength provisions of ACI 318-63 as modified by the preceding "theory" discussion.

Φ Factors

The yield strength of all load carrying structural elements that comprised the containment structure was reduced by a yield capacity reduction factor (Φ) as listed in Subsection 5.8.3.1.3. This factor provides for "the possibility that small adverse variations in material strengths, workmanship, dimensions, control and degree of supervision while individually within required tolerance and the limits of good practice, occasionally may combine to result in undercapacity" (Reference 2).

The design strength of the concrete cross sections, except under shear, was determined by multiplying the yield strength equation, or portion thereof, by the appropriate Φ factor(s). For shear, the design strength was determined by multiplying the basic permissible unit shear by the Φ factor. The yield strength equation gives the "ideal" strength, assuming materials are as strong as specified, sizes are as shown on the drawings, the workmanship is excellent and the strength equation itself is theoretically correct. The practical, dependable strength (design strength) may be something less, due to the variability of these parameters.

The ACI Code provides for the variability of these parameters by using the following Φ factors:

Φ = 0.90 for reinforced concrete in flexure

Φ = 0.85 for shear (diagonal tension), bond and anchorage in reinforced concrete

Φ = 0.75 for spirally reinforced concrete compression members

Φ = 0.70 for tied reinforced concrete compression members

Φ values for flexural members (beams) are larger than those for compression members (columns). This reflects not only the fact that the variability in reinforcing steel properties is less than the variability in concrete properties but also the nature of the potential failure. Conventional concrete design of beams requires that the design be controlled by yielding of the tensile reinforcing steel, a ductile failure; therefore, even if the concrete is slightly understrength, it will not significantly affect the safety of the structure. On the other hand, concrete strength is critical for columns which are subject to brittle failure; therefore, understrength can significantly affect the safety of the structure.

The additional Φ values (see Subsection 5.8.3.1.3) represent Bechtel's best judgment of how much understrength should be considered for materials or reinforcement conditions not directly covered by the ACI Code. Embedded structural steel that is considered to be a part of the reinforcement has been assigned a value of Φ = 0.90. For members in flexure, ACI uses Φ = 0.90. Using the same line of reasoning as that upon which the code value was founded, a value of Φ = 0.90 has been assigned for reinforcing steel in direct tension. The Φ = 0.85 code value mentioned above for bond includes the situation in which reinforcing steel is lap spliced. A separate entry has been tabulated for lap splices to emphasize the distinction between lap splices and welded or mechanical splices. For prestressing tendons in direct tension, a value of 0.95 was assigned. This Φ value was higher than the value used for conventional reinforcement because:

1. During installation the tendons were to be jacked to about 94% of their yield strength (0.8 f'_s) before being seated at a lower stress level. Hence, each tendon would be effectively proof-tested.

2. The method of manufacturing prestressing steel (cold drawing and stress relieving) ensures that the tendons are a higher quality product than conventional reinforcing steel.

5.8.5.2.5 Results

The results of the design effort are summarized in Table 5.8-1 for several of the load combinations of Subsection 5.8.3 which were considered to be significant in the design of the containment structure. The load combinations for which stresses are presented include both axisymmetric and nonaxisymmetric loads and encompass all three loading conditions (construction, working stress and yield strength). This table presents a tabulation of computed and allowable stresses for both the concrete and the reinforcing steel for selected cross sections. For each section, this table also includes f'_c, the concrete thickness, the type of reinforcing steel and the percent reinforcement on both the inside and outside faces. No stresses are shown for the prestressing tendons since they exhibit an almost constant stress level regardless of the loading condition.

5.8.5.3 Prestressing System

5.8.5.3.1 Tendons

Prestress Losses

In accordance with the ACI 318-63, the following sources of loss of prestress were considered in the design of the prestressing tendons:

Source of Loss	Assumed Value
Friction due to intended or unintended curvature in the tendons	K = 0.0003, μ = 0.156
Seating of anchorage	None
Elastic shortening of concrete(a)(b)	$\frac{f_{cpi}}{(5.5 \times 10^6)}$ in/in/psi
Creep of concrete	0.22×10^{-6} in/in/psi
Shrinkage of concrete	70×10^{-6} in/in
Relaxation of tendon stress	8% of 0.65 f'_s = 12.5 Ksi

(a) f_{cpi} is the concrete membrane stress due to initial prestress (units of psi).
(b) 5.5×10^6 psi is the value for E_c.

The following statements provide background information on the selection of numerical values to represent the prestress losses:

1. Frictional loss parameters for unintentional curvature (K) and intentional curvature (μ) were conservatively based on full-scale friction test data.

   Test Data Value K = 0.0003, μ = 0.125

   Design Value K = 0.0003, μ = 0.156

2. There was no allowance for a seating loss for the BBRV anchor since no slippage will occur in the anchor during transfer of the tendon load into the structure. Sample lift-off readings will be taken to confirm that any seating loss is negligible.

3. The loss of tendon stress due to elastic shortening was based on the predicted change in strain in the first tendon stressed in the group (ie, dome, hoop and vertical) relative to the last tendon stressed in the group. Because the amount of prestressing steel per foot of cross section is different for all three groups, there were three distinct values of elastic loss.

4. For design purposes, a creep loss value of 0.22 x 10^{-6} in/in/psi was assumed and f_{cpi} in the hoop direction was conservatively selected as 1,500 psi. The computed creep loss value for the hoop direction (330 x 10^{-6} in/in) was used throughout the structure. This resulted in a prestress loss of approximately 9.7 Ksi using 29,000 for E_s.

   The concrete properties study conducted at the University of California (Reference 3), which was performed subsequent to the design, indicated an actual creep value of 0.125 x 10^{-6} in/in/psi; hence, the assumed value was conservative. Conversion of this unit creep to dome, hoop and vertical tendon stress losses, using individual f_{cpi} values, produced the following values:

Dome	5.5 Ksi
Hoop	5.5 Ksi
Vertical	2.8 Ksi

   These values indicate that 9.7 Ksi design loss was conservative.

5. The value used for shrinkage loss represents only that shrinkage that could occur after stressing. Since the concrete was, in general, well aged at the time of stressing, little shrinkage was left to occur.

6. The value of the relaxation loss was based on information furnished by the tendon system vendor, Inland-Ryerson Construction Products Company.

Prestress Forces

The following tabulation shows the magnitude of the design losses and the design final effective prestress at end of 40 years for a typical dome, hoop and vertical tendon. This tabulation is based upon a tendon jacking stress of 0.80 f'_s (192,000 psi) and use of the prestress loss parameters discussed above. At the conclusion of the Twentieth and Twenty-Fifth year tendon surveillances, regression analyses were performed utilizing the surveillance data (References 36, 37, 44, and 45). The results consistently indicated that the effective group tendon forces, dome/hoop/vertical, were significantly higher than predicted values beyond the 40-year time period.

	Dome (Ksi)	Hoop (Ksi)	Vertical (Ksi)
Jacking stress	192	192	192
Friction loss	19	21.3(a)	21
Seating loss	0	0	0
Seating stress = initial prestress (F_i)	173	170.7	171
Elastic loss	8.1	8.4	3.7
Creep loss	9.7	9.7	9.7
Shrinkage loss	2.1	2.1	2.1
Relaxation loss	12.5	12.5	12.5
Final effective stress(b) = final prestress (F_f)	140.6	138.0	43.0

(a) Average of crossing tendons.
(b) This tendon force does not include the effect of containment pressurization resulting from the DBA. During this event, the tendons are subjected to a small stress increase of 2% to 3% of f'_s.

5.8.5.3.2 Tendon Anchorage Zones

Because anchorage loads create high localized stresses, both compressive (under the bearing plate) and tensile (bursting zones), special design consideration was given to the tendon anchorage zones.

The maximum allowable compressive stress directly under the bearing plate was in accordance with ACI 318-63, Section 2605, with the exception that f_{cp} was limited to f'_c. Reinforcement in the anchorage zone, as required by Section 2605, was located sufficiently close to the bursting zones to both resist and confine the tensile stresses. This reinforcement is adequate to resist the bursting stresses in combination with those stresses resulting from the loading combinations deemed most severe for the anchorages. In addition to this reinforcement, spiral reinforcing steel was provided at the dome tendon anchorages. The maximum allowable tensile stress was limited to 0.9 f_y. Table 5.8-2 summarizes the computed and allowable reinforcing steel stresses.

The special design consideration, just described, in conjunction with the layout of the tendons, ensures that the containment structure will not suffer a delayed rupture or shear failure at any anchorage location.

5.8.5.4 Liner Plate System

5.8.5.4.1 General

In order to satisfy the criterion of Subsection 5.8.5.1 that "the integrity of the liner plate shall be guaranteed under all loading conditions," the following constraints are imposed:

1. The liner plate shall be protected from loss of function due to damage inflicted by missiles generated by a Loss of Coolant Accident (see Subsection 5.5.3).

2. The liner plate strains shall be limited to allowable values that have been shown to result in leak-tight pressure vessels or leak-tight, high-pressure piping.

3. The liner plate shall be prevented from developing significant distortion.

   a. The anchors shall have sufficient strength and ductility so that their energy absorbing capability is sufficient to restrain the maximum force and displacement resulting from the condition where a panel with initial outward curvature is adjacent to a panel with initial inward curvature.

   b. The anchors shall have sufficient flexural strength to resist the bending moment which would result from the condition in Subcriterion a.

   c. The anchors shall have sufficient strength to resist a pullout force.

   d. At discontinuities, including penetrations, the anchors shall be designed to accommodate the forces generated by restraining the liner plate. Special attention shall be given to the design of corner details and connection details in order to minimize these forces.

These constraints are satisfied by the design of the liner plate system and the design of missile barriers (containment structure shell and various containment internal structures).

5.8.5.4.2 Liner Plate

There are no design conditions under which the liner plate is relied upon to assist the concrete in maintaining the integrity of the structure even though the liner will, at times, provide assistance in order to maintain deformation compatibility. Fatigue stresses are insignificant (see Subsection 5.8.4.3.1).

Forces are transmitted between the liner plate and the concrete through the anchorage system and through direct contact (pressure). At times, forces may also be transmitted by bond and/or friction. These forces cause, or are caused by, liner plate strains. The liner plate is designed to withstand the predicted strains. The effect of concrete cracking on the liner plate has also been considered.

The most appropriate basis for establishing allowable liner plate strains was considered to be the ASME B&PV Code, Section III, Article 4, 1965. Specifically, the following sections were adopted as guides in establishing allowable strain limits:

1. Paragraph N-412(m) Thermal Stress, Subparagraph 2

2. Paragraph N-412(n) Operational Cycle

3. Paragraph N-414.5 Peak Stress Intensity
   Table N-413
   Figures N-414 and N-415(a)

4. Paragraph N-415.1 Vessels Not Requiring Analysis for Cyclic Operation

Because the liner plate is restrained against significant distortion by continuous angle anchors, the limiting strain may be determined from a fatigue standpoint (see Paragraph N-412(m)(2)). The allowable strain in the liner plate was obtained from the allowable stress of Figure N-415(a). As discussed below, the critical load combination for the liner plate includes the DBA temperature and since DBA fatigue considerations required only one cycle (see Subsection 5.8.3.2.1), one cycle was to have been used in this analysis as well. However, since this figure does not extend below ten cycles, the allowable stress corresponding to ten cycles was conservatively used.

The 1/4-inch liner plate will yield when subjected to the effects of concrete creep and shrinkage, prestressing forces and high temperature. Because the liner plate shares deformation compatibility with the containment shell along the continuous angle anchors and because the shell is very stiff, these loads do not lead to a severe condition in the liner plate. From this loading condition, the membrane compressive strains will be approximately 2-1/2 times the yield strain (0.0025 in/in) when based on a guaranteed minimum yield. The combined membrane and flexural strains due to possible inward deformation of the liner plate could reach a compressive value of approximately 4-1/2 times yield strain (0.0045 in/in). The above conditions, which imposed the highest computed strain, will occur shortly after the DBA when the pressure has dropped off, but the temperature is still relatively high. The allowable strain for 10 cycles is approximately 0.02 in/in. This allowable value was conservatively reduced to 0.005 in/in; it exceeds the maximum predicted value of 0.0045 in/in.

5.8.5.4.3 Liner Plate Anchors

The following factors were considered in the design of the anchorage system:

1. Initial inward curvature of the liner plate between anchors due to fabrication and erection inaccuracies

2. Variation of anchor spacing including spacing to reflect the possibility of a missing or failed anchor

3. Misalignment of liner plate seams

4. Variation of plate thickness

5. Variation of liner plate material yield stress

6. Variation of Poisson's ratio for the liner plate material

7. Cracking of concrete in the anchor zone

8. Variation of the anchor stiffness

Factors 3 and 6 were found to be insignificant. Factors 7 and 8 were considered by decreasing the anchor stiffness.

Many of the preceding factors were considered using the analytical procedure of Subsection 5.8.4.3 and the design cases detailed below:

Case I Simulates a plate with a yield stress of 32 Ksi(a) and no variation in any other parameters.

(a) The actual yield stress of the liner plate material is 30 Ksi. However, use of the larger value is conservative in this application.

Case II Simulates a 25% increase in yield stress and no variation in any other parameters.

Case III Simulates a 25% increase in yield stress, a 16% increase in plate thickness and an 8% increase for all other parameters.

Case IV Simulates the maximum reasonable variation that could exist in the liner plate yield stress, liner plate thickness, concrete modulus of elasticity, etc, by considering an 88% increase in yield stress with no variation of any other parameters.

Case V Is the same as Case III except the anchor spacing and initial inward displacement have been doubled to simulate what happens if an anchor is missing or has failed.

The parameter variation of Cases I through V is summarized in Table 5.8-3 along with the resulting factor of safety for the critical anchor, that anchor located adjacent to the panel with the initial inward curvature.

Dividing the amount of energy that the liner anchors will absorb by the most probable maximum energy, resulted in the following factors of safety:

Case	Factor of Safety Against Failure for the Critical Anchor
I	37.0
II	19.4
III	9.9
IV	6.28
V	4.25

In order to withstand the design conditions without failure, the critical anchor will have to deform approximately 0.05 inch (Case IV). In order for the critical anchor to achieve this displacement, the concrete will have to yield excessively along the first 1/2 inch of the anchor. However, the concrete can easily redistribute the imposed loads to survive this condition since the ultimate deformation capacity of the anchorage system (anchor, liner plate, concrete), as shown in tests, is approximately 0.15 inch.

5.8.5.4.4 Brackets

In designing the liner plate system for bracket loads applied either perpendicular or parallel to the plane of the liner plate, the following criteria and methods were used:

1. The liner plate was thickened to reduce the predicted stress level in the plane of the liner plate. The use of a thickened plate and a corresponding thicker weld to attach the bracket to this plate reduces the probability of a leak at a bracket location.

2. Under the application of a tensile load, applied perpendicular to the plane of the liner plate, no yielding shall occur in the perpendicular direction. This criterion was satisfied, limiting the predicted through-direction strain to 90% of the guaranteed minimum yield.

3. The allowable stress in the perpendicular direction was calculated using the allowable strain in this direction (Item 2) together with the predicted strains in the plane of the liner plate. The use of predicted strains, rather than actual strains, provides for a conservative determination of the allowable stress.

4. If the bracket does not have a matched extension welded to the rear of the liner plate, the allowable through-direction stress was limited to 75% of the stress permitted in Item 3. This reduced stress reflects the fact that the in-plane strength of a plate is not the same in both directions; the direction perpendicular to rolling has the lower strength.

5. The necessary plate characteristics were ensured by ultrasonic examination.

5.8.5.5 <u>Penetrations</u>

The design of penetrations is discussed in Subsection 5.8.6.4.

5.8.6 PENETRATIONS

5.8.6.1 Design Basis

Penetrations in the containment structure provide access and egress for personnel and equipment and for piping, ventilation and electrical systems. These penetrations are designed to maintain the leak tightness of the containment structure under normal and accident conditions. They are designated as CP Co Design Class 1 components of the containment structure.

5.8.6.2 General Description

Throughout this chapter the term "penetration" is used to mean either:

1. The steel component, consisting of a "penetration assembly" and a mechanism for sealing the shell opening, or

2. The opening in the concrete shell

The correct meaning can be determined from the context in which the term is used.

The term "penetration assembly" describes the steel assembly composed of a penetration sleeve, a thickened liner plate and anchor bolts. For piping and ventilating penetrations, the assembly also includes a circular or conical stub ring. For some assemblies, including those for the main steam, feedwater, equipment hatch and air locks, a 1/4-inch plate surrounds the openings and is welded to the penetration sleeve and the thickened liner plate. Penetration assemblies were shop fabricated and field welded to the containment liner plate prior to placement of the adjacent concrete.

Mechanisms for sealing the shell openings vary depending on the type of penetration. The personnel and escape air locks are closed cylindrical units inserted in and field welded to their penetration assemblies. The equipment hatch consists of a door and door frame field welded to its penetration assembly. Piping and ventilation penetrations incorporate a steel closure which is field welded to both the stub ring and the process line. (Note: That portion of the process line that passes through the containment shell is not considered to be part of the "penetration.") Electrical penetrations consist of electrical canisters inserted in and field welded to their penetration assemblies.

All penetration welds necessary to maintain the leak-tight integrity of the containment were originally specified as full penetration welds. Penetration welds performed subsequent to original construction need not be full penetration welds provided they are demonstrated to be structurally acceptable.

Exterior portions of all containment penetrations are located in enclosures that are heated, with the exception of the Emergency Escape Air Lock (Reference 43), and tornado protected. Penetrations are described within the following functional categories:

1. Personnel and equipment openings - the personnel air lock, the escape air lock and the equipment hatch

2. Other openings - penetrations for piping, ventilation and electrical systems

5.8.6.2.1 Personnel and Equipment Openings

The personnel air lock connects the containment building with the 611-foot level of the auxiliary building (see Figure 1-4). The air lock consists of a steel cylinder with 3-foot 6-inch x 6-foot 8-inch doors at each end interlocked so that only one door can be open at any time. The air lock is designed to withstand all containment structure design conditions with either or both doors closed and locked. Doors open toward the center of the containment building.

Double gaskets are provided to seal each door. This permits periodic pressurizing of the space between the gaskets for testing the gasket seal. The air lock barrel may be pressurized to test its leak tightness without pressurizing the containment building. For this test, auxiliary restraint beams are attached to the inner door to help the locking bars resist the resulting internal lock pressure, which is greatly in excess of the containment structure external design pressure of 3 psig. The personnel air lock was pneumatically shop tested to ensure that it met the design requirements for pressure capacity while simultaneously maintaining its leak-tight integrity.

Operation of the personnel air lock is manual. Operating procedures require personnel using the air lock to close the door behind them. Figure 5.8-6 shows the principal features of the personnel air lock. (Note that the electrohydraulic mechanism which permitted operation of the far door was removed in 1984.)

The escape air lock, with 30-inch-diameter doors, connects the containment building with the 625-foot level of the electrical penetration enclosure (see Figure 1-5). Its features are identical to the personnel air lock except that no electrohydraulic mechanism is provided for door operation and pressurizing the space between the door seal gaskets is not normally performed. Figure 5.8-7 shows the principal features of this air lock.

The 12-foot-diameter equipment hatch connects the reactor refueling floor with the spent fuel floor in the auxiliary building (see Figure 1-6). The door is secured by bolts on the inside of the containment shell and can be opened only from inside the containment building. It is opened only when the reactor is at cold shutdown. Double gaskets on the door permit its seals to be pressurized from outside the containment to check the integrity of the seals. Figure 5.8-8 shows the principal features of this door. The spent fuel pool enclosure outside the equipment hatch is heated but is not designed to protect against tornado missiles. A reinforced concrete structure does protect the equipment hatch against tornado missiles from all directions except to the north. As noted in Section 5.5.1.1.1, the equipment hatch itself is not vulnerable to the only credible tornado missile that can directly strike the equipment hatch from the north. Although not designed for tornado missile protection, the radiation shield wall located in front of the equipment hatch would provide some degree of tornado protection to the hatch.

Local leak rate tests are periodically performed on the air locks and equipment hatch in accordance with 10 CFR 50, Appendix J (see Subsection 5.8.8.2).

5.8.6.2.2 Other Openings

All piping and ventilation penetrations are of the rigid welded type and are solidly anchored to the containment shell, thus precluding any requirement for expansion bellows. Pipe whip restraints (external guides and stops) are provided to limit displacements which, if left unrestrained, might create bending and torsional stresses large enough to rupture the penetration. Piping and ventilation penetrations have no provision for individual testing. For the main steam and feedwater penetrations, forced-air cooling is provided in the air gap between the insulated pipe and the penetration sleeve. For typical details of piping penetrations, see Figure 5.8-2.

Typical original electrical penetrations consist of penetration assemblies and carbon steel pipe canisters with stainless steel header plates attached to each end. Identical, hermetically glass-sealed multipin connectors are attached into both headers for all conductors rated less than 600 volts. High-voltage conductors utilize single conductor, hermetically sealed, ceramic bushings attached to both header plates. Thus, each canister affords a double barrier against leakage. A flange on each canister is field welded to the penetration assembly. Conduction and radiation paths are sufficient to prevent damage to seals or conductors during field welding.

The canister design permits pressure and leakage tests to be performed simply and reliably both at the shop and after installation. A tap is provided for pressurizing the canister from outside the containment building.

The terminations of the conductors to the connectors inside the canisters are potted to protect against moisture. A drain plug was originally provided to permit draining of condensed moisture and, in conjunction with the pressure tap, to permit purging with dry nitrogen.

During plant construction, the plug in the pressure tap of each electrical penetration canister was removed and a nitrogen blanket supply system was attached in order to limit corrosion. The drain plugs were left installed.

Each electrical penetration room has its own nitrogen blanket supply system that provides low pressure nitrogen to the internals of the penetration canisters. The nitrogen systems are shown on Figure 5.8-31. The nitrogen supply systems incorporate a supply check valve to maintain a double barrier against leakage. The tubing manifold system is also used to perform Local Leak Rate tests of the electrical penetrations.

For typical details of electrical penetrations, see Figure 5.8-3.

5.8.6.3 Design Criteria

5.8.6.3.1 Concrete Openings

The concrete openings for the equipment hatch and personnel air lock were designed in accordance with the provisions of ACI 318-63, Building Code Requirements for Reinforced Concrete, except as noted in Subsections 5.8.5.2.3 and 5.8.5.2.4. These openings were designed for the containment structure load combinations associated with both the working stress and yield strength conditions (see Subsection 5.8.3.1).

5.8.6.3.2 Steel Penetrations

Applicable Standards

Penetrations conform to the applicable sections of ASA N6.2-1965, "Safety Standard for the Design, Fabrication and Maintenance of Steel Containment Structures for Stationary Nuclear Power Reactors." The personnel air lock, the escape air lock, and all portions of the equipment hatch extending beyond the concrete shell conform in all respects to the requirements of the ASME B&PV Code, Section III. Piping, ventilating and electrical penetrations are designed, fabricated, inspected and installed in accordance with the ASME B&PV Code, Section III, Subsection B.

The basis for limiting strains in the penetration steel is the ASME B&PV Code, Section III, Article 4, 1965 with the exception of penetrations 5, 6, 16, and 55, which were analyzed in accordance with the ASME B&PV Code, Section III, Subsection NC, 1986 Edition. Based on this, the penetration structural and leak-tight integrity is maintained.

The stress level in anchor bolts for penetration assemblies is in accordance with the AISC Code.

Loads and Load Combinations

The following loads were considered in the design of the steel penetration:

1. Concrete dead load or penetration self-weight (where appropriate)

2. Prestress (includes creep and shrinkage effects)

3. Thermal gradients (thermal loads via the process pipe)(a)

4. Operating temperature

5. External pressure (maximum pressure in the open-ended annulus between the pipe and the penetration resulting from the rupture of any one process line; main steam and feedwater penetrations excluded)(a)

6. Pipe rupture reactions (thrust, moment and torque resulting from rupture of the penetration's process line either inside or outside containment)(a)

7. Test pressure

8. DBA pressure

9. DBA temperature

10. OBE

(a) Applicable only to pipe penetrations

The following comments apply to the choice of loads used in the design of the steel penetrations:

1. Loads 5 and 6 were considered only in conjunction with the normal operating condition (see Equation 1) since a pipe failure adjacent to the liner was not assumed to occur simultaneously with the DBA accident.

2. Wind loads and tornado missiles were not considered. All penetrations are located inside CP Co Design Class 1 enclosures.

3. Jet impingement and localized heating due to pipe ruptures were not considered for the following reasons:

   a. No jet from the Primary Coolant System could reach the containment wall since all possible jets are intercepted by the steam generator compartment walls.

   b. No safety hazard would result from damage of the liner plate caused by rupture of pipes outside the Primary Coolant System. Should such damage occur, the Plant would be shut down for repair of the affected liner plate.

Steel penetrations were designed for the following working stress load combinations:

1. $D + F_f + L + T_o + P_r + R_p$

2. $D + F_f + L + P + T_a + E(\text{or } W)$

3. $D + F_f + L + P'$

In addition, the equipment hatch and personnel air lock were also designed for the following working stress load combinations, as noted:

4. $D + F_f + T_o$ equipment hatch

5. $D + 1.5P + F_f + T_o$

6. $D + F_f + P + T_o$ personnel air lock

Where:

D = dead load

L = appropriate live load

F_f = final prestress loads (defined in Subsection 5.8.5.3.1)

P = DBA pressure load

P_r = external pressure (maximum pressure in the open-ended annulus between the pipe and the penetration resulting from the rupture of any one process line; main steam and feedwater penetrations excluded)

R_p = pipe rupture reactions (thrust, moment and torque resulting from rupture of the penetration's process line either inside or outside containment)

T_o = thermal loads due to operating temperature

T_a = thermal loads due to DBA temperature

E = OBE load resulting from a ground surface acceleration of 0.1 g

P' = test pressure (1.15 x DBA pressure)

W = wind load resulting from a 100 mi/h wind

Penetration assemblies and pipe penetrations (assembly plus closure) were designed to withstand the forces and moments resulting from the loading combinations described above. Pipe whip restraints (external stops and guides), increased pipe thickness or other means are provided to make the penetrations the strongest part of the system. Pipe whip restraints limit displacements, which if left unrestrained, might create bending and torsional stresses large enough to rupture pipe penetrations. The main steam and feedwater piping, between a point inside the attachment of the process line to the penetration and the containment isolation valves located outside containment, was designed as the strongest piping in each system. Use of thickened pipe sections, in conjunction with special quality control and special nondestructive testing (both conducted during Plant construction) and an augmented inservice inspection program, ensures that rupture of these systems will not occur within these regions.

Construction Details

Materials used in construction of the penetrations are discussed in Subsection 5.8.7.1. Construction quality control is discussed in Subsection 5.8.7.2.

5.8.6.4 <u>Analysis and Design</u>

In general, special consideration is given to all openings in the containment structure. The degree of analysis and design attention required for various openings increases as the penetration size increases.

For the purpose of analysis and design, small penetrations are defined to be those openings in the concrete shell with diameters less than or equal to eight feet. Openings for all piping, ventilation, and electrical penetrations and the opening for the escape air lock are classified as small penetrations. Large penetrations are defined to be those openings with diameters greater than eight feet. Openings for the personnel air lock and equipment hatch are classified as large penetrations.

5.8.6.4.1 Small Penetrations

Concrete Shell

Reference 4 indicates that the curvature of the shell has a negligible effect upon the resulting stresses for openings with diameters smaller than 2-1/2 times the shell thickness. For Palisades, this thickness criteria was interpreted to mean openings with diameters less than or equal to 8 feet (small penetrations, as defined above). Therefore, small penetrations were analyzed as holes in a plane sheet.

In general, the shell thickness was found to be capable of withstanding the imposed stresses upon the addition of supplementary reinforcement, and the thickness was increased only where additional space was required for the radially deflected tendons. The stresses created by normal thermal gradients and postulated pipe rupture conditions fall off rapidly from the concrete surface on which the loads are applied; these stresses are not significant in comparison to the stresses produced by the many load combinations for which the shell was analyzed. These stresses are distributed through the shell in accordance with Reference 5.

The only high-temperature lines that penetrate the containment structure are the main steam and feedwater lines. Using the generalized heat transfer program, steady-state temperature gradients were determined for these lines for the case of no cooling and maximum insulation. The results indicated that no cooling was necessary. Figure 5.8-18 illustrates the thermal gradients around the main steam penetration. However, forced-air cooling has been provided in the air gap between the insulated pipe and the penetration sleeve for these penetrations.

Subsequent analysis, determined that due to the increased ambient containment temperatures specified in the EEQ program, the maximum boundary temperature in the main steam penetration room near the main steam penetration increased from, 104°F to 110°F, and the reactor containment temperature near the main steam penetration increased from 104°F to 110°F (Reference 29). The analysis concluded the maximum temperature of the concrete at the main steam penetration increased from 143°F to <179°F which is less than the limit imposed by ACI 349 of 200°F and provides adequate margin.

Local heating of the concrete immediately around these and other penetrations develops a compressive stress in the concrete adjacent to the penetration and a negligible amount of tensile stress over a large area. Mild reinforcing steel is added around the affected penetrations to distribute the compressive stresses.

Penetrations

Stress concentrations around openings in the liner plate were calculated using the theory of elasticity. These stress concentrations were then reduced by thickening the liner plate around each penetration in accordance with the ASME B&PV Code, Section III, 1965.

Anchor bolts are provided as part of each penetration assembly. When the penetration assembly has no significant external loads, the anchors maintain the strain compatibility between the liner plate and the concrete. When significant loads are present, the anchors control the inward displacement of the liner plate. The stress level in the anchor bolts from external loads is in accordance with the AISC Code.

Pipe penetrations were analyzed for the external loads and load combinations discussed in Subsection 5.8.6.3.2. These loads produce stresses which are categorized as "primary," "secondary" or "peak" stresses by the ASME B&PV Code, Section III, Article 4. For each load combination, the following operations were performed in accordance with Figure N-414 of the referenced ASME Code article:

1. Stresses in the same category were added.

2. The resulting stresses from (1) were combined using the code prescribed load combinations.

3. The combined stresses of (2) were compared to the appropriate stress intensity (Sm, Sa or multiple thereof). The allowable value of stress intensity Sa was determined from Figure N-415(a) of the referenced ASME Code article.

The pipe penetrations were designed such that the allowable stress intensities were not exceeded. Figure 5.8-19 shows a typical pipe penetration and the applied loads.

Under the action of the pipe rupture reactions, the pipe penetration stresses were limited to 0.9 times the yield strength of the material. However, when combined with the stresses from other loads, according to the load combinations of Subsection 5.8.6.3.2, the penetrations yielded. The highest state of strain will exist as a flexural strain in the connection of the penetration to the process pipe. Under DBA conditions, the process pipe will tend to grow radially while the restraining plate will tend to expand into the pipe, thus creating a ring load on the pipe. This phenomenon is self-limiting. For all load combinations, the strains in the pipe penetrations did not exceed the values given in the ASME B&PV Code, Figure N-415(a).

The ASME B&PV Code, Section III, requires that each penetration assembly be stress relieved as a unit. Hence, the assemblies were shop fabricated and shop stress relieved. Where many penetrations were closely spaced, it was decided to fabricate a single, large shop assembly. The emphasis on shop fabrication resulted in a minimum of field welding at the penetrations.

5.8.6.4.2 Large Penetrations

Concrete Shell

The analyses for the equipment hatch and personnel air lock openings, which are discussed below, were all linear elastic analyses. These analyses were performed for the containment structure load combinations associated with both the working stress and yield strength conditions (see Subsection 5.8.3.1). For both sets of load combinations, the design employed the working stress concept of a linear stress profile. Details of the concrete design are discussed in Subsection 5.8.5.2.

1. Major Loads

   The major loads used for the analysis and design of the concrete around both the equipment hatch and the personnel air lock openings were dead, earthquake, pressure, prestress and thermal. The primary loads listed are mainly membrane loads with the exception of the thermal loads. In addition to membrane loads, accident pressure also produces punching shear around the edge of these openings. The values of these loads for design purposes were the magnitudes of these loads at the center of these openings. These are fairly simple to establish knowing the values of hoop and vertical prestressing, accident pressure, and the geometry and location of these openings.

2. Secondary Stresses

Under the action of the applied loads, secondary stresses occur around the equipment hatch and personnel air lock openings due to the placement of the tendons and the geometry of the shell. The stresses can be significant. The magnitude of these secondary stresses, or their associated stress resultants, was predicted as described below:

a. Tendon Deflection

The effect of the deflection of the tendons around the equipment hatch opening on the membrane stress concentration factors was analyzed for a flat plate by the finite element method. The resulting stresses were compared with those predicted by conventional stress concentration factors. This comparison demonstrated that the deflection of the tendons does not significantly affect the stress concentrations. This plane stress analysis did not include the effect of the curvature of the shell. However, it gives an assurance of the correctness of the assumed membrane stress pattern caused by the prestressing around the opening. Results of this analysis are shown in Figure 5.8-20.

b. Shell Curvature

Stress resultants around these openings were found for various loading cases with the help of Reference 4. Comparison of these results with the results of a flat plate of uniform thickness with a circular hole showed the effect of the cylindrical curvature on stress concentrations around the opening.

Normal shear stress resultants (relative to the opening) were modified to account for the effect of twisting moments as shown in Reference 4. These modified shear forces are called Kirschoff's shear forces. Horizontal wall ties are provided to resist a portion of these shear forces.

c. Shell Thickening

The effect of thickening the outside face around these openings was considered using several methods. Reference 6 was used to evaluate the effect of thickening on stress concentration factors for membrane stress. A separate axisymmetric finite element computer analysis for a flat plate with anticipated thickening on the outside face was prepared to handle both axisymmetric and nonaxisymmetric loads. It was used to predict the effect of the concentration of hoop tendons at the top and bottom of these openings.

3. Thermal Stresses

   A uniformly distributed moment, equal but opposite to the thermal moment existing on the rest of the shell, was applied at the edge of the equipment hatch and personnel air lock openings and evaluated using the methods described in Reference 4. The resulting stresses were then superimposed on the stresses calculated from the other loads.

   In the case of 1.5P (prestress fully neutralized) + 1.OT_a (accident temperature), the cracked concrete with highly strained tension reinforcement constitutes a shell with stiffness decreased but still essentially constant in all directions. In order to control the increased hoop moment around these openings, the hoop reinforcement (concentric circular bars) is about twice that of the radial reinforcement (see Figure 5.8-4).

4. Governing Design Condition

   The DBA is the governing design condition for the outside face of the thickened shell surrounding the equipment hatch and personnel air lock openings. Reinforcement is provided adjacent to the equipment hatch opening to accommodate the sum of the following stresses:

   a. Thermal stresses, approximately 60% of the total

   b. Normal stresses resulting from membrane forces, including those caused by shell thickening, approximately minus 14% of the total

   c. Flexural stresses resulting from the moments caused by shell thickening, approximately 60% of the total

   d. Normal and flexural stresses resulting from membrane forces and moments caused by cylindrical curvature, approximately minus 6% of the total

5. Tendon Deflection

   The inside row of vertical tendons is deflected outward as it is curved around the equipment hatch and personnel air lock openings (see Figure 5.8-4). This outward deflection produces several localized effects. First, it induces local bending stresses which reduce the effect of the tensile stresses near the outside face. Second, it reduces the magnitude of the inward acting radial forces (directed toward center of containment building) at the top and bottom of these openings. The concentration of these forces results from the accumulation of hoop tendons which are bent around these openings. And third, it induces inward acting radial forces at the sides of these openings.

6. Shell Thickening

The concrete shell was thickened at the equipment hatch and personnel air lock openings for the following reasons:

a. To reduce predicted membrane stresses around the opening to acceptable levels

b. To accommodate tendon placement

c. To accommodate reinforcing steel placement

d. To compensate for reduction in the overall shell stiffness due to the opening

5.8.6.4.3 Other Design Details

Reinforcement and Prestressing Tendons

At penetrations, additional reinforcement is provided to resist local membrane and flexural forces. This reinforcement takes the form of a horizontal and vertical grid on the inside and outside face of the concrete, and for openings over 3 feet in diameter it takes the form of bars that encircle the opening. For the equipment hatch, personnel air lock, escape air lock and fuel transfer tube penetrations, the perpendicular grid is ASTM A-432 and the circular bars, with the exception of those for the equipment hatch, are ASTM A-15. Around this hatch, A-432 circular reinforcement is mechanically spliced using T-series CADWELDS. A-432 high-strength steel is utilized because it provides an additional margin of elastic strain capacity compared to A-15. At the high-temperature openings (feedwater and main steam), additional A-15 steel is used to distribute thermal compressive stresses. Also, A-15 stirrups are used, as required, to assist in resisting shear forces.

The deflected tendons are continuous past all penetrations. Because the tendons are unbonded, local effects of stress concentrations on the tendons are insignificant.

Local crushing of the concrete due to deflection of the bonded reinforcement or the prestress tendons is precluded by the following details:

1. The surface reinforcement either has a very large radius, such as the hoop bars concentric with the penetration, or it is practically straight, having only standard hooks where necessary for anchorage.

2. The tendons are bent around the penetrations at a minimum radius of approximately 20 feet. The maximum tendon force, approximately 850 KIPs, occurs during the post-tensioning operation (see "jacking stress" in Subsection 5.8.5.3.1). Due to the curvature of the tendons, this load resulted in a maximum bearing stress (tendon on concrete) of about 880 psi.

Personnel Air Lock and Escape Air Lock

The seismic analysis of these locks is discussed in Subsection 5.7.7.5.

Design live load on the floor of the personnel air lock was 200 lb/ft^2.

5.8.7 CONSTRUCTION

Throughout this subsection, the applicable code or standard was the one in effect at the time the work was done.

5.8.7.1 Materials

5.8.7.1.1 Concrete

1. Ingredients

   Cement - ASTM C-150, Type II

   Fly Ash - ASTM C-150

   Air Entraining Agent - ASTM C-260

   Water Reducing Agent - ASTM C-494, Type D (Pozzolith 8, Improved)

   Aggregate - ASTM C-33 (Fine and Coarse Aggregate Is Crushed Dolomite)

   Walter Flood and Co performed concrete strength and shrinkage tests with various reducing agents to establish the particular additive with the most desirable characteristics for this application. On the basis of these tests, Pozzolith 8, Improved, manufactured by Master Builders, was selected.

Calcium chloride was not used in the concrete mixes. Additionally, all the concrete ingredients were chemically analyzed prior to use. The results of these analyses showed no significant amounts of chloride were present.

2. Strengths

Base Slab - 4,000 Psi at 90 Days

Walls and Dome - 5,000 Psi at 28 Days

3. Principal Placement Properties

Slump, Maximum - 2-1/2 Inches at the Form

Air Content - 3% to 5% at the Mixer

Temperature, Maximum – 70°F

4. Concrete Design Mixes

Concrete Design Strength	Cement (Sk/yd^3)	Fly Ash (Sk/yd^3)	Sand	Aggregates (lb/yd^3) 3/4"	1-1/2"	3"	Water
4,000 Psi	4.13	1.37	1,420	1,960	-	-	234
@ 90 Days	3.94	1.32	1,283	1,024	1,091	-	228
	3.74	1.26	1,149	662	761	906	217
5,000 Psi	5.32	0.94	1,333	1,918	-	-	259
@ 28 Days	5.10	0.90	1,227	996	1,095	-	250

Mixes were designed and tested in accordance with ACI 613 by Walter Flood and Co. During construction, the field inspection personnel made minor modifications that were necessitated by variations in aggregate gradation or moisture content.

5.8.7.1.2 Reinforcing Steel

ASTM Specifications for reinforcing steel are the following:

A-15 Billet Steel - Intermediate Grade (fy = 40 Ksi)

A-432 Billet Steel - High Strength (fy = 60 Ksi)

5.8.7.1.3 Prestressing Tendons and Hardware

Tendon Wires	ASTM A-421, Latest
Bearing Plate	ASTM A-36, Latest
Anchor Head	AISI 1141, Special Quality
Shims	AISI 1141, Special Quality
Tendon Sheathing, 24 Gauge	ASTM A-366-66T

Bearing plate material ordered from the mills under ASTM A-36 was made to fine grain practice from silicon killed heats. The bearing plates in the post-tensioning gallery are 20/35 hot rolled steel rather than ASTM A-36. This material is acceptable since the physical and chemical properties conform to ASTM A-36. However, the HR20/35 plates may not be made to fine grain practice.

5.8.7.1.4 Liner Plate

The liner plate conforms to ASTM A-442, Grade 55, flange quality. This material was selected in preference to ASTM A-36 in view of its greater ductility and its improved chemistry which provides better weldability. Steel conforming to ASTM A-442 is tested on a per-plate basis rather than each heat as for A-36, therefore giving better material control and greater reliability.

5.8.7.1.5 Steel Penetrations

1. Elements Resisting Containment Pressure

   Pipe Sleeve Material - ASTM A-333 (ASME SA516 GR 70 for penetrations 5 and 6)

   Plate Material - ASTM A-516, Grade 70, Firebox Quality (ASME SA516 GR 70 for penetrations 5, 6, 16 and 55)

   For both of the above materials, impact specimens were Charpy V-notch tested at a temperature of 0EF in accordance with the ASME B&PV Code, Section III, Paragraph 1211(a).

2. Miscellaneous

Penetration Anchor Bolts - ASTM A-307, Grade A

Penetration HS Anchor Bolts - ASTM A-193, Grade B7

Steel Arc-Welding Electrode - ASTM A-233 and A-559, Type E6010

Truss Bolts - ASTM A-325-64

Structural Steel for Inserts and Supports - ASTM A-36-63T

5.8.7.1.6 Sheathing Filler

The tendon sheathing filler was required to meet certain limitations for deleterious water soluble salts. The product selected was Visconorust 2090P, manufactured by Viscosity Oil Company. Improved corrosion resistant sheathing fillers, Visconorust 2090P-2 and 2090P-4, have been used for subsequent tendon surveillances.

Temporary corrosion protection of the tendons and the interior face of sheathing was applied prior to shipment.

5.8.7.2 Quality Control

5.8.7.2.1 Concrete Mix Design

1. Aggregates

Aggregates to be used in the design mix were tested to assure compliance with ASTM C-33 and to provide data for the design mix calculations. The following tests were performed by Walter Flood and Co:

Test	ASTM
Los Angeles Abrasion	C-131
Clay Lumps Natural Aggregate	C-142
Material Finer Than #200 Sieve	C-117
Mortar-Making Properties	C-87
Organic Impurities	C-40
Potential Reactivity (Chemical)	C-289
Potential Reactivity (Mortar Bar)	C-227
Sieve Analysis	C-136
Soundness	C-88
Specific Gravity and Absorption	C-127
Specific Gravity and Absorption	C-128
Petrographic	C-295

2. Mixes

Mixes were designed and tested in accordance with ACI 613 by Walter Flood and Co. The following tests were run to evaluate the design mixes shown in Subsection 5.8.7.1.1:

Test	ASTM
Air Content	C-231
Slump	C-143
Bleeding	C-232
Making and Curing Cylinders in Lab	C-192
Compressive Strength Tests	C-39

5.8.7.2.2 Concrete Materials

In addition to the tests used to qualify materials for concrete mix designs, the following periodic tests were made:

1. Cement

Mill tests, verifying compliance with the specification, were furnished by the cement manufacturer with each shipment of cement to the job site batch plant. In addition, grab samples were taken and tested by Walter Flood and Co for compliance with ASTM C-150.

2. Fly Ash

Fly ash was sampled and tested in accordance with ASTM C-311. The type and frequency of testing were as follows:

Fineness, strength, drying, shrinkage, soundness, uniformity requirements, sulfur trioxide and loss of ignition - every 100 tons received

Silicon dioxide, magnesium oxide and available alkalies - every 1,000 tons received

3. Aggregates

Sieve analyses were performed weekly to verify that acceptable gradation was maintained. Field laboratory tests were performed for reactivity, Los Angeles abrasion, and soundness on samples from each 2,500 cubic yards of aggregate used in CP Co Design Class I structures.

5.8.7.2.3 Concrete

Six concrete compression cylinders were cast for every 100 cubic yards placed. The cylinders were tested according to the following schedule:

2 at 7 days, 2 at 28 days, 2 at 90 days

A slump test was made for every 35 cubic yards of concrete placed. Each month the coefficient of variation of compressive strength was computed to determine the consistency of concrete production.

5.8.7.2.4 Reinforcing Steel and CADWELD Splices

Mill test reports were furnished by the fabricator. In addition, samples were periodically cut from reinforcing steel in the field and sent to an independent testing laboratory for a check.

Prior to placing, a visual inspection of the shop fabricated reinforcing steel was performed to ascertain the dimensional conformance with design specifications and drawings. This was followed by a check "in place" to assure both dimensional and location conformance.

All personnel engaged in the making of CADWELD splices were trained and supervised by the manufacturer's representative and had passed all the necessary qualification tests and procedures before production splicing.

Prior to splicing operations, bar ends were inspected for damaged deformations. All completed splices were visually inspected at both ends of the splice sleeve and at the tap hole to verify that the filler material was sound, nonporous, recessed no more than 1/4 inch from the sleeve end, and present for the full 360 degrees.

Randomly selected splices were tensile tested in accordance with the following schedule:

1 out of first 10 splices

3 out of next 100 splices

2 out of next and subsequent units of 100 splices

The strength of the CADWELD joints, as verified by tests on sample bars, was greater than 125% of the ASTM specified minimum yield strength of the reinforcing bars used. Also, the average strength of all test splices exceeded the ASTM ultimate strength of the reinforcing bar used.

5.8.7.2.5 Prestressing Tendons and Hardware

Flame cutting and welding were required to fabricate the bearing plates. Flame cutting followed the prestressing system supplier's procedure. Welding was done by qualified welders in accordance with the provisions of the American Welding Society Code for Welding in Building Construction, Part II, Article D, AWS D1.0-63. No flame cutting or welding was done on any other prestressing system component.

The AISI 1141 steel furnished to date had the following properties before heat treating:

	Yield Strength (Psi)	Tensile Strength (Psi)	Elongation in 2" Gauge Length
Shop Anchor Head	57,000	108,000	10.5%
Field Anchor Head	88,300	120,000	13.5%
Shop Anchor Head Bushing	58,430	95,980	27%

The test method used to determine the above properties was ASTM A-370. The bearing surface of the anchor heads after heat treating had Rockwell "C" values between 29 and 33. Anchor head bushings were not heat treated.

Sampling and testing of tendon wires conformed to ASTM A-421. Tendons and anchorage assemblies were shipped with tags identifying each lot and mill heat. From each shipping release, one completely fabricated prestressing test specimen consisting of a five-foot-long tendon, including anchorages at both ends, was tested. The anchorages developed the minimum guaranteed ultimate strength of the tendons and the minimum elongation of the tendon material. In addition, dynamic tests of five anchorage assemblies were run.

All prestressing system installation work was continuously inspected by a qualified engineer. All measuring equipment used for the installation was calibrated and certified by an independent testing laboratory. Proper tendon stress was achieved by comparing both the jack pressure and the tendon elongation against previously calculated values.

5.8.7.2.6 Liner Plate

The tolerances on liner plate erection were as follows:

The radial location of any point on the liner plate did not vary from the design radius by more than ± 2-1/2 inches. A 15-foot-long template curved to the required radius was used to verify that the following tolerances were not exceeded:

1. A maximum 3/4-inch deviation when placed against the completed surface of the shell within a single plate section

2. A maximum 1-inch deviation when placed across one or more welded seams

The maximum inward deflection (toward the center of the structure) of the 1/4-inch plate, between the angle stiffeners spaced at 15 inches, was 1/16 inch when measured using a 15-inch straightedge placed horizontally, and 1/8 inch when placed across the welded seam at the buttresses.

The qualifications of all welding procedures and welders were performed in accordance with the ASME Boiler and Pressure Vessel Code, Section IX, Part A (ASME B&PV Code, Section IX, Part A). All liner angle welding was visually inspected prior to, during and after welding to ensure that both the quality and the general workmanship met the requirements of the applicable welding procedure specification.

The techniques for radiographic inspection and the acceptance of welds were in accordance with the ASME B&PV Code, Section VIII. Welds of questionable quality and nonradiographable joints were inspected by dye penetrant or magnetic particle testing in accordance with the ASME B&PV Code, Section VIII.

During the welding operation, a minimum of 10% of the work, randomly selected by the inspector, was spot radiographed using a 12-inch length of film. Where radiographs disclosed cracks or lack of fusion, the weld was rejected. Where radiographs indicated that welding flaws might be present, two additional spots, 12 inches long, were examined at locations away from the original spot. If these spots were satisfactory, then the welding represented by the three radiographs was considered to be satisfactory. If these additional spots were unsatisfactory, the entire weld seam associated with the three radiographs was removed and replaced. Then, using the criteria above, the rewelded seam was inspected to determine whether the weld was satisfactory.

All liner plate welding which is required for leak-tight integrity was vacuum box soap bubble tested. A soap solution containing equal parts of corn syrup, liquid detergent and glycerin was applied over the welds to be tested. A vacuum box containing a window was placed over the area to be tested and evacuated to produce approximately a 5 psig pressure differential. Any appearance of soap bubbles indicated deficient welding. Seams in the floor liner plate were also checked by pressurization of the leak channels. These channels are divided into areas. Any leakage was indicated by pressure decay.

Additional inspection was performed on the liner plate welds below elevation 610 feet as part of the investigation program to determine the extent and cause of horizontal butt-weld cracking which occurred during liner plate erection (see Subsection 5.8.7.4). This program also included chemical testing on several liner plate specimens. This testing was conducted by Anamet Laboratories, Inc, Berkeley, California (Reference 7). On the basis of the test results, it was concluded that the liner plate material met the chemical requirements of ASTM A-442.

5.8.7.2.7 Steel Penetrations

Penetration sleeve-to-penetration cap/head welds were inspected with either dye penetrant or magnetic particle tests.

5.8.7.2.8 Sheathing Filler

The supplier performed tests for every batch of factory production and furnished the certified results. At the job site, a representative sample was taken from every shipment and tested for the water soluble salts using ASTM D-512-62T, D-1255 and D-992-52.

5.8.7.3 <u>Construction Methods</u>

5.8.7.3.1 Governing Codes

The following codes of practice were used to establish standards of construction procedure:

ACI 301 - Specification for Structural Concrete for Buildings (Proposed)

ACI 315 - Manual of Standard Practice for Detailing Reinforced Concrete Structures

ACI 318 - Building Code Requirements for Reinforced Concrete

ACI 347 - Recommended Practice for Concrete Framework

ACI 605 - Recommended Practice for Hot Weather Concreting

ACI 613 - Recommended Practice for Selecting Proportions for Concrete

ACI 614 - Recommended Practice for Measuring, Mixing and Placing Concrete

ASME - Boiler and Pressure Vessel Code, Sections III, VIII and IX

AISC - Steel Construction Manual

PCI - Inspection Manual

5.8.7.3.2 Concrete

Cast-in-place concrete was used to construct the containment shell. The base slab construction was performed in seven large block pours. After the completion of the base slab and steel liner erection and testing, an additional 18-inch-thick concrete slab was placed to provide protection for the floor liner.

The concrete placement in the walls was done in 10-foot-high lifts with vertical joints at the radial center line of each of 6 buttresses. Cantilevered jump forms on the exterior face and the interior steel liner plate served as the forms for the wall concrete. After cracking was discovered in the liner plate welds, the sequencing of wall pours was coordinated with the liner plate erection (see Subsection 5.8.7.4.1).

The dome liner plate, temporarily supported by 18 radial steel trusses and purlins, served as an inner form for the initial 7-inch-thick pour in the dome. The weight of the subsequent pour was supported in turn by the initial 7-inch pour. The trusses were lowered away from the liner plate after the initial 7 inches of concrete had reached design strength, but prior to placing the balance of the dome concrete.

The horizontal and the vertical construction joints were prepared by dry sandblasting followed by cleaning and wetting. Immediately prior to concrete placement, the horizontal surfaces were covered with approximately a 1/4-inch-thick mortar of the same cement-sand ratio as used in the concrete.

5.8.7.3.3 Reinforcing Steel and CADWELD Splices

Whenever required, mechanical splices were made by the CADWELD process using clamping devices, sleeves, charges, etc, as specified by the manufacturer for "T" series connections. Prior to splicing operations, bar ends were power brushed to remove all loose mill scale, rust and other foreign material. Immediately before the splice sleeve positioning, bar ends were preheated to ensure a complete absence of moisture.

5.8.7.3.4 Prestressing System

Tendons were delivered to the site coated with a temporary rust preventative and encased in polyethylene bags. Each tendon came precut to exact length, with one end unfinished and the other end shop buttonheaded, and with its anchor head attached.

The tendon installation procedure was carried out as follows:

1. To assure a clear passage for the tendons, a "sheathing rabbit" was run through the sheathing both prior to and following placement of the concrete.

2. Tendons were uncoiled and pulled through the sheathing, unfinished end first.

3. The unfinished end of the tendons was pulled out with enough length exposed so that field attachment of the anchor head and buttonheading could be performed. To allow this operation, trumplates on the opposite end had an enlarged diameter to permit pulling in the shop-finished ends with their anchor heads.

4. The anchor heads were attached and the tendon wires buttonheaded.

5. The shop-finished end of the tendon was pulled back and the stressing jacks were attached.

6. All tendons were post-tensioned until they reached a code allowable, nominal stress level of 0.80 f_s'. Shims, which were precut to match calculated elongations, were installed and the tendons were seated at a nominal stress level of 0.73 f_s'. Proper tendon stress was achieved by comparing both jack pressure and tendon elongation against previously calculated values. The vertical tendons were posttensioned from one end, while the horizontal and dome tendons were posttensioned from both ends.

7. The grease caps were bolted onto anchorages at both ends and made ready for pumping the tendon sheathing filler material.

8. The tendon sheaths and grease caps were filled with sheathing filler and sealed off.

A tensioning sequence was developed to minimize cracking during the post-tensioning operation and to allow the construction opening to remain after the tensioning had started. A summary of this sequence is as follows:

1. Bands of hoop tendons were tensioned starting approximately 20 feet above the construction opening and extending up to just below the ring girder. The number of bands, the number of tendons tensioned within each band, and the sequence for tensioning bands were all chosen to minimize meridional bending.

2. Dome tendons were tensioned in a sequence which provided balanced tensioning around the periphery of the dome.

3. The construction opening was closed. After the concrete gained the required strength, a portion of the hoop tendons from approximately 20 feet above the construction opening down to the base slab was tensioned.

4. Vertical tendons were tensioned in a sequence which provided balanced tensioning around the periphery of the cylinder.

5. All remaining hoop tendons were tensioned in a sequence chosen to minimize meridional bending.

5.8.7.3.5 Liner Plate

Construction of the liner plate conformed to the applicable portions of the ASME B&PV Code, Section VIII, Part UW. Specifically, Paragraphs UW-26 through UW-38, inclusive, applied in their entirety.

The erection of the liner plate was as follows:

After the floor insert plates on the foundation slab had been placed and welded, the concrete was poured flush. The floor liner plates were then placed and welded. Concurrently, the wall liner plates were erected in 60-degree segments and 10-foot-high courses. Containment wall concrete was then placed against the wall liner plates. After cracking was discovered in the liner plate welds, the erection procedures were modified so that at least one 10-foot-high course was completely placed, welded and tested above the top of any concrete pour (see Subsection 5.8.7.4.1). This pattern was followed to the dome spring line.

5.8.7.4 Construction Problems

5.8.7.4.1 Cracking at Welds in Containment Liner Plate

1. Cracking in Wall Liner Plate Welds

   On September 27, 1967, Bechtel field inspectors reported the discovery of 12 transverse cracks in completed horizontal butt welds in the containment liner plate at elevations 590 feet and 600 feet. All the cracks, except one, were discovered after successful completion of spot radiography and 100% vacuum box inspections. The cracks occurred both where concrete had been poured behind the liner plate and where this had not yet been done. However, the cracks occurred only where small gaps had been left in the backing strips.

   Placing of concrete was halted immediately, and an investigative program was initiated to determine the extent and causes of cracking and to formulate repair procedures. A Bechtel welding and metallurgical engineer and a structural engineer were sent from San Francisco to conduct a thorough investigation.

   The following observations and conclusions were made by the inspection team:

   a. It was confirmed that all cracks had occurred in horizontal butt welds at gaps in backing strips. These gaps varied from 1/32 inch to 1/2 inch. See Figure 5.8-21.

   b. None of the locations chosen for routine spot radiographs taken during the liner plate erection coincided with a crack. This was not a coincidence. No radiography was performed at backing strip discontinuities because it was believed that it would be difficult to evaluate the radiographs due to the varying thickness of the material.

   c. At least six passes were made for the liner plate butt joints due to wide gaps between the plates.

   d. The liner plate shell was subjected to considerable loads, both before and after welding, to facilitate fit-up and to meet the specified radial tolerances.

During this investigation, seven additional areas where discontinuities occurred were inspected for cracks by visual examination, vacuum box, dye penetrant, magnetic particle and radiographic means. Radiography provided inconclusive evidence of cracking in six of these areas and magnetic particle examination revealed a crack in the seventh. All seven areas were removed for laboratory testing. Laboratory cross-section and macroetch examination revealed that five of six samples exhibited a lack of fusion, three of six exhibited incipient cracking, and the seventh had a through thickness crack. Chemical analysis of the sample containing the crack indicated that the chemistry of the materials did not contribute to cracking.

Additional magnetic particle examination made by Superior Industrial X-Ray Company on October 24, 1967 revealed five more cracks in horizontal seams.

2. Reasons for Weld Cracking

On the basis of the foregoing examination and testing, tentative conclusions were reached as to why the liner plate welds cracked. The results of the subsequent weld test program and additional nondestructive testing did not alter these conclusions.

It was concluded that the combination of stresses, discontinuities and lack of fusion caused the welds to crack. Stresses were induced by:

a. Weld shrinkage - these stresses were aggravated because the backing strips were rigidly held by the concrete.

b. Ambient temperature changes.

c. Wind loads.

d. Excessive post-weld jacking and pulling of the liner plate to achieve the construction tolerances.

The discontinuities in the backing strips acted as stress risers. However, the mere existence of a discontinuity was not in itself responsible for the cracks as evidenced by the many discontinuities at which no cracking occurred. Instead, cracking was initiated at only those discontinuities where a lack of fusion was present in addition to a high stress level.

3. Revisions to Liner Plate Erection Techniques

In line with the above tentative findings, the following revisions were made to the liner plate erection techniques to prevent further cracking:

a. Dimensional requirements for vertical alignment and horizontal curvature were met at fit-up prior to welding. Jacking or pulling was permitted only during fit-up operations when the plates were tack welded.

b. Concrete placement was sequenced with liner plate erection so that at least one 10-foot-high course was completely placed, welded and tested above the top of any concrete pour.

c. All backing strips were made continuous by full penetration welds at the required splices with the exception of a few specific locations in the dome liner plate where erection procedures made it impractical to perform this weld. For these few locations, a special magnetic particle inspection was performed.

4. Weld Test Program

As a continuation of the investigation into the cause of cracks in the liner plate, a program was initiated to test representative portions of the welds that had been judged satisfactory.

Two test specimens shown in Figure 5.8-22, which had been repaired, were removed by flame cutting. Testing was conducted by Anamet Laboratories, Inc, Berkeley, California. Tensile and bend tests were used to determine the strength and ductility of welds in accordance with the ASME B&PV Code, Section IX. Chemical analyses were performed on plates and welds. Weld cross sections were etched to observe the quality of the welds.

It was concluded, from a thorough examination of all test results (Reference 7), that the welds represented by the test specimens met all test requirements and were acceptable for the intended service.

5. Cracking in Floor Liner Plate Welds

On February 7, 1968, cracks were found in the floor plates at elevation 588 feet 6 inches and later in the reactor sump at elevation 583 feet 6 inches. These cracks were found by a second pressurizing of the leak chase system before putting on the 18-inch cover concrete. All welds had previously passed the vacuum box test, and the leak chase channels had been previously pressurized with satisfactory results. These cracks were also at discontinuities in the backing strip and the previous conclusions also applied here. However, in this case, the backing strips were H-beams embedded in concrete before the floor plates were welded. Thus, the weld shrinkage stresses were aggravated by the fact that the backing strips were rigidly held by the concrete. The floor plate was less able to accommodate thermal movements than in the walls where concrete was placed after welding.

6. Program To Locate and Repair All Cracks in the Liner Plate Welds

In order to be sure that all cracks had been located, a combination of ultrasonic and magnetic particle techniques were employed. Bechtel Welding Laboratory in San Francisco developed a program in which ultrasonic techniques were used to locate backing strip discontinuities, and magnetic particle examination was used to locate surface and subsurface cracks within the 1/4-inch liner plate.

Automation Industries, Inc of Danbury, Connecticut was engaged to conduct the search for discontinuities and cracking. The search was limited to those welds, primarily horizontal welds, completed prior to the change in erection procedures. The areas searched included all locations where cracks had previously been repaired. Figure 5.8-23 shows, for some of these areas, the liner plate details, the inspection and repair procedures used, and the locations of the cracks.

All cracks or indications of cracks were removed by grinding, and complete removal was verified by magnetic particle or liquid penetrant testing. Repair welding was conducted in accordance with procedures qualified by General American Field Erection Corporation (GATX) for welding plate against concrete. These procedures, M-1-H4 and M-1-V2, utilized E6010 electrodes for the first pass and E7018 electrodes for the completion of the weld. They were based upon techniques developed by Bechtel in San Francisco. The complete repairs were 100% magnetic particle inspected.

5.8.8 CONTAINMENT STRUCTURE TESTING

5.8.8.1 Integrated Leak Rate Testing

The containment has been designed to allow a leak rate of no more than 0.10 weight percent/day at a design pressure of 55 psig and a design temperature of 283°F. With this leakage rate, the public exposure following an accident would be below applicable 10 CFR 50.67 limits in the event of a Maximum Hypothetical Accident (see Section 14.22). Containment leakage rate testing is performed to quantify the leakage rate from containment under a set of defined post accident conditions. There are two types of containment leakage rate testing, Integrated Leakage Rate Testing and Local Leakage Rate Testing. The performance of periodic Integrated Leakage Rate Tests (ILRTs) during the plant life provides an update of the magnitude of potential total leakage from containment in the event of an accident that would pressurize the interior of the containment. Integrated leak rate testing is referred to as a Type A Testing in 10CFR50, Appendix J.

5.8.8.1.1 Historical Summary

The Preoperational Type A Tests - The preoperational Type A tests were conducted in May 1970 (Reference 8) in accordance with Bechtel Start-Up Standard 60 (Reference 8). At that time, the proposed Technical Specifications had not been approved by the NRC and Appendix J had not yet been published. Two tests were completed, a peak pressure test of 55 psig and a reduced pressure test of 28 psig. The acceptance criteria for maximum allowable leakage rate at 55 psig (L_p) was 0.072 weight percent/day after correction to test temperature. The maximum allowable leakage rate at 28 psig (L_t) was 0.0514 weight percent/day. Both test results met the acceptance criteria.

The First Postoperational Type A Test - The first postoperational Type A test was begun on April 30, 1974. This was a reduced pressure test (28 psig). This test was conducted within the general guidelines of the Technical Specifications, 10CFR50 Appendix J, and ANSI N45.4 (Reference 9). The acceptance criteria was 75% of L_t or L_{to} = 0.0386 weight percent/day. The results were found to be within acceptance criteria (Reference 10).

The Second Postoperational Type A Test - A second postoperational Type A test was completed on March 28, 1978 (Reference 11). This was a reduced pressure test, (28 psig). This test was conducted within the general guidelines of the Technical Specifications, 10 CFR 50 Appendix J, and ANSI N45.4 (Reference 9).

On February 11, 1975, the Technical Specifications were amended and the temperature correction factor was eliminated. The basis for this change was that such a correction is not required by 10 CFR 50 Appendix J, which was issued several years after the original Technical Specifications. This corrections factor was not applied to this test.

During containment pressurization, a leak was found on the 48-inch containment purge air exhaust penetration. The leak was measured and recorded and a grease fitting was replaced to correct the problem. The penetration was then leak tested again. The containment leakage rate after repair of the penetration was well within the acceptance criteria. In fact the calculations showed a net inflow into the containment.

The combined calculated containment leak plus the adjusted measured penetration leak were above the acceptance criteria.

The negative leakage, (net inflow) was attributed to instrument error which for this test was found to be approximately 40% of the acceptance criteria value. Since this magnitude of error can significantly impact future test results, Consumers Power Company performed a review of test monitoring equipment and procedures. As a result of this review, new instrumentation requirements were established for the ILRT, using ANS 56.8-1981, as a guide.

The Third Postoperational Type A Test - The third postoperational Type A test was concluded on November 18, 1981 (Reference 12). This was a reduced pressure test, (28 psig). This test was conducted within the general guidelines of the Technical Specification, 10 CFR 50 Appendix J, and ANSI N45.4 (Reference 9).

The containment leakage rate was within the acceptance criteria, but NRC Violation 255\86-005-04 later resulted in a requirement that repairs and adjustments be added to the leak rate. This resulted in the As Found leakage rate exceeding the acceptance criteria.

The Fourth Postoperational Type A Test - The fourth postoperational Type A test was conducted in January 1986. This was a reduced pressure test, (28 psig). This test was conducted within the general guidelines of the Technical Specifications, 10 CFR 50 Appendix J, and ANSI N45.4 (Reference 9).

The As Found leakage rate exceeded the acceptance criteria when the repairs and adjustments were added to the calculated leak rate.

The Fifth Postoperational Type A Test - The fifth postoperational Type A test was conducted in November 1988. This was a reduced pressure test, (28 psig). This test was conducted within the general guidelines of the Technical Specifications, 10 CFR 50 Appendix J, and ANSI N45.4 (Reference 9).

The containment leakage rate was within the acceptance criteria.

The Sixth Postoperational Type A Test - The sixth postoperational Type A test was conducted in February 1991, as a follow-up to the temporary containment opening for steam generator replacement. This was a full pressure test, (55 psig).

The containment leakage rate was within the acceptance criteria.

The Seventh Postoperational Type A Test - The seventh postoperational Type A test was completed in May 2001. The test was conducted within the general guidelines of the Technical Specifications, 10 CFR 50 Appendix J Option B, ANSI/ANS 56.9 - 1994 (Reference 30) and NEI 94-01 Rev. 0 (Reference 31). This was a full pressure (Pa) test.

The containment leakage rate was within the acceptance criteria.

5.8.8.1.2 Regulatory Basis for Current Program

A Technical Specification Change was approved by the NRC in 1996 to allow performance based Type A testing in accordance with 10CFR50 Appendix J, Option B. All subsequent Type A tests were to be performed in accordance with Appendix J, Option B and its accompanying Regulatory Guide 1.163, "Performance-Based Containment Leak-Test Program" (Reference 32).

The Regulatory Guide references NEI Industry Guideline Document NEI 94-01 Revision 0, (Reference 31). This document, as modified by its errata sheet, (Reference 39), provides the basis for the performance based program. Test intervals and performance evaluation criteria are all contained in this document. The technical direction for performing and evaluating Type A test results are contained in ANSI-ANS 56.8-1994, "Containment System Leakage Testing Requirements" (Reference 30).

Alternately, the BN-TOP-1 method of Type A testing, (Reference 38) may be used in place of the mass plot method, (Reference 30). The NRC (AEC) staff approved the use of BN-TOP-1 in 1972, (Reference 40) and it remains an acceptable method for use under Option B (Reference 41 and 42).

By letter dated April 23, 2012, "Palisades Nuclear Plant – Issuance of Amendment to Extend the Containment Type A Leak Rate Test Frequency to 15 Years (TAC No. ME5997)," the NRC approved Amendment 247 to the Palisades Technical Specifications (TS). The amendment revised TS 5.5.14, "Containment Leak Rate Testing Program," by replacing the reference to Regulatory Guide 1.163 with a reference to NEI 94-01, Revision 2-A, "Industry Guideline for Implementing Performance-Based Option of 10 CFR [Title 10 of the Code of Federal Regulations] Part 50, Appendix J" (Reference 50), as the implementation document for the 10 CFR 50 Appendix J, Option B, performance-based containment leak rate testing program. ANSI/ANS 56.8-2002 (Reference 51) was endorsed by NEI 94-01, Revision 2-A, for performing and evaluating the Type A test results.

5.8.8.1.3 Type A Test Performance Under Option B

Section 9.2.3 of NEI 94-01 Rev. 0, (Reference 31) allows for extended ILRT intervals of up to 10 years if an acceptable Type A test performance history has been demonstrated. This was accomplished by meeting the following criteria:

- The plant passed its previous two consecutive ILRTs when evaluated on a performance basis.
- The elapsed time between the first and last test must be at least 24 months
- At least one of the two ILRTs must have been performed at full pressure

Evaluation of Palisades fifth and sixth periodic Type A tests showed that all of the above criteria were met. The initial Type A test interval under Option B was established as ten years.

The seventh postoperational Type A test was completed in May of 2001 to meet the ten year surveillance requirement. The ILRT was successfully completed meeting all of the acceptance criteria including the performance leakage rate criteria. Based on the test meeting the extended ILRT interval requirements of NEI 94-01, the ten year interval for Type A testing was not revised.

By letter dated April 23, 2012, “Palisades Nuclear Plant – Issuance of Amendment to Extend the Containment Type A Leak Rate Test Frequency to 15 Years (TAC No. ME5997),” the NRC approved Amendment 247 to the Palisades Technical Specifications. This amendment allows Palisades to extend its performance-based Type A test interval up to 15 years.

The next Type A test under Option B is due in accordance with the Technical Specifications.

5.8.8.2 <u>Local Leakage Rate Testing</u>

The containment has been designed to allow a leak rate of no more than 0.10 weight percent/day at a design pressure of 55 psig and a design temperature of 283°F. With this leakage rate, the public exposure following an accident would be below applicable 10 CFR 50.67 limits in the event of a Maximum Hypothetical Accident (see Section 14.22). Containment Leakage Rate Testing is performed to quantify the leakage rate from containment under a set of defined post accident conditions. There are two types of containment leakage rate testing, Integrated Leakage Rate Testing and Local Leakage Rate Testing. The performance of periodic Local Leakage Rate Tests, (LLRTs) during the plant life provides an update of the magnitude of potential Appendix J leakage from Type B and C tested pathways in the event

of an accident that would pressurize the interior of the containment. It also established the functionality of individual components for the IST program and is the basis for determining the components' repair intervals. Local Leak Rate Testing is referred to as Type B and C Testing in 10CFR50, Appendix J.

5.8.8.2.1 Historical Summary

Preoperational Local Leak Rate Tests - Preoperational local leak rate tests were performed in 1970. The results indicated that the total leakage from those tests were well below the limit of acceptance (Reference 13). These tests were conducted in accordance with Bechtel Start-Up Standard Number 60 (Reference 13) since the proposed Technical Specifications had not yet been approved by the NRC. These tests had an acceptance criteria for the total leakage from all penetrations and isolation valves to not exceed 0.50 L_p, where L_p was the maximum allowable leakage rate after a temperature correction. At standard test temperature, L_p = 0.072 weight percent/day.

Initial Postoperations Test - Postoperational tests conducted under the guidelines of the original Technical Specification, the acceptance criteria was 0.45 L_p.

June 13, 1973 Technical Specification Amendment - The Technical Specifications were amended with a new acceptance criteria for total leakage of 0.60 L_p.

February 11, 1975 Technical Specification Amendment - This revision incorporated the requirements of 10 CFR 50 Appendix J which eliminated the use of a temperature correction factor. The acceptance limit then became 0.60 L_a with L_a being the total allowable leakage of 0.10 weight percent/day.

Surveillance Procedure RO-32 - The draft copy of Surveillance Procedure RO-32 (Rev 0) was issued on July 10, 1980. The basis section of this procedure document converts the acceptance criteria of weight loss into volume loss for direct comparison to individual local leakage test results (Reference 14).

June 1, 1989 Technical Specification Amendment - This amendment incorporated a reduced pressure ($\geq$ 10 psig) between the seals test of the containment air locks. The acceptance criteria states "the leakage for an air lock door seal test shall not exceed 0.023 L_a."

September 30, 1997 Technical Specification Amendment - This amendment (and exemption) provided relief from the requirement to perform additional air lock leakage rate testing after opening the Emergency Escape Air Lock doors for post test restoration or seal adjustment following air lock leakage rate testing.

March 30, 2001 Technical Specification Amendment - This amendment allows performance based Type B and C testing in accordance with 10 CFR 50 Appendix J Option B.

April 23, 2012 Technical Specification Amendment - This amendment allows performance based Type B and C testing in accordance with 10 CFR 50 Appendix J Option B and NEI 94-01, Revision 2-A.

5.8.8.2.2 Regulatory Basis for Current Program

A Technical Specification Amendment was approved by the NRC on March 30, 2001, which allows performance based Type B and C testing in accordance with 10 CFR 50 Appendix J, Option B. All subsequent Type B and C tests were to be performed in accordance with Appendix J, Option B and Regulatory Guide 1.163, "Performance- Based Containment Leak - Test Program," with three exceptions as listed in Technical Specifications ADMIN 5.5.14a.

The Regulatory Guide, with some restrictions, endorses the use of NEI 94-01, Revision 0, (Reference 31) for compliance with the requirements of Option B. The basis for the performance based Type B and C test program is NEI 94-01 which provides test intervals and performance evaluation criteria. The technical direction for performing and evaluating Type B and C test results are contained in ANSI/ANS 56.8-1994, (Reference 30).

By letter dated April 23, 2012, "Palisades Nuclear Plant – Issuance of Amendment to Extend the Containment Type A Leak Rate Test Frequency to 15 Years (TAC No. ME5997)," the NRC approved Amendment 247 to the Palisades Technical Specifications (TS). The amendment revised TS 5.5.14, "Containment Leak Rate Testing Program," by replacing the reference to Regulatory Guide 1.163 with a reference to NEI 94-01, Revision 2-A, "Industry Guideline for Implementing Performance-Based Option of 10 CFR [Title 10 of the Code of Federal Regulations] Part 50, Appendix J" (Reference 50), as the implementation document for the 10 CFR 50 Appendix J, Option B, performance-based containment leak rate testing program. ANSI/ANS 56.8-2002 (Reference 51) was endorsed by NEI 94-01, Revision 2-A, for performing and evaluating the Type A test results.

5.8.8.2.3 Scope of the Type B and C Testing Program

Periodic Type B or C tests shall be performed on the types of components listed below:

1. Containment penetrations that employ resilient seal gaskets, sealant compounds or bellows

2. Air locks and equipment door seals

3. Fuel transfer tube

4. Isolation valves on lines penetrating the containment that are potential Post-LOCA air leakage paths

The specific Appendix J Type B and C test requirements for each containment penetration are given in Table 5.8-4.

5.8.8.2.4 Performance Based Type B and C Program Overview

The implementation of 10 CFR 50, Appendix J, Option B for Type B and C tests allows the adoption of performance based test intervals. To adopt performance based test intervals administrative leakage limits are established for each Type B and C tested component. Performance baselines are established for each component based upon the component's measured leakage rates compared against its administrative limits for the last two or three tests. Type B and C components with poor histories or those which have not yet established histories must be tested on intervals not to exceed 30 months.

Type B and C components (except airlocks) may be tested every 60 months following two consecutive successful tests not less than 24 months (or normal refueling cycle) apart. Type B tested components (except airlocks) may be put on 120 month test intervals following three consecutive successful tests spaced from first to last not less than 24 months (or normal refueling cycle) apart. Any additional criteria specified by Regulatory Guide 1.163 and NEI 94-01 must also be met in order to qualify for extended test intervals.

The Type B and C testing intervals listed here may be extended by up to 25 percent although in no case more than 15 months.

The Containment Personnel Air Lock and the Emergency Escape Air Lock, are subject to fixed testing intervals as specified by NEI 94-01. Local leak rate tests (between the seals) of the Personnel Air Lock door seals shall be performed at a pressure of not less than 10 psig. The Personnel Air Lock door seals are tested at a lower pressure than P_a due to the design of the air lock doors. This pressure is sufficient to determine seal operability. Local leak rate testing (between the seals) of the Emergency Air Lock door seals is not routinely performed because of the design. Technical Specification Amendment number 177 and an exemption were approved by the NRC on September 30, 1997, for the Emergency Escape Air Lock. The amendment and exemption permit the performing of a door seal contact verification in lieu of a pressure test following the opening of the doors for post test restoration or seal adjustment.

Local leak rate tests are to be performed at a differential pressure of not less than P_a. If the applied test pressure assists the closure of a barrier, then test pressure shall not exceed 1.1 P_a. Acceptable methods of testing for leaks are halogen gas detection, soap bubble, pressure decay, makeup flow rate or equivalent.

5.8.8.3 Prestressing System Surveillance

5.8.8.3.1 Basis for Program

The prestressing system surveillance program is a systematic means of assessing the continuing quality and the structural performance of the containment post-tensioning system (References 15, 16, 17, 18, 35, 36 and 37). This program consists of a periodic surveillance of randomly selected tendons (randomness became effective with the 5-year surveillance). Each surveillance includes measurement of tendon lift-off forces; tendon wire continuity testing, visual inspection and tensile strength testing; inspection of tendon anchorage assemblies; and inspection, sampling and testing of sheathing filler. Information derived from this inspection and testing provides a basis for confidence in the physical condition and functional capability of the prestressing system. If adverse conditions that might lead to tendon deterioration or malfunction are detected, timely corrective measures can be taken.

The prestressing system surveillance program was established to satisfy the intent of Subsection 4.5.4 of the Plant Technical Specifications. This program has been conducted in general conformity with the requirements of this subsection in effect at the time of each surveillance. A complete revision of Subsection 4.5.4, approved in 1975, made it consistent with the requirements of Regulatory Guide 1.35, Revision 0 (Reference 19). Subsequent to the 10-year surveillance, as a result of SEP Topic III-7.A discussions and the proposed Revision 3 to Regulatory Guide 1.35, CP Co agreed to modify the prestressing system surveillance program (References 25 and 26). The Plant Technical Specifications were updated to reflect the modifications of Revision 3 to Regulatory Guide 1.35, with the exception of the "common tendon" criteria.

With implementation of Technical Specification Amendment 189, the requirements for Containment Inservice Inspection (CISI) including prestressing system surveillance were moved to Technical Specification ADMIN 5.5.5. Technical Specification ADMIN requires CISI be conducted in accordance with the American Society of Mechanical Engineers, Boiler, and Pressure Vessel Code, Section XI, Subsections IWE and IWL.

5.8.8.3.2 Surveillance Period

Tendon inspection shall be accomplished in accordance with the following schedule:

1. One year after the structural integrity test

2. Three years after the structural integrity test

3. Five years after the structural integrity test

4. At five-year intervals thereafter for the life of the Plant

5.8.8.3.3 Surveillance Guidelines

The Technical Specifications "surveillance guidelines," shown below, were in effect for the 5- and 10-year surveillance. Although these guidelines are more extensive than those in effect at the time of the 1- and 3-year surveillances, the actual inspection and testing has been nearly the same for all surveillances.

During the 5 and 10 year surveillances, the following field testing and inspection was performed:

1. Lift-off readings shall be taken for each of the surveillance tendons. The lift-off test included a maximum test lift-off force greater than the maximum inservice prestressing force and an unloading cycle going to essentially complete detensioning of the tendon to identify broken or damaged wires.

   If the force measured for a tendon was less than the lower bound curve of the force-time graph for the corresponding tendon group (dome, hoop, vertical), two adjacent tendons were tested. If either of the adjacent or more than one of the original sample population fell below the lower bound of the force-time graph, an investigation was conducted before the next scheduled surveillance. The investigation was made to determine whether the rate of force reduction is indeed occurring for other tendons. If the rate of reduction was confirmed, the investigation was extended so as to identify the cause of the rate of force reduction. The extension of the investigation determined the needed changes in the surveillance inspection schedule and the criteria and initial planning for corrective action.

   If the force measured for a tendon at any time exceeds the upper bound curve of the band on the force-time graph, an investigation was made to determine the cause.

2. While the tendon was in the detensioned state, each wire in the tendon was checked for continuity.

3. Three wires, one from each of a vertical, a hoop and a dome tendon were removed and identified for inspection. At each successive surveillance, the wires will be selected from different tendons. Each of the inspection wires removed was visually inspected for evidence of corrosion or other deleterious effects and samples taken for laboratory testing.

4. The sheathing filler was inspected visually for color and coverage, and samples were obtained for laboratory testing.

5. Tendon anchorage hardware such as bearing plates, stressing washers, shims and buttonheads were visually inspected for evidence of corrosion or other deleterious effects.

6. After lift-off test and inspection, the tendons were retensioned to the unit stress measured at the initial lift-off test and then checked by a final lift-off reading.

Subsequent to the field testing of surveillance tendons, the following laboratory testing was done:

1. Three tensile test specimens were cut from each of the three inspection wires removed (one specimen from near each end and one from the middle) plus one additional specimen shown by the field visual inspection to have the greatest amount of corrosion of all of the wires removed. Each of the wire samples shall be tested for ultimate strength.

2. The sheathing filler samples were laboratory tested to detect any significant change in corrosion resistant properties of the filler.

If, as a result of a prestressing system inspection, corrective retensioning of 5% (8 tendons) or more of the total number of dome tendons was necessary to restore their lift-off forces to a level that falls within the acceptance criteria, a dome delamination inspection was performed. This inspection was performed within 90 days following such corrective retensioning and the results of this inspection were reported to the NRC.

The modifications to be in effect for the 15-year surveillance incorporate the NRC concerns that:

1. The acceptance criteria used to evaluate containment tendon surveillance test results should vary with time to reflect the fact that overall prestress decreases in a reasonably predictable manner with time. This is accomplished by imposing limits on the existing predicted prestress loss curves.

2. Same means should be used to correlate individual tendon surveillance test results with the expected prestress value for that tendon. This is accomplished by normalizing individual tendon test results to correlate each tendon with the average tendon.

3. A more detailed concrete anchorage inspection criteria was necessary.

The following requirements were made effective for the 15-year surveillance:

1. The containment shall be considered to have satisfied the prestressing system examination if the following tendon force (average of forces measured at both ends, if applicable) requirements are met:

   a. The average of all measured tendon forces for each type of tendon is equal to or greater than the minimum required prestress level for that type.

   b. The force measured in any individual tendon is equal to or greater than 95% of its predicted prestress force at the time of the test.

   c. The tendon force measurement of any individual tendon divided by the total area of the effective prestressing elements, as defined in the Construction Specifications, which comprise the tendon shall not exceed 70% of the minimum specified ultimate strength of the prestressing elements.

2. Cracks observed in the immediate vicinity of the tendon anchorages being inspected, which exceed a maximum width of 0.010 inch, will be mapped and evaluated.

3. Each required evaluation will consider, as applicable, crack location, crack size, crack depth if known, location and arrangement of rebar in the vicinity, comparison with previous maps of the same crack if available, concrete temperature as well as experience with the behavior of other typical concrete structures.

4. If an evaluation concludes that an observed crack may be severe enough to potentially degrade containment integrity, that crack will be inspected again with the containment under pressure during the next ILRT. Final resolution of the condition then would be based on the crack's behavior during pressurization as well as the other factors discussed above.

See References 25 and 26 for additional details.

5.8.8.3.4 Acceptance Criteria

The Technical Specifications acceptance criteria in effect during the 1-year and 3-year surveillances were as follows:

1. No additional broken wires since the last inspection.

2. The force time trend line for each tendon, as extrapolated, shall not intersect either the upper or lower bound of the predicted design band.

3. No unexpected change in corrosion conditions or sheathing filler properties.

The Technical Specifications acceptance criteria, which became effective prior to the 5-year surveillance, are as follows:

1. The measure of the lift-off force per tendon shall not be more than 815 KIPs per tendon nor less than 584 KIPs per tendon for dome tendons nor less than 615 KIPs per tendon for hoop and vertical tendons.

   If one sample tendon fails to meet these criteria, an adjacent tendon on each side of that tendon shall also be tested. If both of these tendons meet the criteria, then the inspection program shall proceed considering the single deficiency as unique and acceptable. However, if either adjacent tendon fails to meet the criteria or if more than one tendon out of the original sample population fails to meet the criteria, the testing shall be aborted and further analysis made to determine the reason for the unacceptable readings.

2. Inspection wires shall indicate no significant loss of section by corrosion or pitting.

3. Tensile test specimens cut from inspection wires shall be tested for an ultimate strength of not less than 11.78 KIPs for each of the test samples.

4. Tendon anchorage hardware shall be free of significant corrosion, pitting, cracks or other deleterious effects.

The failure of any element of the prestressing system to meet the acceptance criteria listed above, or any indication of abnormal degradation of this system, was included in the surveillance report submitted to the Nuclear Regulatory Commission (NRC).

Current testing and acceptance criteria are in accordance with existing Technical Specification requirements.

5.8.8.3.5 Historical Summary

The original Technical Specifications required that nine surveillance tendons be selected, three from each of the dome, vertical and hoop families as follows (see Figure 5.8-24):

1. Three dome tendons located approximately 120 degrees apart

2. Three vertical tendons located approximately 120 degrees apart

3. One hoop tendon from each of three 120-degree sectors of the containment

These nine tendons were not required as part of the containment structure design. Instead, they were installed for surveillance purposes only. They were to be reinspected at each surveillance during the life of the Plant.

Prior to the 5-year surveillance, the Technical Specifications were revised. This revision required that the surveillance tendons be randomly selected for all future surveillances. Also, the pool of available tendons was limited to those whose routing was not modified to clear penetrations.

For the 5-year surveillance, the Technical Specifications required that the surveillance tendons be randomly selected to obtain:

1. Two dome tendons from each of the three dome tendon groups

2. Five vertical tendons located approximately 72 degrees apart

3. Three hoop tendons from each of three sectors of the containment plus one additional hoop tendon

For the 10 year surveillance, the Technical Specifications required that surveillance tendons be randomly selected to obtain:

1. One dome tendon from each of the three dome tendon groups

2. Three vertical tendons located approximately 120 degrees apart

3. One hoop tendon from each of three 120-degree sectors of the containment

Prior to the 15 year surveillance, Technical Specifications were revised for subsequent testing. The current requirements and acceptance criteria are specified in the Technical Specifications.

For the 1-year and 3-year surveillances, lift-off forces for the surveillance tendons were determined using the "change in sound" approach. This procedure involved tapping the shims with a hammer while increasing the tendon tension until the ringing sound of any given shim was replaced by a dull thud, thus indicating that the struck shim was no longer in compression.

Comparison of the 1- and 3-year lift-off forces for identical tendons indicated that 2 vertical tendons had lost 12% to 16% of their prestress in 2 years. An investigation was undertaken to identify the cause of this apparent loss of prestress. It was concluded that the "change in sound" approach to defining lift-off yielded values that were nonrepresentative of the actual tendon tension. Therefore, during the 3-year surveillance, a new approach known as the "all shims loose" approach was evaluated using ten additional vertical tendons. This procedure involved tapping the shims with a hammer while increasing tendon tension until all shims (ie, both halves of a single shim) were displaced by the hammer. Lift-off values for these tendons fell above the expected value based upon losses with the exception of one tendon which had a borderline value. The "all shims loose" approach was adopted for all subsequent surveillances.

Based upon the results of the inspections, tests and investigations performed during the surveillances conducted to date, there is no indication of any abnormal degradation of the containment post-tensioning system.

Highlights of each individual surveillance are presented below:

1. One-Year Surveillance

   The 1-year surveillance of the post-tensioning system was performed in 1971 (Reference 15). Lift-off forces were recorded for the nine preselected tendons and, except for one, were within the range of values expected based upon estimated losses due to concrete creep and shrinkage, steel relaxation and initial structural deformation. Because of the low lift-off readings for one vertical tendon (V-204), two additional vertical tendons were included in this surveillance. Lift-off forces were also obtained for 15 additional vertical tendons. This additional data showed that the one low reading was not typical.

   Three wires were removed for inspection. Besides the onsite visual inspection a laboratory examination was performed to provide a correlation between the surface appearance and the depth of the surface imperfections and to determine the nature of these imperfections. The tendon wires were found to contain minor nicks and scratches from handling, die marks, heat treating discolorations and some localized corrosion sparsely distributed along the length of the wires.

2. Three-Year Surveillance

The 3-year surveillance was performed in 1975 (Reference 16). The 11 surveillance tendons employed in the 1-year surveillance were reevaluated. Lift-off forces for these tendons were determined using the "change of sound" approach. Three of the five vertical tendons had lift-off forces that fell below the expected value. Ten additional tendons, all different than the 15 additional tendons of the 1-year surveillance, were selected for lift-off testing using the "all shims loose" approach. The average lift-off value for this group of tendons was within the expected range based upon estimated losses.

Approximately two cups of free water were found in the end cap of a dome tendon (D2-53). It was hypothesized that this water resulted from infiltration at the sheath-to-trumplate connection. Laboratory examination of the sheathing filler samples, including the four discolored grease samples and the D2-53 free water, indicated that the deleterious product content of all samples was within the established acceptance limits.

3. Five-Year Surveillance

The 5-year surveillance was performed in 1975 (Reference 17). Twenty-one randomly selected surveillance tendons (six dome, five vertical and ten hoop) were evaluated. This group did not include any of the 11 tendons examined during the 1- and 3-year surveillances. Every surveillance tendon had a lift-off force within the prescribed acceptance limits.

A total of 5 discontinuous wires was found in the 21 surveillance tendons. A visual examination of these wires was sufficient to determine that four of the discontinuities resulted from improper installation. However, it was necessary to subject the fifth wire to laboratory examination in order to determine why it broke. Based upon the results of this examination, it was concluded that this wire failed because of a material defect.

4. Ten-Year Surveillance

The 10-year surveillance was performed in 1981 (Reference 18). Nine randomly selected surveillance tendons were evaluated. Every surveillance tendon had a lift-off force within the prescribed acceptance criteria.

A total of three discontinuous wires was found in the nine surveillance tendons. A visual examination of these wires indicated that the discontinuities occurred during installation.

Tests on tendon grease and tendon wire samples were conducted in the laboratory. Even though an excessive water content was found in some grease samples, no degradation of wires was observed. In addition, results of the tensile testing of wire samples showed that the tensile strength of all the samples met the acceptance criteria. For more details, refer to Reference 18.

5. Fifteen-Year Surveillance

The 15-year surveillance was performed in 1987. Thirteen tendons were evaluated. The evaluation found that every surveillance tendon had a lift-off force within the prescribed acceptance criteria.

Two tendons (74BF and V-62) were found to have missing wires not previously recorded. These anomalies were not determined to be detrimental to the post-tensioning system.

All wire samples tested were found to be acceptable in diameter, yield strength, tensile strength, and elongation.

The sheathing filler samples tested had acceptable levels of water content and water soluble ions (chlorides, nitrates, and sulfides).

Two surveillance tendons (V-124 and D1-38) were found to have water contained in their grease cans. Tendon V-124 (field end) had drops of water observed and D1-38 (shop end) had eight ounces of water measured. Non-surveillance tendon V-202 also had drops observed. Water was not found in any other surveillance tendons or any of the other non-surveillance tendons during the removal of the grease can or inside the removed can.

Based on the results of the 15-year surveillance, the conclusion was reached that no abnormal degradation was evident in the post-tensioning system. (Reference 35)

6. Twentieth-Year Surveillance

The twenty-year surveillance was performed in 1992. Fourteen tendons were evaluated. Of these fourteen tendons, five were either detensioned, removed or had a low liftoff force during the steam generator replacement outage of 1990. Every surveillance tendon had a liftoff force within the prescribed acceptance criteria during the surveillance.

Three tendons (D1-42, V-20 and H52AE) were found to have missing buttonheads not previously recorded. These anomalies were not determined to be detrimental to the post-tensioning system.

All wire samples tested were found to be acceptable in diameter, yield strength, tensile strength, and elongation.

The sheathing filler samples tested had acceptable levels of water soluble ions (chlorides, nitrates, and sulfides) and water content, with the exception of dome tendon D2-23, whose shop end had a water content of 15.2 percent (refer to Deviation Report D-Pal-92-117 for additional information).

Surveillance tendons V-20,V-72, and D2-23 were found to have small quantities of water contained in their grease cans. Tendons V-20 and V-72 had drops of water while D2-23 had less than a pint of water. Water was not found in any other ISI surveillance tendons or any of the other non-surveillance tendons during the removal of the grease can or inside of the removed can.

Based on the results of the twenty-year surveillance, the conclusion was reached that no abnormal degradation was evident in the post-tensioning system (Reference 36). The NRC issued a Safety Evaluation on the twentieth year tendon surveillance, dated October 28, 1994. The NRC concurred that the Palisades containment building post-tensioning system had not experienced abnormal degradation. However, the NRC recommended that all available data should be used and CPCo should perform an adequate regression analysis with the complete data before and after the scheduled surveillance in 1997.

5.8.8.4 Structural Integrity Test

5.8.8.4.1 Basis for Test

The purpose of the Structural Integrity Test was to monitor the response of the containment structure to loads imposed by the prestressing system and by a pressurization test.

The prestressing operations occurred during the period May 1969 through September 1969, and the pressurization test was performed during the period of March 23, 1970 through March 31, 1970. During the containment pressurization, the internal pressure was raised in increments up to 63.3 psig, which is 115% of the design pressure.

The subsequent comparisons of measured response to that predicted by analysis was used to assess design methods and parameters and to confirm the structural integrity of the containment.

5.8.8.4.2 Test Guidelines

Test measurements included concrete, reinforcing steel, liner strains, concrete temperatures, prestressing tendon forces, overall containment displacements and concrete cracking patterns. Approximately 450 sensors were installed to obtain the test data.

Test measurements were made at locations where analytical predictions of measured parameters were expected to be accurate and at locations where supplemental information on structural behavior was deemed useful as follows:

1. Strains, displacements and concrete temperatures were measured along a buttress and a typical section and around the equipment opening. Strain measurements were both circumferential and meridional and near both the inner and outer faces of concrete as well as in the liner.

2. Tendon forces were measured on two tendons from each group - hoop, vertical and dome.

3. Radial and/or vertical displacements of the containment were measured at regular sections as well as around the equipment opening.

4. Concrete crack patterns were plotted both for areas away from discontinuities and for areas where concrete surface stress concentrations were expected.

Test data were recorded starting in the early phases of construction and continuing through the end of the pressure test. The test data were reduced and evaluated at periodic intervals to determine both sensor and structural behavior. Time base strain and temperature data cover the period beginning at the start of post-tensioning and continuing through the pressure test. Tendon force and containment displacement data cover only the pressure test period.

5.8.8.4.3 Objectives

1. Strain Measurement

   Concrete and reinforcing bar strains were measured to (1) verify the validity of the assumptions used in the structural analyses, (2) determine if the concrete remained in compression under combined prestress and maximum test pressure, (3) assess the behavior of the structure in regions of discontinuity, and (4) monitor structural behavior during pressurization. The test measurements of reinforcing steel strain were considered to represent the effective strains in the reinforced composite.

Concrete temperatures were measured in the vicinity of several concrete strain sensors to allow evaluation of the thermal strain component resulting from temperature gradients in the concrete.

Liner strains were measured to determine how the liner interacted structurally with the concrete shell under the prestressing forces and subsequent internal pressure. The liner was fastened to the concrete at the anchorages but was expected to exhibit independent structural behavior elsewhere.

2. Displacement Measurement

Containment surface displacements were measured during pressurization to determine (1) the degree of correlation between strain and gross dimensional changes (integrated strain), and (2) the patterns of diameter change due to the presence of buttresses, openings and nonaxisymmetric features.

3. Tendon Loads

Tendon loads were measured to evaluate the interaction between the tendons and the concrete shell, and to provide assurance that the tendon force change during pressurization remained small.

4. Concrete Cracking

Concrete crack patterns were observed prior to and during prestressing, and measured in selected areas during the pressure test. The size, growth and pattern of cracks were indicative of the state of strain at the concrete surface and, in areas of stress concentrations, the crack data were a supplement to the strain measurements.

5.8.8.4.4 Test Data and Results

1. Strain Measurement

Prior to the prestressing operation, strain values were recorded and were used to define a reference strain. These values were subtracted from all subsequent readings to obtain the changes in strain with respect to the start of prestressing.

A review of strain histories recorded during the prestressing period indicates that all strains were found to be acceptable. The general response of strain gauges located at the buttress section was similar to the response at the typical wall section.

A review of strain versus time plots during the pressure test shows good correlation between pressure and strain gauge readings. Strain readings for gauges in the outside face of the containment show daily variations which parallel outside temperature readings for those days. In all cases, the tensile strain recorded during the pressure test was less than the corresponding compressive strain recorded during pre-stressing. This indicates that a net residual compressive strain remained in the containment structure during the pressure test. See Figure 5.8-25 for examples of strain profiles at typical wall locations.

2. Displacement Measurement

Measurements of containment displacements were made during the pressure test at a buttress section (azimuth 85 degrees) and a typical wall and roof section (azimuth 176 degrees). The buttress and wall measurements were horizontal (radial) and roof measurements were vertical.

As expected, displacements were found to be proportional to pressure and are greatest near the mid-height of the structure.

There was relatively close agreement between the measured radial displacements (average of wall and buttress values) and the displacements computed from hoop strains measured at the typical wall section (see Figure 5.8-26).

3. Tendon Loads

Representative dome, hoop and vertical prestressing tendons were instrumented with load cells to measure induced loading resulting from containment expansion during the pressure test. The dome and hoop tendons had a cell at both anchorages and the vertical tendons had only one cell at the ring girder anchorage. Of the ten anchorages instrumented, seven had load cells and three (two vertical and one hoop) had calibrated stressing jacks. The jacks provided a good comparative check of the load cell data.

Measured data indicated a maximum 2% tendon load change during the pressure test. The hoop tendon load cell plots (see Figure 5.8-27, Sheet 1, for a typical plot) showed some indication of the expected induced load change, but the dome and vertical tendon plots (see Figure 5.8-27, Sheet 2, for a typical plot) did not indicate a definite trend.

4. Concrete Cracking

   Concrete crack patterns were recorded prior to and during the pressure test. Concrete crack patterns were plotted both for areas away from discontinuities and for areas where concrete surface stress concentrations were expected. Prior to the start of the test, crack widths of up to 0.025 inch were measured.

   The cracks which opened up under pressure were found to be randomly oriented. The change in crack width due to maximum pressure ranged from 0.001 inch to 0.030 inch with the vast majority being below a 0.005 inch change.

5.8.8.4.5 Summary

It was concluded that the test measurements confirmed the predicted structural behavior during pressurization and that the design criteria methods were sufficient (Reference 20).

For the prestressing operations phase of the test, the following major conclusions were drawn:

1. The comparison of predicted and measured strain showed relative agreement. In general, measured strain exceeded predicted values and this was largely attributed to concrete creep.

2. As predicted, the cylinder hoop strains and the dome strains were among the largest. The vertical cylinder strains and those at discontinuities such as the ring girder were among the smallest.

3. The strains measured at buttresses differed from those away from buttresses, but not significantly so when compared with the strain variations that are attributable to creep.

4. As was expected, some of the strains at the equipment opening produced the largest variations from predicted strains and in some instances were of the opposite sign. However, these strains were the closest to zero strain and, therefore, were not considered to be a significant variation.

During the pressure test, strain measurements were generally in agreement with predictions which showed that compression induced by the prestressing reduces with a pressure increase but that the containment concrete remains in compression at maximum test pressure. Actual strain deformations were found to be approximately 5/6 (83%) of the predicted values. This is attributed to a higher concrete modulus of elasticity than was assumed.

The pressurization of the containment resulted in an increase of stress in the prestressed tendons of less than 2% of the ultimate strength of the tendon. This was considered negligible and demonstrates that pressurization of the containment causes negligible cycling of loads on the tendons.

5.8.8.5 Liner Plate and Penetration Surveillance Program

5.8.8.5.1 Basis for Program

The liner plate system (plate plus anchors) and the penetration assemblies were a relatively new design. Therefore, a surveillance program was established to provide further assurance that these components would maintain their functional integrity (Reference 21). The liner plate system examination included taking inward deformation measurements, inspecting for anchor/plate contact, and inspecting the liner plate for indications of strain concentration. The penetration assembly examination consisted of inspecting assemblies for indications of strain concentrations.

The liner plate and penetration surveillance program was established to satisfy the requirements of the Plant Technical Specifications.

5.8.8.5.2 Surveillance Period

Surveillance was conducted at the following times:

1. Before the pressurization phase of the structural integrity test (PPSIT) - March 22, 1970

2. After the PPSIT - March 31, 1970

3. One year after initial start-up - February 27 and March 2, 1973

4. One-and-one-half years after initial start-up - August 20, 1973

The initial program consisted of three scheduled surveillances. However, at the 1-year surveillance, 2 of the 4 sets of profile measurement points could not be relocated. To compensate for their loss, it was decided to establish two new sets of points, in the general area of the lost points, and to conduct an additional surveillance at 1-1/2 years after start-up.

5.8.8.5.3 Details of Program

Prior to the PPSIT, the following surveillance operations were performed:

1. Four liner plate areas were chosen that had inward deformations from the theoretical radius of the liner plate. These areas were scattered at various elevations and azimuths around the containment. For each area, profile measurement data was obtained at 9 equally spaced points along a horizontal 60-inch chord which was set to span 4 anchor positions. These areas were permanently marked. Containment internal air temperature and external concrete temperature were measured adjacent to these areas.

2. The areas of the liner that were profiled were also visually inspected for indications of strain concentrations such as cracks in the paint, welds or liner plate. These inspections were performed after removing any grease or foreign material from the plate.

3. Seven penetration assemblies were visually inspected for indications of strain concentrations such as large deformations or cracks in the paint, welds or assembly material. These inspections were performed after removing any grease or foreign material from the assemblies.

After the PPSIT, profile measurements were taken on three additional occasions (see Subsection 5.8.8.5.2) and temperatures were recorded twice more. Also, at each post-SIT surveillance, all profiled areas and the seven penetration assemblies were reinspected for indications of strain concentrations. During the 1-1/2-year surveillance, subtracted of the four liner plate areas were performed using a ball peen hammer.

5.8.8.5.4 Summary

Comparison of matched sets of profile data indicated insignificant liner plate movement with relative measurements differing by less than .04 inch at all locations. No evidence of strain concentrations were observed at the areas where the liner plate was profiled at the seven penetration assemblies. Examination indicated that the liner plate was in contact with the concrete at all anchor locations within the examined areas. Examinations also indicated that a separation between the liner plate and the concrete existed in a few areas between adjacent anchors. This condition was acceptable and it was considered in the design of the liner plate system.

On the basis of these results, it was concluded that the liner plate system and penetration assemblies were performing as predicted. Therefore, the surveillance program was terminated.

5.8.8.6 End Anchorage Concrete Surveillance

5.8.8.6.1 Basis for Program

The concrete adjacent to the tendon anchorages (end anchorage concrete) is vital to the overall structural integrity of the containment. At these locations virtually all of the tendon force is transferred into the concrete. If the end anchorage concrete was to fail, the containment would remain standing but it would not be capable of maintaining its structural integrity under DBA or seismic events.

The post-tensioning process has demonstrated that the end anchorage concrete can withstand loads in excess of the tendon seating load. The tendons were stressed to 0.8 f's and seated at approximately 0.7 f's. However, since the containment was a new design, a surveillance program was established to provide further assurance that the end anchorage concrete would maintain its structural integrity (Reference 22). This program included inspecting selected buttress locations for excessive cracking. Buttress anchorages were chosen for two reasons. First, the hoop prestressing force per foot of containment height is significantly larger than the meridional prestressing force. And second, the buttress is a geometric abnormality which, if not properly designed and constructed, could shear off under the action of prestressing loads alone.

The end anchorage concrete surveillance program was established to satisfy the requirements of the Plant Technical Specifications.

5.8.8.6.2 Surveillance Period

The pressurization phase of the structural integrity test (PPSIT) was conducted between March 23 and March 31, 1970. Observations were made before, during and after the PPSIT as follows:

1. One day before the start of pressurization
2. At the following pressures during pressurization: 28, 55 and 63 psig
3. At the following pressures during depressurization: 55 and 28 psig
4. One day after the end of depressurization

Subsequent observations were made at the following times:

1. February 12, 1971 (Winter)
2. June 7, 1971 (Summer)

5.8.8.6.3 Surveillance Locations

Seven surveillance locations were chosen along the buttress at azimuth 205 degrees. The interval between selected tendon anchorages was 20 to 40 feet with the smaller intervals in the top half of the buttress as shown in Figure 5.8-28. An eighth location was chosen in the ring girder at this azimuth to observe the anchorage of a dome tendon. In addition, the anchorage of Tendon EA-5 was selected (azimuth 205 degrees, elevation 598 feet 6 inches). Since no tendons were installed in the spare sheaths above, below and opposite this tendon, this location represented the largest eccentric load on a buttress.

During the March 1970 inspection period, all nine locations were visually examined. The five locations found to have the most significant cracking were reinspected during subsequent inspection periods.

5.8.8.6.4 Details and Results

1. Cracking Observations

   During the March 1970 inspection period, a visual examination of all the surveillance locations was performed at each of the selected pressure levels. A hand-held optical comparator was used to measure the width of the observed cracks, and the length of cracks was measured. A single drawing of the crack patterns, including widths, was made for each location. These drawings utilized a different symbol for each pressure level.

   The width of cracks observed prior to pressurization ranged from 0.001 inch to 0.005 inch. Most of these cracks did not increase in width during pressurization although some grew in length.

   A few minor cracks developed during the pressurization of the containment. As an example, a crack of 0.002 inch in width developed in the ring girder location at the 55 psig pressure but it did not increase in width under the 63.3 psig pressure.

   The five locations found to have the most significant cracking were reinspected during subsequent inspection periods. No change in the crack pattern or their width was observed during these reinspections.

   The following temperatures were observed during the surveillances:

	Inside Temp (°F)	Outside Temp (°F)
March 1970	-	35 to 48
February 1971	68	22
June 1971	85	92

2. Movement

During the March 1970 inspection period, the end anchorage assembly of Hoop Tendon EA-5 was instrumented in order to measure anchor movements. Five dial gage extensometers were utilized. They were set up to measure radial and tangential displacements relative to a test rig anchored to the containment. The dial gage displacements were recorded for each pressure level.

5.8.8.6.5 Summary

Cracking observed prior to pressurization was attributed to surface shrinkage. During this test some additional cracking was observed in the mapped areas due to the increased internal pressure. Some new cracks appeared. While most of the prepressurization cracks did not increase in width, some became longer. Subsequent cold and hot weather surveillance failed to locate any new cracking. All observed crack widths were smaller than those commonly found in concrete structures.

The recorded movements of the EA-5 end anchor assembly were very, very small. They have been attributed to thermal expansion and contraction of the test rig.

Based upon these results, it was concluded that the end anchorage concrete was sound and free of significant cracking. Therefore, the surveillance program was terminated.

5.8.9 STEAM GENERATOR REPLACEMENT CONSTRUCTION OPENING

5.8.9.1 General Description

The Steam Generator Replacement Project necessitated the installation of a temporary construction opening on the southeast quadrant of the containment wall near buttress "F" @ azimuth 145°. The temporary construction opening was centered at azimuth 120° and elevation 664; and was approximately 26 feet wide by 28 feet high as shown in Figure 5.8-29.

This required the detensioning and removal of tendons, cutting concrete, reinforcing steel, tendon sheathing, and the liner plate in the area of the construction opening. The replaced rebar, liner plate, and tendons either met or exceeded the requirements of the original design, both in terms of material requirements and quality provided. The replaced rebar were CADWELD to the existing rebar. The liner plate was welded and subjected to NDE consistent with that required in the original design. Upon installation completion of the new steam generators and closing of the temporary construction of the new steam generators and closing of the temporary construction opening, a Structural Integrity Test (SIT) was performed.

5.8.9.2 Containment Reevaluation

The analysis and criteria for the reevaluation of the containment was similar to the existing requirements contained in Sections 5.8.3, 5.8.4, and 5.8.5 with the clarifications noted below. The reevaluation analysis differed from the original analysis only where it was necessary to properly represent the effects of the temporary construction opening. This evaluation addressed conditions both with the concrete in the opening removed (plant shutdown) and with the concrete replaced and the tendons retensioned.

The reanalysis was performed utilizing a 3D finite element model which consists of approximately 1400 thin shell elements and the BSAP computer program. This computer program is fully verified in accordance with QA program requirements.

The horizontal tendons from approximately elevation 637' to elevation 691' and the vertical tendons from approximately azimuth 49N to azimuth 189N were detensioned prior to removing the existing concrete from the temporary construction opening. Detensioning these tendons provided a means to maintain a comparable prestress level between the existing and replacement concrete after retensioning. A second precaution to minimize differences in prestress level between existing and replacement concrete was to provide a mix design which closely simulated the properties of the existing concrete. Development of the mix design and associated testing was conducted by the University of Illinois at Urbana-Champaign.

After detensioning the tendons a small amount of residual prestress remained in the concrete around the proposed temporary construction opening. To properly capture the effect of this residual prestress, two separate containment configurations, and the associated load components, were considered:

a. Detensioned Containment (concrete in opening removed)

   i. Dead Load
   ii. Prestress
   iii. Seismic OBE and SSE
   iv. Thermal
   v. Wind
   vi. Crane Load (for steam generator lift)

b. Retensioned Containment (opening replaced)

   i. Dead Load (replaced concrete)
   ii. Prestress (retensioned tendons)
   iii. Operating Thermal (increase from the open containment configuration)
   iv. Design Basis Accident (DBA) Pressure and Temperature
   v. OBE and SSE

To simulate the containment conditions when the concrete was removed from the temporary construction opening, stress results were obtained for the load components of configuration 'a' and evaluated for the normal operation load combinations (e.g., those not including DBA pressure loads) identified in Subsections 5.8.3.1.2 and 5.8.3.1.3. In addition to normal operating loads, the effect of a concurrent OBE, extreme wind, and tornado loads were also separately evaluated.

To evaluate the containment with the concrete replaced and the tendons retensioned, the results of configuration 'a' and 'b' were combined. More specifically, to simulate the presence of the residual prestress, the load components for configuration 'b' included only the additional effects imposed on the containment by the replaced concrete and the tendon retensioning operation. These results were added algebraically to load components 'i', 'ii', 'iii', and 'iv' of configuration 'a' to represent the total stress condition after retensioning. All load cases identified in Subsections 5.8.3.1.2 and 5.8.3.1.3 were either evaluated or enveloped. For example, the load case which includes wind load was shown to be enveloped by the seismic load cases.

Since the replacement concrete will not have the same properties as the original concrete, either at the time of loading or at the end of licensed plant life, a parametric study was performed comparing the effect that these differences may have on the distribution in stresses. Both the properties at the time of tensioning and at the end of licensed plant life were considered. The properties of the original concrete and the replacement concrete, at their current age and at the end of licensed plant life, were based both upon test results and the methodology included in ACI 209R. From this parametric study, a representative Young's Modulus was selected for the replacement concrete to account for the potential differences in the original and replacement concrete.

The results of the analyses, which were thoroughly reviewed by the NRC staff, indicate that the containment remains within the acceptance criteria contained in Subsection 5.8.5.2 and no margins are reduced.

5.8.9.3 Materials

5.8.9.3.1 Concrete

1. Ingredients

   Cement - ASTM C 150, Type II

   Air Entraining Agent - Micro-Air manufactured by Master Builders

   Water Reducing Agent - Rheobuild 1000 (superplasticizer) manufactured by Master Builders

   Fine Aggregate - ASTM C 33

   Coarse Aggregate - ASTM C 33, No. 57 aggregates from North Branford Quarry, Tilcon Connecticut, Inc.

2. Strength

   5000 psi at 7 days

3. Principal Placement Properties

   temperature, maximum – 60NF

   air content - 3% to 7%

   slump working limit(1) - 4 inches

   slump rejection limit(1) - 8 inches

(1) Slump limits apply to the point of discharge of the delivery truck or the pump line when pumping is used. The slump at the sampling point should be less than or equal to the working limit. Slumps between the working limit and rejection limit are within the "inadvertency margin" for occasional batches that may inadvertently exceed the working limit.

4. Concrete Mix Design

The University of Illinois at Urbana-Champaign developed the concrete mix design and performed the associated testing. The concrete had to meet the high early strength requirement along with exhibiting low creep characteristics.

The concrete mix design is shown below:

	Weight[(1)]
Fine aggregates	1376 lb
Coarse aggregates	1897 lb
Cement	719 lb
Water	231 lb
Air-entraining admixture (Micro-Air) [(2)]	846 ml
Water-reducing admixture (Rheobuild 1000)[(2)]	3217 ml

(1) Weight is per cubic yard of concrete on a Saturated Surface Dry (SSD) basis.

(2) During concrete placement, field personnel were permitted to adjust the admixtures to achieve the specified slump and air content provided the amounts did not exceed the maximum limits recommended by Master Builders.

5. Masterpatch 20

"Masterpatch 20", manufactured by Master Builders, was used at the top of the temporary construction opening. The "Masterpatch 20" met the following requirements:

strength - 5000 psi at 7 days

temperature, maximum – 60NF

slump working limit - 3 inches

slump rejection limit - 7 inches

5.8.9.3.2 Reinforcing Steel

ASTM A 615, Deformed - Grade 60 (fy = 60 ksi)

5.8.9.3.3 Prestressing Tendons and Hardware

Tendon Wires - ASTM A 421, Type BA

Anchor Head - HR C1141

Tendon Sheathing - ASTM A 527, 24 gage

5.8.9.3.4 Liner Plate and Hardware

Liner Plate - ASTM A 442, Grade 55

Structural Shapes, Plates, and Backing Strips - ASTM A 36

5.8.9.3.5 Sheathing Filler

Visconorust 2090-P4

5.8.9.4 Quality Control

5.8.9.4.1 Concrete Mix Design

1. Aggregates

   Aggregates to be used in the concrete design mix were tested in accordance with the ASTM standards listed in Subsection 5.8.7.2.1, Part 1, along with the following additional standards.

   ASTM C 123 - Standard Test Method for Lightweight pieces in Aggregate

   ASTM C 535 - Standard Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine

   CRD C 119 - Flat and Elongated Particles for Aggregates

   The following standards were not used for testing the aggregates.

   ASTM C 87 - Mortar-Making Properties

   ASTM C 88 - Soundness

   ASTM C 227 - Potential Reactivity (Mortar Bar)

2. Mixes

The concrete mix design was developed by the University of Illinois at Urbana-Champaign in accordance with ACI 211.1. The concrete design mix shown in Subsection 5.8.9.3.1, Part 4, was tested in accordance with the ASTM standards listed in Subsection 5.8.7.2.1, Part 2, along with the following additional standards.

ASTM C 29 - Standard Test Method for Unit Weight and Voids in Aggregate

ASTM C 125 - Standard Definitions of Terms Relating to Concrete and Concrete Aggregates

ASTM C 138 - Standard Method of Test for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete

ASTM C 150 - Standard Specification for Portland Cement

ASTM C 469 - Standard Method of Test for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression

ASTM C 566 - Standard Method of Test for Total Moisture Content of Aggregate by Drying

ASTM C 617 - Standard Method of Capping Cylindrical Concrete Specimens

ASTM D 75 - Standard Practice for Sampling Aggregates

5.8.9.4.2 Concrete Materials

Concrete material testing for water, cement, and fine and coarse aggregates was performed by an independent testing laboratory prior to use by the University of Illinois at Urbana-Champaign and the concrete batch plant.

All production sampling and testing was performed by an independent testing laboratory. Samples were taken at the point of discharge of the delivery truck or the pump line when pumping was used, and included the following:

a. Compressive strength tests for concrete according to ASTM C 31, ASTM C 39 and ACI 318, Section 4.7

b. Slump tests according to ASTM C 143

c. Air content tests according to ASTM C 231

d. Temperature

e. Unit weight according to ASTM C 138

f. Compressive strength tests for grout according to ASTM C 109

5.8.9.4.3 Concrete

Sixteen concrete compression cylinders were cast in accordance with ASTM C 172 for each 50 cubic yards, or fraction thereof, placed. The cylinders were tested according to the following schedule:

3 each at 3, 7, 14, and 28 days

Slump, temperature, air content, and unit weight tests were performed when cylinders were prepared for the compressive strength test.

5.8.9.4.4 Reinforcing Steel and CADWELD Splices

Reinforcing steel and CADWELD splices are in general accordance with the requirements contained in Subsection 5.8.7.2.4.

CADWELD splices were tested to determine conformance with the following standards:

a. The strength of each sample tested shall equal or exceed 125% of the minimum yield strength of the reinforcing bar.

b. The average strength of each distinct group of 15 consecutive samples shall equal or exceed the minimum ultimate tensile strength specified for the reinforcing bar.

CADWELD splices were tensile tested in accordance with the following schedule for each position, for each bar size, and for each splicing crew. Steps 'b' and 'c' may be prorated based on the actual quantity of production splices made:

a. Three out of the first 10.

b. Six out of next 90.

c. Four out of next and subsequent units of 100 splices.

In addition to the splices made for tensile testing, spare splices were made for testing, if the initial samples failed to meet the provisions stated above. These spare splices were made in accordance with the following schedule:

a. Three out of the first ten.

b. Three out of the next 90.

c. Six out of next and subsequent units of 100 splices.

Test CADWELD splices were made-by having test bars of 3-foot length spliced in sequence with the production bars.

5.8.9.4.5 Prestressing Tendons

Sampling and testing of tendon wires conformed to ASTM A 421 and as stated in Subsection 5.8.7.2.5.

All prestressing installation work was continuously under the supervision of a Project Engineer and in general accordance with Subsection 5.8.7.2.5.

5.8.9.4.6 Liner Plate

The tolerances on the liner plate erection were based on the following criteria:

An 8-foot-long template curved to the required radius shall not show deviations of more than 1/4-inch when placed against the completed surface of the shell within a single plate section and not closer than 12-inches at any point to a welded seam. When the template is placed across one or more welded seams, the deviation shall not exceed 1/2-inch. The effect of change in plate thickness or of weld reinforcement shall be excluded when determining deviations.

Maximum inward deflection (toward the center of the structure) of the 1/4-inch plate between the angle stiffeners spaced at 15-inches shall be 3/16-inch when measured using a 15-inch straightedge placed horizontally against the plate. When the 15-inch straightedge is placed horizontally across the welded seam at the buttresses, the maximum inward deflection shall be 1/8-inch.

Sharp bends are not permitted unless provision was made for them in the design. A sharp bend is defined as any local bend that deviates from the design radius or a vertical straightedge by an offset of more than 1/2-inch. The template used to measure the local deviations shall be only one to two feet longer than the area of the deviation itself.

All welding inspection was performed in general accordance with the requirements of Subsection 5.8.7.2.6.

5.8.9.4.7 Sheathing Filler

The sheathing was supplied in general accordance with the requirements of Subsection 5.8.7.2.8.

5.8.9.5 Construction Methods

5.8.9.5.1 Governing Codes

Portions of the following codes and manuals were used for replacement of the temporary construction opening:

ACI 301 -	Specification for Structural Concrete for Buildings
ACI 302.1R -	Guide for Concrete Floor and Slab Construction
ACI 304 -	Guide for Measuring, Mixing, Transporting, and Placing Concrete
ACI 305R -	Hot Weather Concreting
ACI 306R -	Cold Weather Concreting
ACI 308 -	Standard Practice for Curing Concrete
ACI 309 -	Standard Practice for Consolidation of Concrete
ACI 318 -	Building Code Requirements for Reinforced Concrete
ACI 347 -	Recommended Practice for Concrete Formwork
ACI SP 2 -	Manual of Concrete Inspection
AISC -	Steel Construction Manual
ASME -	Boiler and Pressure Vessel Code

5.8.9.5.2 Concrete

The concrete placement in the temporary construction opening was done at the maximum rate of 4 feet per hour.

"Masterpatch 20" was used in the top portion of the temporary construction opening with limitation that the depth could not exceed 14 inches.

5.8.9.5.3 Reinforcing Steel and CADWELD Splices

Reinforcing steel and CADWELD splices were installed in accordance with Subsection 5.8.7.3.3.

5.8.9.5.4 Prestressing System

The tendons were detensioned, removed, and retensioned in accordance with the sequence shown in Figure 5.8-30.

The tendon installation is in general accordance with Subsection 5.8.7.3.4.

5.8.9.6 Containment Testing

5.8.9.6.1 Integrated Leak Rate Testing

A successful primary reactor containment Integrated Leak Rate Test (ILRT) was performed on the containment building during the period from February 13 through February 17, 1991.

5.8.9.6.2 Structural Integrity Test

A successful primary reactor containment Structural Integrity Test (SIT) was performed on the containment building during the period from February 13 through February 17, 1991.

The containment was pressurized to 63.25 psig and then depressurized with various intermediate constant pressure holds as required and to conduct the Integrated Leakage Rate Test. The maximum test pressure was 0.25 psig above the minimum specified level of 63.25 psig, which is 1.15 times the 55 psig containment design pressure. Containment shell displacements and surface crack widths were measured during the pressure cycle. In addition, accessible tendon end anchorages were examined prior to the start of pressurization, at maximum test pressure and following the completion of depressurization. Containment exterior surfaces were examined for signs of damage before and after the test.

Containment shell displacements varied essentially linearly with pressure demonstrating that the concrete remained within the elastic range. Measured vertical and average radial displacements at maximum test pressure remained below predicted (and well below allowable) values which confirms that the assumptions used in the analysis are conservative. Residual displacements at the completion of depressurization were within allowable values.

The maximum crack width growth was 0.001 inch in any of the five examination areas. No signs of structural damage were observed during the tendon end anchorage and general examinations.

The results of the Structural Integrity Test demonstrate that the containment is fully restored to the design condition existing prior to the steam generator replacement (Reference 27).

5.8.10 CONTAINMENT STRUCTURAL INTEGRITY SURVEILLANCE PROGRAM

In accordance with NRC rule 61CFR154, Palisades was required to implement ASME Section XI, 1992 Edition, 1992 Addenda, Subsection IWE and IWL by September 9, 2001. The portion of Subsection IWE and IWL associated with repairs and replacements became effective September 9, 1996, although this is not specifically described in the NRC rule. Palisades currently implements ASME Section XI, Subsections IWE and IWL, 2004 Edition. The containment inservice inspection, testing and aging management program is described in Site Program Section SEP-CISI-PLP-001.

5.8.10.1 Requirement for Metal Containment Examinations

Examinations, inservice inspection and repair and replacement of the containment metallic liner, penetration assemblies and integral attachments shall be performed in accordance with the ASME Section XI, Subsection IWE.

5.8.10.2 Requirements for Concrete Containment Examinations

Examination, inservice inspection and repair and replacement of the containment reinforced concrete and post-tensioning systems shall be performed in accordance with the ASME Section XI, Subsection IWL.

5.8.10.2.1 Surveillance of Unbonded Post-Tensioning System

The unbonded post-tensioning system of the Palisades Nuclear Plant, Containment Building shall be examined in accordance with the requirements of IWL-2520 once per 5 years, based on the 1987 completion Technical Specification Change Amendment Number 109. Examinations shall commence not more than 1 year prior to the specified date and shall be completed not more than 1 year after the specified date.

5.8.10.2.2 Surveillance of Containment Structural Concrete

Concrete containments shall be inspected at 5 year intervals following the completion of the Structural Integrity Test (SIT). The Palisades schedule is based on the 1987 completion of Technical Specification Change Amendment Number 109. The 10-year and subsequent examinations shall commence not more than 1 year prior to the specified dates and shall be completed not more than 1 year after such dates.

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5.9 OTHER STRUCTURES

5.9.1 DESIGN CRITERIA

5.9.1.1 CP Co Design Class 1 Structures

CP Co Design Class 1 structures, except the containment and the auxiliary building TSC/EER/HVAC addition, were designed in accordance with the design criteria presented below. In general, reinforced concrete structures were designed using the ultimate strength method, and steel structures were designed using the working stress method. The containment was designed in accordance with the criteria set forth in Subsections 5.8.3 and 5.8.5. The auxiliary building TSC/EER/HVAC addition was designed in accordance with the criteria set forth in Subsection 5.9.6. CP Co Design Class 1 structures are defined in Section 5.2.

5.9.1.1.1 Design Methods

General

CP Co Design Class 1 structures were designed in accordance with the provisions of ACI 318-63, Building Code Requirements for Reinforced Concrete, and Part 1 of the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, 1963 edition.

Beginning in 1989, CP Co Design Class 1 structures were designed in accordance with the AISC Structural Steel Specification, Eighth Edition, 1980.

Accident, Wind and Seismic Conditions

In general, CP Co Design Class 1 structures were designed to maintain elastic behavior when subjected to various combinations of dead loads, accident loads, thermal loads and wind or seismic loads. The upper limit of elastic behavior was considered to be the yield strength of the effective load-carrying structural materials. The yield strength for steel, including reinforcing steel, was considered to be the guaranteed minimum given in the appropriate ASTM specification. Concrete structures were designed for ductile behavior whenever possible, that is, with the steel stress controlling the design. The ultimate strength provisions of the ACI 318-63 code were used in determining "Y," the required yield strength of the structure.

Under the action of dead load, live load and the OBE load, calculated stresses were limited to the code allowable values. However, for the wind combination, allowable stresses were increased 33-1/3%. For the containment interior structures, under the action of dead load, live load and thermal loads, the calculated stresses were limited to 133% of the code allowable values.

The deflections of supports for critical equipment (all CP Co Design Class 1 and CP Co Design Class 2 equipment) and structures housing this equipment were reviewed to ensure that they would not impair the functioning of the critical equipment. Local yielding of pipe whip restraints, and jet impingement and missile barriers was permitted provided there would be no general failure of the barrier. The deflections caused by this local yielding were checked to ensure that affected CP Co Design Class 1 systems and equipment, except reactor vessel internals under DBA loadings, were not stressed beyond the limits given in Subsection 5.10.1.

5.9.1.1.2 Loads and Load Combinations

Normal Operating Conditions

For CP Co Design Class 1 structures, the loads and load combinations used for normal operating conditions were in accordance with the provisions of the codes specified in Subsection 5.9.1.1.1.

Accident, Wind and Seismic Conditions

Accident, wind and seismic loads and their associated load combinations are presented below. The design loads were increased by load factors to reflect built-in conservatism for loads subject to large variations and to reflect the probability that each load would occur simultaneously with certain other loads in specified load combinations. The final design for Class I structures, except the containment shell, satisfied the most severe of the following equations:

1. $Y = 1/\Phi\ (1.25D + 1.0R + 1.25E)$

2. $Y = 1/\Phi\ (1.25D + 1.25H + 1.25E)$(a)

3. $Y = 1/\Phi\ (1.25D + 1.25H + 1.25W)$(a)

4. $Y = 1/\Phi\ (1.0D + 1.0R + 1.0E')$

5. $Y = 1/\Phi\ (1.0D + 1.0H + 1.0E')$

(a) 0.9D was used whenever the addition of the dead load stress reduced the critical stress.

Where:

Y = required yield strength of the structures.

Φ = yield capacity reduction factor (see discussion in Subsection 5.8.5.2.4).

Φ = 0.90 for reinforced concrete in flexure.

Φ = 0.85 for shear (diagonal tension), bond and anchorage in reinforced concrete.

Φ = 0.75 for spirally reinforced concrete compression members.

Φ = 0.70 for tied reinforced concrete compression members.

Φ = 0.90 for fabricated structural steel.

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

R = force and/or pressure on structure due to rupture of any one high-energy line. The following pipe rupture loads are included, as appropriate: pipe reactions, jet impingement, pipe whip and containment pressurization.

H = force on structure due to thermal expansion of pipes under operating conditions.

E = OBE loads resulting from a ground surface acceleration of 0.1 g. The seismic analysis of CP Co Design Class 1 structures is presented in Section 5.7.

E' = SSE loads resulting from a ground surface acceleration of 0.2 g. The seismic analysis of CP Co Design Class 1 structures is presented in Section 5.7.

W = wind loads resulting from a 100 mi/h wind. Wind loads are presented in Subsection 5.3.1.

In a few instances, such as the floor of the control room, the yield strength load combinations presented above did not produce more conservative results than ACI 318-63, Equation 15-1 (U = 1.5D + 1.8L). In these cases, Equation 15-1 was used to determine the required yield strength.

Tornado

The forces resulting from a tornado (wind pressure and differential pressure) were combined with dead loads only. Dead loads included piping and all other permanently attached or located items. No live loads were included. The only significant live loads were considered to be crane loads, and sufficient time would be available after sighting a tornado to remove these loads. Tornado loads are presented in Subsection 5.3.2.

For both structural and reinforcing steel, the tensile stress was limited to 0.9 f_y unless it was demonstrated that a safe shutdown could be assured with a higher stress limit and some inelastic deformation. For concrete, the compressive stress was limited to 0.85 f'_c except that local crushing was permitted at missile impact zones. In all cases, structures supporting or housing critical equipment were reviewed to ensure that their deflections would not result in a loss of equipment function.

Missiles

Credible missiles and their effect on the CP Co Design Class 1 structures are discussed in Section 5.5.

OTHER

Loads common to all structures (CP Co Design Classes 1, 2 and 3) are presented in Subsection 5.9.1.4.

5.9.1.2 CP Co Design Class 2 Structures

There are no CP Co Design Class 2 structures according to the definition presented in Subsection 5.2.2.2.

5.9.1.3 CP Co Design Class 3 Structures

CP Co Design Class 3 structures, as defined in Subsection 5.2.2.3, were designed in accordance with the provisions of the following codes insofar as they were applicable: ACI 318-63, Building Code Requirements for Reinforced Concrete; AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, Part I, 1963 edition; and the Uniform Building Code (UBC), 1964 edition. Seismic design was in accordance with UBC-64 with the appropriate working stress allowance and shear coefficients. Wind loads conformed to the requirements of Section 2308 of UBC-64. The basic wind pressure at the Palisades site is 30 lb/ft^2 as defined in UBC-64.

5.9.1.4 Loads Common to All Structures

The following loads were common to all structures:

1. Hydrostatic - All below grade portions of structures were designed for hydrostatic pressures based upon a groundwater elevation of 585 feet.

2. Flooding - No loads were considered for floods. However, as part of the Nuclear Regulatory Commission's (NRC) Systematic Evaluation Program (SEP), CP Co Design Class 1 structures were evaluated for their ability to withstand a "design basis flood" (see Subsection 5.4.1).

3. Ice or Snow - A uniformly distributed live load of 40 pounds per square foot (50 lb/ft^2 for auxiliary building TSC/EER/HVAC addition) on all roofs provided for any anticipated snow and/or ice loading at the Palisades Plant.

4. Temperature - The Plant was designed for an outside ambient air temperature range of -25°F to 95°F.

5.9.2 CONTAINMENT INTERIOR STRUCTURES

5.9.2.1 General Description

The containment interior structures, which consist of all structural elements within the containment shell, are CP Co Design Class 1 structures. The principal interior structures are:

1. The primary shield wall which forms the reactor cavity

2. Two steam generator compartments

3. A refueling pool which is located between the steam generator compartments and above the reactor cavity

4. An enclosed sump under the reactor cavity

5. Major equipment supports including the steam generator pedestals

Structures 1 through 4 are shown in Figure 5.9-1. The layout of the containment building is shown in the figures in Section 1 of the FSAR.

The steam generator compartment walls form the secondary shield walls around the primary coolant loops. The primary functions of these walls are:

1. To provide biological shielding,

2. To provide missile barriers, and

3. To provide barriers to resist the jet impingement loads, and associated compartment pressurization, resulting from the rupture of any one primary coolant pipe.

These compartment walls span horizontally between vertical beams which, in turn, are restrained by the floor systems at elevations 607 feet 0 inch and 649 feet 0 inch and by structural steel tie struts at intermediate levels.

The roof slab of the sump is supported by the primary shield wall.

Pipe whip restraints are provided for the primary coolant pump suction, main steam, feedwater and other high-pressure piping in accordance with the criteria presented in Subsection 5.6.3. Pipe whip barriers are also provided in accordance with the criteria in the aforementioned subsection. For example, a reinforced concrete slab is provided above the top head of the pressurizer to protect it from a ruptured main steam line.

5.9.2.2 <u>Loads</u>

The loads for the design of the containment interior structures, which are discussed below, amplify, or are in addition to, those presented in Subsection 5.9.1.1.

The containment interior structures were designed to resist the jet impingement forces resulting from the rupture of a single pipe in the Primary Coolant System. This included consideration that the jet could knock out any one of the steel tie struts mentioned above.

Missile barriers for internally generated missiles are discussed in Subsection 5.5.3.

The following temperature differentials were considered in the design of these structures:

Wall	Temperature	Basis
Primary shield wall	33°F (Effects on the reactor sump cover slab was not considered)	Temperature gradients through the primary shield wall during reactor operation (see Figure 9-3 through 9-6)
Refueling pool walls and slabs	50°F	Allowance for radiation from reactor head and control rod drives
Steam generator compartment walls and slabs	0°F	Temperature equalization following a DBA occurs so quickly that these compartments never see a differential temperature

The design pressure differentials across walls and slabs of enclosed compartments of the containment structures were as follows:

Primary shield wall	72 psid (bursting)
North steam generator compartment walls	31 psid (bursting)
South steam generator compartment walls	27 psid (bursting)
Containment sump roof slab	7.3 psid (uplift)
Operating floor slab outside steam generator compartments	6 psid (uplift)

Except for the operating floor slab outside the steam generator compartments, the analysis on which the above listed compartment pressures are based is discussed in Section 14.18.3.

The primary shield wall was also designed to contain safety injection water up to the level of the reactor nozzles.

5.9.2.3 Analysis and Design

General

The containment interior structures were designed in accordance with the design criteria presented in Subsection 5.9.1.1 with the exceptions discussed below. Also, wind and tornado loads were not applicable.

The deflections of supports for engineered safeguards equipment and structures housing this equipment were reviewed to ensure that they would not impair the functioning of the engineered safeguards equipment.

Pipe whip restraints and jet impingement and missile barriers were designed in accordance with the yield strength criteria of Subsection 5.9.1.1 except that local yielding was permitted provided there would be no general failure of the barrier.

Walls and slabs were checked for shear stresses resulting from loads on adjacent walls and slabs which are in a different plane. Reinforcing in addition to that required for bending was provided where there were axial tensions introduced from these adjacent walls. Axial compressive forces were combined with bending moments in accordance with the rules in ACI 318-63.

Primary Shield Wall

In May of 1975, the NRC was informed by one utility that asymmetric loading resulting from a postulated pipe rupture could have a significant effect on reactor vessel support structures. Subsequent investigation made by the NRC resulted in the issuance of NUREG-0609, "Asymmetric Blowdown Loads on PWR Primary System." Utilities who possessed PWR nuclear plants were to evaluate the integrity of reactor vessel support structures.

In 1981, Palisades primary shield wall was evaluated for the most critical asymmetrical load resulting from a guillotine pipe break at the Reactor Vessel Inlet Nozzle 1A (see Reference 1). A finite element computer model was developed. The resultant element stresses were compared to the actual structural capacities. The results of this analysis indicated that the primary shield wall has the ultimate strength to resist the subject pipe rupture load.

<u>Major Equipment Supports</u>

For the reactor vessel, steam generators, pressurizer, primary coolant pumps and safety injection tanks, supports and tie-downs not provided by the equipment vendor were designed in accordance with the yield strength criteria of Subsection 5.9.1.1, except as noted below. The design was controlled by Equations 1 and 4 because the effects of pipe rupture loads were more severe than those created by the operating thermal loads. The rupture loads are much larger than the seismic loads.

For the design of these supports, the definitions of "D" and "R" found in Subsection 5.9.1.1.2 were replaced by the following:

D = Operating weight of equipment. In the case of the reactor vessel, steam generators and primary coolant pumps, the effect of thermal loads, erection "cold spring" and frictional effects of thermal expansion are also included.

R = Force on support caused by pipe reactions and jet impingement associated with rupture of the following lines:

Reactor Vessel - 42-inch hot leg or 30-inch cold leg

Steam Generators - 42-inch hot leg, 30-inch cold leg, main steam line or feedwater line

Primary Coolant Pumps - 30-inch cold leg suction or discharge line

Pressurizer - 12-inch pressurizer surge line

Safety Injection Tanks - 12-inch injection line

Stresses were limited to the yield strength of the effective load carrying structural materials (see Table 5.9-1) reduced by an appropriate yield capacity reduction factor. No limits were specified for allowable strains since the supports are not permitted to yield.

NSSS equipment supports were reevaluated during 1979 and 1981 for loads derived from the actual support capacities and asymmetrical loads resulting from guillotine pipe break. The support structures were found to be adequate under the postulated loads.

5.9.2.4 Materials of Construction

The major structural materials used in the construction of the containment interior structures, except for the major equipment supports, are as follows:

Concrete	f'_c = 4,000 psi at 90 days
Reinforcing steel	ASTM A-432
Structural steel	ASTM A-36
High-strength alloy steel coolant pipe rupture stops)	ASTM A-514, Type F (for primary
Refueling pool stainless steel liner plate	ASTM A-167, Type 304L

The major structural materials used in construction of the major equipment supports are shown in Table 5.9-1.

5.9.3 AUXILIARY BUILDING

5.9.3.1 General Description

The auxiliary building, with the exception of the administration area and the access control area, is a CP Co Design Class 1 structure. Although the enclosure over the spent fuel pool is designated CP Co Design Class 1, it is not designed for tornado loads. The layout of the auxiliary building is shown in the figures in Section 1 of the FSAR.

The following facilities, systems and equipment are among those located in the auxiliary building:

1. Component Cooling System (majority)

2. Containment Spray System (majority)

3. Control room

4. Emergency diesel generators and related auxiliaries

5. New and spent fuel handling, storage and shipment facilities

6. Radwaste, chemical and volume control equipment

7. Safety Injection System (majority)

The reinforced concrete enclosure containing the engineered safeguards equipment is located below grade. It is partitioned into two rooms so that one room is operable in the event a pipe rupture floods the other room. The partition wall is designed to withstand hydrostatic loading over its full height.

This building also houses the access control area, which controls access to and exit from the various radiation controlled zones, and includes the various monitoring devices for detecting personnel contamination. Facilities are provided for decontaminating personnel leaving the radiation zones in order to prevent spreading contamination outside the radiation controlled areas.

The new and spent fuel pools are located adjacent to the three-story concrete enclosure which houses the control room, switchgear and the emergency diesel generators. In the event that water is lost from the spent fuel pool due to a tornado, makeup water can be added from Lake Michigan. To ensure that the spent fuel pool and tilt pits have a high degree of leak tightness, the walls and floors of these cavities are lined with stainless steel plates. Monitoring trenches have been provided behind the liner plates to detect any leakage that might occur. In 1977 the spent fuel pool storage capacity was increased by increasing both the number and density of racks. The reevaluation of the fuel pool for the new loads is discussed in Subsection 9.11.3.3.

The main steam and main feedwater lines pass through the southwest corner of the auxiliary building. In this region, pipe whip restraints are provided on the turbine building side of the main steam and main feedwater isolation valves in accordance with the criteria presented in Subsection 5.6.3. These structural steel frames are anchored to various auxiliary building walls and slabs. In addition, the jet forces resulting from a failure in either one of these lines were considered in the design of the walls and slabs that separate these lines from the switchgear and cable spreading rooms and from the containment ventilation isolation valves.

5.9.3.2 Loads

The loads and conditions for the design of CP Co Design Class 1 portions of the auxiliary building, which are listed below, amplify, or are in addition to, those presented in Subsection 5.9.1.1.

1. Flooding in the room housing the engineered safeguards systems due to pipe rupture and the resulting hydrostatic load.

2. Special requirements to prevent criticality of new and spent fuel bundles.

3. The steel frame enclosure over the spent fuel pool was not designed for tornado, wind and differential pressure loads.

5.9.3.3 Analysis and Design

General

The CP Co Design Class 1 portions of the auxiliary building were designed in accordance with the design criteria presented in Subsection 5.9.1.1.

This building is constructed on a mat foundation. The reinforced concrete slabs and walls were designed as two-way slabs and bearing walls, respectively, for dead, live, and wind, or dead and tornado load combinations, wherever applicable. Some walls in the fuel pool area and in the area adjoining the containment structure were designed to act as deep beams. In the design, seismic, wind and other appropriate lateral loads were assumed to be resisted and carried down to the foundations by diaphragm action of the slabs and the shear wall action of the walls. Care was taken to design the concrete to perform in a ductile manner, thereby better enabling it to resist dynamic loads (seismic, missile impact and jet impingement).

Spent Fuel Pool Walls

The spent fuel pool walls will be subjected to a thermal loading only when the pool is filled with water. Hence, the stresses given below are combined stresses due to thermal and hydrostatic loadings. Under normal operating conditions, pool water at 125°F, the maximum combined stresses in the fuel pool walls are as follows:

Compressive stress in concrete 184 psi

Tensile stress in reinforcement 12,240 psi

Under prolonged outage of the fuel pool cooling system, the pool water temperature could reach 212°F. Under these conditions the maximum stresses are:

Compressive stress in concrete 504 psi

Tensile stress in reinforcement 22,888 psi

No special provisions have been made to control cracking of the concrete structure under the above noted thermal stresses, but these cracks are generally minute and do not extend through the full depth of the walls. Moreover, these cracks are not unusual in a reinforced concrete structure. A stainless steel liner has been provided on the inside face of the pool. This liner plate, due to its ductile nature, will absorb the strain due to the cracking of the concrete in the walls and along with the concrete walls will guarantee the leak tightness of the pool for a water temperature less than 212°F.

5.9.3.4 Materials of Construction

The major structural materials used in the construction of this building are as follows:

Concrete	f'_c = 3,000 psi at 28 days
Reinforcing steel	ASTM A-15 (f_y = 40 Ksi)
Structural steel	ASTM A-36
High-strength bolts	ASTM A-325
Fuel pool stainless steel liner plate	ASTM A-167, Type 304L

5.9.4 TURBINE BUILDING AND INTAKE STRUCTURE

5.9.4.1 General

The following areas of the turbine building and the intake structure were designed to CP Co Design Class I standards:

1. Portion of the turbine building basement forming the auxiliary feedwater pump rooms.

2. Portion of the turbine building known as the electrical penetration enclosure.

3. Portion of the intake structure above elevation 590 feet. This room houses the service water pumps, fire pumps/drivers, diesel fuel oil transfer pumps, related electrical support components, etc.

The layout of the turbine building and intake structure is shown in the figures in Section 1 of the FSAR.

The CP Co Design Class 1 portions of the turbine building and intake structure were designed in accordance with the design criteria presented in Subsection 5.9.1.1.

5.9.5 AUXILIARY BUILDING RADWASTE ADDITION

5.9.5.1 General Description

During 1972 a 124-foot x 38-foot x 106-foot-high addition (approximate maximum dimensions) was added to the north side of the auxiliary building. This CP Co Design Class 1 structure houses additional gaseous, liquid and solid radwaste equipment. The layout of the auxiliary building radwaste addition is shown in the figures in Section 1 of the FSAR.

5.9.5.2 Analysis and Design

The auxiliary building addition was designed in accordance with the design criteria presented in Subsection 5.9.1.1 with the following exceptions:

1. Pipe Loads "R" and "H" were not applicable.
2. The 1969 AISC Specification was used in place of the earlier edition.

The building is constructed on a mat foundation. The reinforced concrete slabs and walls were designed as two-way slabs and bearing walls, respectively, for dead, live and wind or dead and tornado load combinations, wherever applicable. In the design, seismic, wind and other appropriate lateral loads were assumed to be resisted and carried to the foundations by diaphragm action of the slabs and the shear wall action of the walls.

5.9.5.3 Materials of Construction

The major structural materials used in construction of this addition are as follows:

Concrete	f'_c = 3,000 psi at 28 days
Reinforcing steel	ASTM A-615, Grade 60
Structural steel	ASTM A-36
High-strength bolts	ASTM A-325

5.9.6 AUXILIARY BUILDING TSC/EER/HVAC ADDITION

5.9.6.1 General Description

During 1983 an addition was appended to the north side of the auxiliary building for a technical support center (TSC), an electrical equipment room (EER) and a heating, ventilating and air conditioning (HVAC) area. The TSC was constructed pursuant to NUREG-0696, the HVAC area as a result of the control room habitability requirements of NUREG-0737 and the EER area as a result of loads placed on the electrical system by the addition of the TSC and HVAC areas. The L-shaped structure was added to the roofs of the existing waste gas decay tank/baler room (elevation 607 feet 6 inches) and the diesel generator exhaust muffler enclosure (elevation 629 feet 2 inches). The TSC/EER portion, which is located above elevation 607 feet 6 inches, is approximately 26 feet x 64 feet x 32 feet high. The HVAC portion which is located above elevation 629 feet 2 inches is approximately 25 feet x 44 feet x 30 feet high. The layout of this addition is shown in the figures in Section 1 of the FSAR.

5.9.6.2 Loads and Load Combinations

5.9.6.2.1 Loads

The following loads were considered in the design of the auxiliary building TSC/EER/HVAC addition:

D = Dead load of structure and equipment. In addition, a portion of the "live load" is added when it includes items such as piping, cable and trays suspended from floors. An allowance is also made for future permanent loads.

L = Conventional floor and roof live loads and movable equipment loads including piping, cable and trays suspended from floors.

R_o = Force on structure due to thermal expansion of pipes under operating conditions.

T_o = Thermal loads due to normal operating conditions.

NOTE: The combination of internal and ambient air temperatures that produces the most critical transient or steady-state thermal gradient shall be used.

E = OBE loads resulting from a ground surface acceleration of 0.1 g. The seismic analysis of this addition is presented in Section 5.7.

E' = SSE loads resulting from a ground surface acceleration of 0.2 g. The seismic analysis of this addition is presented in Section 5.7.

W = Wind loads resulting from a 100 mi/h wind. Wind loads are presented in Subsection 5.3.1.

W' = Tornado loads (wind pressure, differential pressure, missile impact). Tornado loads are presented in Subsection 5.3.2.

5.9.6.2.2 Load Combinations

The auxiliary building TSC/EER/HVAC addition was designed to resist the most severe of the following load combinations:

Concrete Structures

1. $U = 1.4D + 1.7L$
2. $U = 1.4D + 1.7L + 1.9E$ or $U = 1.2D + 1.9E$ for $L = 0$
3. $U = 1.4D + 1.7L + 1.7W$ or $U = 1.2D + 1.7W$ for $L = 0$
4. $U = 3/4(1.4D + 1.7L + 1.7T_o + 1.7R_o)$
5. $U = 3/4(1.4D + 1.7L + 1.9E + 1.7T_o + 1.7R_o)$
6. $U = 3/4(1.4D + 1.7L + 1.7W + 1.7T_o + 1.7R_o)$
7. $U = D + L + T_o + R_o + E'$
8. $U = D + L + T_o + R_o + W'$

Steel Structures

1. $S = D + L$
2. $S = D + L + E$
3. $S = D + L + W$
4. $1.5S = D + L + T_o + R_o$
5. $1.5S = D + L + T_o + R_o + E$
6. $1.5S = D + L + T_o + R_o + W$
7. $1.6S = D + L + T_o + R_o + E'$
8. $1.6S = D + L + T_o + R_o + W'$

Where:

U = required strength to resist factored loads or their related internal moments and forces as defined in ACI 318-77, Building Code Requirements for Reinforced Concrete.

S = required section strength based on the elastic design methods and the allowable stresses defined in Part 1 of the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, 1978 edition.

Load definitions are presented in Subsection 5.9.6.2.1.

5.9.6.3 Analysis and Design

The auxiliary building TSC/EER/HVAC addition was designed in accordance with the ultimate strength provisions of ACI 318-77, Building Code Requirements for Reinforced Concrete, and Part 1 of the AISC Specification for the Design, Fabrication and Erection of Structural Steel Buildings, 1978 edition.

The capacity reduction factors (Φ) used in the design of reinforced concrete were:

Φ= 0.90 for reinforced concrete in flexure

Φ= 0.85 for shear (diagonal tension), bond and anchorage in reinforced concrete

Φ= 0.75 for spirally reinforced concrete compression members

Φ= 0.70 for tied reinforced concrete compression members

Φ= 0.90 for reinforcing steel in direct tension

Design allowables for steel were determined in accordance with Part 1 of the AISC specification, with the following exceptions:

1. The maximum total stress in bending and tension did not exceed 0.9 f_y.

2. The maximum total stress in shear did not exceed 0.5 f_y.

5.9.6.4 Materials of Construction

The major structural materials used in the construction of this addition are as follows:

Concrete	f'_c = 4,000 psi at 28 days
Reinforcing steel	ASTM A-615, Grade 60
Structural steel	ASTM A-36

REFERENCES

1. "Reactor Coolant System Asymmetric Loads," Final Report, prepared by Combustion Engineering, Inc, June 30, 1980.

5.10 SYSTEMS AND COMPONENTS

5.10.1 DESIGN CRITERIA FOR CP CO DESIGN CLASS 1 SYSTEMS AND COMPONENTS

5.10.1.1 CP Co Design Class 1 Piping

During the implementation of IE Bulletin 79-14 work, CP Co Design Class 1 piping, except the main primary coolant piping, was designed using the criteria in this paragraph. The piping was designed to the USAS B31.1.0 (1967) Power Piping Code with three modifications. Firstly, the primary pipe stresses incorporated a 0.75i factor. This factor was introduced into the ANSI B31.1 Power Piping Code (1973) and into the ASME Boiler and Pressure Vessel Code, Section III, Subsection NC (1974). Secondly, the criteria included a faulted allowable of $2.4S_h$. The $2.4S_h$ allowable was introduced into the ASME Boiler and Pressure Vessel Code, Section III, Subsection NC, with the 1976 winter addenda. Thirdly, an allowable stress of $1.1S_y$ was permitted for load combination number 5 (see Subsection 5.10.1.3).

In 1990, for integral welded connections, the use of ASME Boiler and Pressure Vessel Code, Section III, Code Cases N-318-4 and N-392-1 for the design evaluation of rectangular and circular cross sections, respectively, was adopted. These code cases and their revisions are among the suitable analysis tools for integral welded attachments.

In 1991, the use of ASME Boiler and Pressure Vessel Code, Section III, Code Case N-316, Alternate Rules for Fillet Weld Dimensions for Socket Welded Fittings, Section III, Division 1, Class 1, 2, and 3 was adopted.

As a result of discussions between CP Co and the NRC, the design criteria for CP Co Design Class 1 piping were revised in 1992. For new and existing CP Co Design Class 1 piping (except the main primary coolant piping), the code of record was changed to ANSI B31.1 (1973) Power Piping Code with the Summer (1973) Addenda. The load combinations and allowable stress formulas are as noted below.

Load Combination		Allowable Stress
1.	DP	S_h
2.	P + DW	S_h
3.	TE	S_A
4.	P + DW + SL	$1.2S_h$
5.	P + DW + 2SL	$2.4\ S_h$ or $1.1S_y$ (See Note)
6.	P + DW + F	$1.2S_h$

Where:

DP = design pressure hoop stress

TE = thermal expansion stress (seismic anchor movement (SAM) stresses shall be added absolutely to thermal expansion stresses where applicable)

P = longitudinal pressure stress

DW = deadweight stress

SL = OBE stresses resulting from a ground surface acceleration of 0.1 g

2SL = SSE stresses resulting from a ground surface acceleration of 0.2 g

S_h = allowable code stress at appropriate temperature

S_y = minimum yield strength at appropriate temperature

S_A = allowable code stress range

F = stresses resulting from thrust force of main steam relief valves or pressurizer relief valves (under certain circumstances, these loads can be placed in a 2 SL (faulted) load combination)

NOTE: 2.4 S_h is to be used when Code Case N-411 analysis is performed. 1.1S_y is to be used when the original response spectra are used for analysis. See Section 5.10.3.1.3.

5.10.1.2 CP Co Design Class 1 Pipe Supports

CP Co Design Class 1 pipe supports were designed using the criteria of the USAS B31.1.0 (1967) Code and the AISC Structural Steel Specification, Sixth Edition, 1963, as follows:

1. SL Case

Component	Load Combination	Allowable Stress
Structural	Greater of DW + TE + SL or DW + SL	1.1S
Catalog(a)	Greater of DW + TE + SL or DW + SL	$0.2S_u$

2. 2SL Case

Component	Load Combination	Allowable Stress
Structural	Greater of DW + TE + 2SL or DW + 2SL	Lesser of $1.1S_y$ or 1.65S
Catalog(a)	Greater of DW + TE + 2SL or DW + 2SL	$0.4S_u$

(a) Or vendor specified load rating {load capacity data sheet (LCD)}

Where:

DW = load due to pipe deadweight

TE = load due to pipe thermal expansion

SL = OBE loads resulting from a ground surface acceleration of 0.1 g

2SL = SSE loads resulting from a ground surface acceleration of 0.2 g

S = AISC allowable stress

S_u = minimum ultimate stress

S_y = minimum yield stress

CP Co Design Class 1 pipe supports were requalified to the above criteria as a result of the issuance of IE Bulletins 79-02 and 79-14, except the AISC Structural Steel Specification, Seventh Edition, 1970, was used. See Subsection 5.10.3.1.3 for a discussion of the bulletins.

Beginning in 1989, CP Co Design Class 1 pipe supports were designed to the following load combinations and allowables using the AISC Structural Steel Specification, Eighth Edition, 1980.

Load Condition	Component	Load Combinations	Allowables
Normal	Structural	DW + TE + Friction	S
	Catalog	DW + TE + Friction	Vendor Rated Capacity
Upset(b) (SL Case)	Structural	Greater of DW + TE ± SL or DW ± SL or DW + TE + F or DW + F	1.1S
	Catalog	Greater of DW + TE ± SL or DW ± SL or DW + TE + F or DW + F	0.2 Su or LCD
Faulted(b) (2SL Case)	Structural	Greater of DW + TE ± 2SL or DW ± 2SL	Lesser of 1.1Sy or 1.65S
	Catalog	Greater of DW + TE ± 2SL or DW ± 2SL	0.4 Su or LCD
Test	Structural	DW + Water	Lesser of 0.8Sy or 1.3S
	Catalog	DW + Water	1.5 for General and 1.3 for Compr Strut of Vendor Rated Normal Load Capacity
	Supplementary Steel	DW + Water	S

Where:

DW = load due to pipe deadweight including insulation

TE = load due to pipe thermal expansion

SL = OBE loads resulting from a ground surface acceleration of 0.1 g

2SL = SSE loads resulting from a ground surface acceleration of 0.2 g

S = AISC allowables stress (AISC Manual of Steel Construction, 8th Edition)

Su = minimum ultimate stress or strength

Sy = minimum yield stress

F = hydraulic transient forces resulting from main steam relief valves or pressurizer relief valves. Under certain circumstances, these loads can be placed in 2SL (faulted) load combinations.

(b) SL and 2SL load combinations shall include seismic anchor movements (SAM) where applicable.

Beginning in September 1992, all supports are either designed or evaluated in accordance with the above criteria except single angles. Single angles may be evaluated in accordance with the AISC Manual of Steel Construction, Ninth Edition's "Specification for Allowable Stress Design of Single Angle Members."

5.10.1.3 Other CP Co Design Class 1 Systems and Components

Other CPCo Design Class 1 systems and components were designed using the criteria below. Exceptions include the containment steel penetrations which were designed in accordance with the criteria set forth in Subsections 5.8.6.3.2, the reactor internals which were designed in accordance with the criteria discussed in Subsection 3.2.3 and the vessels and the main loop piping of the primary coolant system which were designed in accordance with the criteria of Chapter 4.

Load Combination		Allowable Stress
1.	MOL + PTT + SL	Applicable Code Allowable Stress (See Table 5.2-3)
2.	MOL + MTT + SL	Minimum Yield Stress at Appropriate Temperature
3.	MOL + MTT + 2SL	110% Minimum Yield Stress at Appropriate Temperature(a)

Where:

MOL = maximum normal operating loads including design pressure, design temperature plus piping and/or support reactions

PTT = normal planned thermal transients associated with expected Plant normal operation transients such as start-up, shutdown and load swings

MTT = maximum thermal transients in the systems functioning during Plant emergency conditions such as full-power reactor trip, turbine generator trip, loss of auxiliary power and the DBA

SL = OBE loads resulting from a ground surface acceleration of 0.1 g

2SL = SSE loads resulting from a ground surface acceleration of 0.2 g

(a) The maximum stress level of 10% over the minimum yield stress at the appropriate temperature for load combination number 3 was used to limit the strain to a value corresponding to the stress level reached by extrapolation beyond the material proportional limit.

5.10.1.4 Interim Operability Criteria

The Interim Operability Criteria provide the requirements for determining the operability of systems that are found to contain safety related piping and associated pipe supports with stresses that exceed the allowables described in Sections 5.10.1.1 and 5.10.1.2. When stresses are shown to satisfy the Interim Operability Criteria, continued system operation is permitted for an interim period until modifications are performed which return the system to within the allowables. These Criteria are described in Reference 17 and were formally approved by the NRC in Reference 18.

5.10.2 DESIGN CRITERIA FOR CP CO DESIGN CLASS 2 AND CLASS 3 SYSTEMS AND COMPONENTS

5.10.2.1 CP Co Design Class 2

CP Co Design Class 2 piping systems and components are designed to the same requirements as for CP Co Design Class 1 piping (see Subsection 5.10.1.1) except that the load combination associated with the OBE (SL) is not considered.

5.10.2.2 CP Co Design Class 3

CP Co Design Class 3 systems and components were originally designed to the requirements of USAS B31.1.0-1967. For new and existing CPCo Design Class 3 piping, design and reanalysis shall be done per ANSI B31.1 (1973) Power Piping Code with Summer (1973) Addenda. This is the same Code as used for CPCo Design Class 1 piping (see Section 5.10.1.1). Non safety related piping should be designed to the requirements of this paragraph. Where applicable, wind loads conform to Section 2308 of the Uniform Building Code. Seismic loads need not be considered in CP Co Design Class 3 Design.

5.10.3 ANCHORAGE MODIFICATIONS FOR SAFETY-RELATED SYSTEMS AND COMPONENTS

5.10.3.1 Piping Systems

5.10.3.1.1 1974 Review

During May 1974, a pipe restraint on the suction line of the low-pressure safety injection pump at the Palisades Plant was pulled loose from its mounting. Following this finding, a physical survey was performed on the as-built supports for certain CP Co Design Class 1 systems, or portions thereof, which are required for safe shutdown during normal or SSE conditions (see Table 5.10-1). The intent of this survey was to verify support compliance with the initial design criteria in the Palisades FSAR.

Due to the constraints of time, support modifications were made, without the benefit of additional analyses, whenever compliance with the design criteria was not readily apparent. A total of 188 supports was added, replaced, modified, removed, relocated or maintained based on this review. The majority of the support modifications, 167 out of 188, were associated with lines 4 inches and smaller. Details of support modifications are provided in Reference 1.

Subsequent to the completion of these work items, further analyses were performed. These analyses showed that 74 of the 188 support modifications were not required to meet original Plant design criteria.

5.10.3.1.2 1979 Reanalysis

In 1978 it was determined that an error existed in an Engineering Data Systems, Inc (EDS) computer code which was used in the EDS analysis of certain piping systems at the Palisades Plant. The original design packages for 39 safety-related systems analyzed by this code were reviewed in 1979. It was determined that 16 systems either were affected by the program error or that they lacked proper documentation. Ten of these, which had been subsequently reanalyzed, met, without modification, the requirements of the American National Standards Institute, Code for Pressure Piping, Power Piping, ANSI B31.1-1973. The remaining six systems were reanalyzed. Only the seismic analyses were of concern since EDS did not do the thermal flexibility and deadweight analyses. The seismic analysis employed was based upon the seismic response spectra used for original design. All loads and stresses were calculated from a linear elastic analysis based on normal mode theory.

The reanalysis identified eight pipe supports which realized increased seismic loads. The supports were modified to ensure that the requirements of FSAR Appendix A (1980 FSAR as amended and revised) were met. Details regarding the affected pipe supports and the modifications are provided in Reference 2.

5.10.3.1.3 Revision of Seismic Piping Criteria - ASME Section III, Code Case N-411

In 1986, analysis of a segment of small bore piping on the high-pressure Safety Injection System revealed piping stresses well above FSAR allowables. The condition was reported to the NRC in Licensing Event Report (LER) 86-022, Revision 2. The high calculated seismic stresses existed because the fundamental frequency of the segment was on the peak of a very steep response spectra for 0.5% damping. The segment of concern had several heavy values, was at high pressure, was of short length and had no hangers. Resolution of the calculated stress condition could have been achieved by introducing a support which would have reduced calculated seismic loads to acceptable levels yet would have made thermal expansion stresses very high.

As an alternative to modification, the system was analyzed to the seismic input of NUREG/CR-1833 which had been used by CP Co and its consultants to evaluate seismic systems, structures and components as part of the Systematic Evaluation Program Topic III-6. Employing the 3% damping response spectra curve contained in NUREG/CR-1833 and the USNRC Regulatory Guides recommended there, the system was only slightly above the FSAR allowable for the loading case of axial pressure plus deadweight bending plus seismic inertial bending. The conclusion from the alternate assessment was that the use of NUREG/CR-1833 along with ASME B&PV Section III, Code Case N-411 for damping values would result in the determination that the piping segment being evaluated would be shown to meet FSAR requirements.

The relatively flat, broad-band spectra from NUREG/CR-1833 appeared to be more realistic spectra for design. They were derived from a synthesis of many time histories and incorporated the effects of soil-structure interaction. Those spectra also included a bounding of response results from separate analysis reflecting the variation of soil spring moduli. The spectra essentially included refinements in seismic design and analysis techniques as developed since the Plant was designed.

In Reference 11, CP Co advised the NRC that it intended to meet the requirements of USNRC Regulatory Guide 1.84, Rev 24 for implementing Code Case N-411. As part of meeting these requirements, CP Co committed to employ the response spectra generated by NRC consultants in NUREG/CR-1833. In Reference 12, the NRC commented on Reference 11 to the effect that the CP Co approach was reasonable but that CP Co must incorporate the effects of higher modes (greater than 33 Hz) explicitly in the seismic analysis.

CP Co will use the seismic response spectra of NUREG/CR-1833 with the Code Case N-411 damping values to draw a single response spectra plot for a given building elevation and loading direction. Those spectra are shown in Reference 16 and are valid for piping only. They will be used for all new systems and may be employed in the evaluation of existing systems as an alternate analysis/design criteria.

5.10.3.1.4 Inspection and Enforcement Bulletins

History

Because of the identification, by a utility company, of safety-related support failures for supports utilizing concrete expansion anchor bolts (CEBs), the Nuclear Regulatory Commission (NRC) issued Inspection and Enforcement Bulletin (IEB) 79-02. This bulletin required the inspection, testing, evaluation and modification, if necessary, of pipe supports on large safety-related piping systems (2-1/2 inches and greater), if they utilized CEBs. A less extensive program was required for small safety related piping systems (less than 2-1/2 inches) if these systems were originally analyzed by the chart method. During the same period, the NRC issued IEB 79-14 with the intent of ensuring that the existing analyses of large safety-related piping systems for all plants reflected the as-built condition of these systems.

Program for Concrete Expansion Bolts

The IEB 79-02 program performed at the Palisades Plant contained the following elements:

1. Inspection of CEBs to verify proper installation

2. Load testing of CEBs

3. Site-specific testing of CEBs to establish torque-tension relationships

4. Evaluation, considering support flexibility, of baseplates or structural steel members with CEBs

5. Modification of baseplates or structural steel members and/or CEBs not satisfying the acceptance criteria

A complete description of the program is given in References 3 and 4.

More than 4,000 CEBs were inspected and tested at the Palisades Plant. Each CEB was inspected to verify adequate thread engagement, CEB size, spacing and distance to a concrete edge. In addition, full expansion of the shell for shell-type CEBs was verified. Each CEB was load tested to twice its allowable tensile value using either a direct pull or applied torque. If the CEB passed the testing and inspection, it was reloaded to a value that ensured a preload greater than the minimum design allowable load. If the CEB did not pass the test, it was replaced by a wedge-type CEB.

All baseplates using CEBs (more than 1,400) were evaluated to account for plate flexibility, bolt stiffness, shear-tension interaction, minimum edge distance and proper CEB spacing. Most of the baseplates were unstiffened and anchored by 4, 6 or 8 CEBs. For these types of baseplates, an analytical method was developed which treated them as a beam on multiple spring supports. In the evaluation, factors of safety of 4 for wedge-type and 5 for shell-type CEBs were used.

<u>Program for Safety-Related Piping and Supports</u>

The IEB 79-14 program performed at the Palisades Plant contained the following elements:

1. Inspection of large and small piping systems. (Table 5.10-2 lists all the Palisades systems that contain safety-related piping.)

2. Creation of as-built piping isometrics and support sketches.

3. Reanalysis of large piping.

4. Reanalysis of large pipe supports.

5. Support modifications as required by the reanalyses. A complete description of the program is given in References 3 and 5.

Approximately 18,100 feet of large safety-related pipe and 1,550 pipe supports were inspected and over 400 pipe support modifications were made. No support modifications were made for small piping systems.

5.10.3.2 Masonry Walls

5.10.3.2.1 History

In the fall of 1979, a utility, which was conducting inspections pursuant to IE Bulletins 79-02 and 79-14, informed the Nuclear Regulatory Commission (NRC) of a problem with the structural integrity of concrete masonry walls with Seismic Category I piping attached to them. Specifically, some walls did not have adequate structural strength to sustain the required piping system support reactions. After further investigations, the NRC concluded that the problem had sufficient generic implications to warrant issuance of an IE Bulletin. In May 1980, the NRC issued IEB 80-11 (see Reference 6) which required identification and reevaluation of all masonry walls which are in proximity to or have attachments from safety-related systems. Masonry walls meeting these criteria are defined to be safety-related masonry walls.

5.10.3.2.2 Identification

The program to identify these walls included a review of design drawings, the plant safety classification and a walk down of the plant. A total of 64 walls were identified to be within the scope of IEB 80-11.

5.10.3.2.3 Reevaluation

Masonry walls were treated as plates for reevaluating forces and moments in the walls. Details regarding loads and load combinations, design allowables, analytical methods and alternative acceptance criteria are given in Reference 7.

Twenty-two walls passed the reevaluation. Safety-related components were relocated away from 6 walls and 4 walls were removed due to new construction. The remaining 32 walls required structural modifications.

5.10.3.2.4 Modifications

Boundary supports were modified on twelve masonry walls. The addition of boundary supports strengthened a wall which would otherwise fail under excessive bending stresses. A typical boundary condition modification was to use a number of clip angle connections between the masonry wall and the adjacent concrete wall. The clip angles were connected to the masonry wall by through bolts. Concrete expansion anchors were used to connect the clip angles to the adjacent concrete wall.

Beam braces were used to modify four walls. The beam brace strengthened a block wall that would otherwise fail under excessive bending and shear stresses. A typical beam brace modification was to attach a square steel tube to the masonry wall by a number of through bolts. The steel tube was connected to the floor and to the ceiling by clip angles and concrete expansion anchors.

Twelve walls were modified both by boundary supports and beam braces. Sample calculations for both boundary supports and beam brace modifications are provided in Reference 8.

One wall was protected by a structural steel missile barrier. Two walls were replaced by reinforced concrete.

The NRC subsequently indicated in their Safety Evaluation (see Reference 13) that Consumers Power's submittals in response to IEB 80-11 satisfactorily addressed all issues raised in the bulletin.

5.10.3.3 Electrical Equipment

5.10.3.3.1 History

During the fall of 1979, seismic design evaluations being performed in connection with the Nuclear Regulatory Commission's (NRC) Systematic Evaluation Program (SEP) indicated a potential safety deficiency related to the anchorage and support of safety-related electrical equipment at several SEP plants. In general, it was observed that there was a lack of engineered anchorages and supports for safety-related electrical equipment. It was also observed that some nonsafety-related ancillary items (dollies, gas bottles, etc) were located such that, during an earthquake, they might be dislodged and impact and damage safety-related equipment during an earthquake. Consequently in January 1980, the NRC required all SEP plants to evaluate the capability of the anchorages and supports for both safety-related electrical equipment and ancillary items to withstand a seismic disturbance. Additionally, remedial measures were to be undertaken, as necessary, to ensure that the anchorages and supports had this capability. In July 1980, the scope of the investigation was extended to include supports for internally attached components and the seismic design of cable trays.

5.10.3.3.2 Identification

A walk down of all safety-related electrical equipment was conducted at the Palisades Plant in March 1980. A list of safety-related electrical equipment and other nonsafety items located in the vicinity of this equipment was prepared (see Reference 9). Anchorage methods were included in this tabulation. Based on this inspection 15 safety-related pieces of electrical equipment were identified as unanchored.

5.10.3.3.3 Evaluation and Modifications

Existing anchorages for safety-related equipment were analyzed for safe shutdown earthquake (SSE) loads and modifications were made for anchorages that did not satisfy the acceptance criteria. Anchorages for previously unattached safety-related equipment were designed to withstand SSE loads.

The adequacy of anchorages for nonsafety-related items was determined on the basis of visual observations and supported, whenever possible, by calculations for similar safety-related items. Items that were determined to be unanchored or inadequately anchored were either tied down by an acceptable anchorage scheme or moved out of sensitive areas.

Details regarding loads, load combinations, analytical methods, acceptance criteria, testing and modifications are given in References 9, 10 and 15.

5.10.4 QUALITY CONTROL

5.10.4.1 Shop Welding

Welding performed by fabricators was in accordance with applicable specifications, codes and standards. When required by specifications, weld procedures were approved by Bechtel metallurgical engineers. When required by applicable codes and standards, the subcontractor qualified his weld procedure before beginning production work. Where welders employed by the fabricators were to make welds which required qualification, they were tested and qualified under the supervision of the subcontractor. The Bechtel inspector verified that welding was performed in accordance with the proper procedures and that the procedure and welder qualifications had been properly executed.

In assuring that welds were made properly, the Bechtel inspector checked the following items as a minimum: base material, weld electrodes, weld preparation and fit-up; current, preheat and interpass temperatures; travel, cleaning and appearance of individual weld beads. If postweld heat treatment was specified, he checked the number and location of thermocouples and inspected the temperature charts for heating rate, holding time and temperature and cooling rates.

During and after welding, appropriate code inspections were made for dimensional compliance, slag, cracks, porosity, incomplete penetration and lack of fusion.

Where radiography, magnetic particle or liquid penetrant inspection was specified, the inspector determined that the proper technique was being followed and the results properly interpreted. In the case of radiography, the inspector reviewed completed film for quality and interpretation of defects. Film having unacceptable quality of questionable indications of defects was cause for reradiographing the affected areas.

Where weld defects were in excess of specified standards, the inspector verified that the affected areas were removed, rewelded and reinspected until the specified standards were met.

5.10.4.2 Field Welding

All field welding, whether produced by Bechtel employees or employees of subcontractors, was under the surveillance of Bechtel field welding inspectors. The inspectors were qualified to judge all the types of welds used on the job as well as being experienced in the fundamentals, techniques and application of the nondestructive inspection methods used; eg, visual, magnetic particle, liquid penetrant and radiography.

All welding was performed in accordance with approved procedures that were qualified in accordance with the governing codes and standards, via ASME Section IX and AWS D1.1. In case of welds produced by Bechtel Construction personnel, etc, these procedures were prepared and qualified by the Bechtel Metallurgical and Quality Control Services Department and were identified in the applicable Bechtel design and fabrication drawings and specifications. Welders were qualified by tests as required by the applicable Bechtel Welder Performance Specifications. These Bechtel specifications encompass the requirements of the governing codes and standards. No welder was permitted to perform production welding until he had passed the necessary tests under the direction of the field welding engineer and had the test record on file at the job site.

Welding performed by subcontractors was in accordance with applicable job specifications, codes and standards. When required by specifications, weld procedures were approved by Bechtel metallurgical engineers.

When required by applicable codes and standards, the subcontractor qualified his weld procedure before beginning production work. Where welders employed by the subcontractors were required to make welds which required qualification, they were tested and qualified under the supervision of the subcontractor. The Bechtel welding inspector verified that welding was performed in accordance with the proper procedures and that the procedures and welder qualifications had been properly executed.

For the 1990/1991 steam generator replacement activities, visual inspection and acceptance of structural welds was performed in accordance with NCIG-01 (Reference 14). This welding inspection criteria is applicable for uncoated structural weldments made per AWS D1.1, and is not applicable to inservice inspections that are required by Section XI of the ASME Code.

5.10.4.3 Inspection of Piping

All piping systems and components were designed, fabricated, tested and inspected to the requirements of all applicable portions of the following standards and codes as minimum requirements:

1. ASME Boiler and Pressure Vessel Code, Sections I, II, III and IX
2. ASA Code for Pressure Piping (ASA B31.1 and ASA B16.5) including applicable Nuclear Code Cases
3. ASTM Standards
4. PFI Standards

In addition to the minimum requirements of the ASA Code, appropriate nondestructive testing was performed to assure quality control on all critical systems such as main steam, high-pressure feedwater and the engineered safeguards systems as identified in the Bechtel mechanical drawings and specifications. Welding procedures and qualifications were approved by Bechtel metallurgical engineers prior to production welding.

Pressure retaining materials were inspected by Bechtel inspectors in the manufacturers' and fabricators' shops as necessary to assure that heat numbers and certification reports corresponded. Shop testing and inspection also assured that materials were free from injurious defects as well as full compliance with the requirements of codes, standards and specifications.

In excess of the above-mentioned code requirements, piping in normally nonradioactive service was subjected to spot checks by nondestructive testing methods (radiograph, liquid penetrant or magnetic particle, as appropriate) in both the shop and the field. Pipe butt welds in systems for radioactive service rated higher than 50 psig and 212°F received 100% radiographic inspection and those in lower rated systems received 10% radiographic inspection. In addition, pipe welds in all radioactive service received spot checks by other techniques in the shop and field, as required by applicable codes. All shop and field radiographs were reviewed in total by Bechtel's shop inspectors or field welding inspectors. Selected weld radiographs were also reviewed by both Consumers Power Company and Combustion Engineering, Incorporated.

Where seam-welded piping was required, ASTM A-358 (stainless steel) or A-155 (low alloy or carbon steel), CP Co Design Class 1 pipe requiring 100% weld radiography was specified in lieu of CP Co Design Class 2 which was optional according to code. This practice was followed for both radioactive and nonradioactive services. The root pass for butt-welding of critical piping was by the TIG process to assure high quality weld joints.

Shop tests were applied to cast valve bodies and fittings in accordance with codes and standards; these tests were witnessed to assure compliance with codes and standards.

Prior to cutting and fabrication, each segment of nuclear piping was marked with the appropriate material specification and spool number. This is to assure positive material and spool identification.

In addition to the material hydrostatic test performed in the shop, all critical systems' piping were hydrostatically tested after erection.

In summary, all shop pipe fabricating and testing procedures, test results and materials were subject to the approval of a Bechtel inspector to assure shop quality control. Field welding, weld inspection and material usage were under the control and approval of Bechtel's field welding inspector to assure fieldwork quality control.

5.10.4.4 Field Inspection of Mechanical Components, Electrical Components and Instrumentation

Upon receipt of equipment in the field, inspection was made for conformance to applicable specifications, codes and drawings. Checks were made for completeness of shop inspection and material certifications if required by specification and/or damage in transit. A record was made of this inspection on a special form. If deficiencies were present, they were noted on the forms and sent to the Job Engineer for correction. Copies of the forms were filed with the Quality Assurance Engineer and the Project Engineer. If the item was not immediately set in its final position, a notation was made on the form regarding the storage of the item and the protection provided.

When the item was set or installed, inspection was employed to confirm that the item was properly set and that piping, electrical and instrumentation items were properly connected. All deficiencies were noted on the applicable installation inspection records and processed similar to deficiencies noted during receiving of equipment.

A final inspection was made prior to start-up testing to verify that the item had been hydrostatically tested, cleaned, lubricated and calibrated as required by specifications and job procedures. This was reported on a special form on which deficiencies were reported for receiving an installation inspection.

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16. Palisades Specification M-195(Q), "Requirements for the Design and Analysis of Palisades Plant Safety Related Piping and Instrument Tubing."

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