SECTION 12

PLANT STRUCTURES AND SHIELDING

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SECTION 12 PLANT STRUCTURES AND SHIELDING

12.1 SCOPE

The special design procedures contained in this section apply to all structures (including the Reactor Building internal structures), systems (including instruments and controls), and all components. 98124

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12.2 PLANT PRINCIPAL STRUCTURES AND EQUIPMENT

12.2.1 Design Basis

12.2.1.1 Classification of Structures and Components

All structures (including the Reactor Building), systems (including instruments and controls), and components are classified as Design Class I, II or III according to their function and importance in relation to the safe operation of the reactor, with emphasis on the degree of integrity required to protect the public. These are listed in Table 12.2-1. The table provides overall design classification of structures and components. For detailed design classifications and boundaries separating different design classes within this overall classification refer to the controlled drawings.

The Turbine Building, Administration Building, Auxiliary Building and Shield Building structures are constructed as a contiguous complex. In general, these structures are identified as either Design Class I or Design Class III by placing emphasis on the predominant use of the structure in its relation to the safe operation of the reactor.

In some instances there may be more than one classification applicable within a building or structure. This situation will be treated as a mixed classification.

The definition of the Nuclear Safety Design Classifications is given in the following paragraphs:

a. Design Class I

   Those structures and components including instruments and controls whose failure might cause or increase the severity of a loss-of-coolant accident or result in an uncontrolled release of substantial[1] amounts of radioactivity, and those structures and components vital to safe shutdown and isolation of the reactor.

b. Design Class I*

   Some items in Table 12.2-1 are designated as Design Class I* indicating that these items have been originally designed or have been subsequently analyzed or tested to Design Class I, Design Basis Earthquake loading (dynamic) only, and that these items are treated as Design Class III items in all other respects. 01491754

[1] A substantial amount of radioactivity is defined as that amount of radioactive material which would produce radiation levels at the site boundary in excess of 1.0% of 10CFR100 limits.

c. Design Class II

Those structures and components which are important to reactor operation[2] but not essential to safe shutdown and isolation of the reactor and whose failure could not result in the release of substantial amounts of radioactivity.

The control rod drive and cavity cooling systems are required for continued power operation although failure of either of these systems in no way jeopardizes plant safety. Failure of control rod drive cooling could result in premature failure of the coils, which would result in a dropped rod and subsequent power reduction. The plant operator would immediately be aware of the event. Ability to shut down the reactor would in no way be compromised. Likewise, failure of the cavity cooling system could affect the performance of the neutron instrumentation, but these systems are specifically designed to fail upscale for this condition. An upscale failure would result in a reactor trip. This failure mode is therefore in the safe direction.

Although the reactor control rod drive cooling system and cavity cooling system provide cooling for design Class I components, failure of these cooling systems would not jeopardize the Class I function of the systems they cool, nor is their operation required to effect a safe reactor shutdown. Since these systems are required for reactor operation, they are designated design Class II.

d. Class III

Those structures and components which are not directly related to reactor operation or containment.

e. Design Class III*

Some items in Table 12.2-1 are designated as Design Class III* indicating that the items are Class III by definition, however, these have been designed to Design Class II seismic loadings.

[2] Reactor operation is defined as the condition where the reactor is producing only that power required to maintain the Reactor Coolant System (RCS) at normal operating pressure and temperature.

For example, the waste holdup tank is not required to effect safe shutdown or isolation of the reactor. Failure of this tank as discussed in Section 9.2.3 would not result in any release to the environment because it is entirely housed in a concrete vault in the Auxiliary Building within the Category I boundary. The vault and building are designed to Class I design requirements and the vault is sized to contain all the contents of the tank in case of failure. Floor drains in the vault are provided with an isolation valve. The waste holdup tank is protected from tornadoes, missiles, pipe whip and steam and water jets. Failure of the tank would not jeopardize the function of any Class I equipment. Since the waste holdup tank could be required for low power operation of the reactor, it is designated as Design Class III* and has been designed to Design Class II seismic loadings.

f. Mixed Classification

This classification includes structures that are combinations of various Design Class I, II or III structures. Mixed classifications apply only to structures and not to any systems and/or components. The design criteria for mixed classification are detailed in Section 12.2.1.4.

a. Design Class I - Part Design Class III

The Spent Fuel Pool and its enclosure are classified as Class I structures. The Auxiliary Building Structure above the Spent Fuel Pool enclosure is classified as a Design Class III* structure.

b. Design Class III - Part Design Class I

Although the turbine plant and auxiliaries are generally Design Class III, the turbine building structure is designed as a Design Class III* structure in accordance with Northern States Power Company's normal practice for the design of steam-electric generating stations. This incorporates the requirements of the Uniform Building Code Earthquake loads, which in this case considers earthquake loads applicable to Zone 1 areas.

The major portion of the turbine plant and its associated systems are designated as Design Class III because failure of these systems would not result in nuclear accidents or releases. The plant design has incorporated adequate offsite and onsite emergency power supplies to affect orderly shutdown of the plant under any design condition. The main turbine generator set is in no way required to effect a safe shutdown of the plant, nor is it essential to reactor operation.

These areas in the Turbine Building that are classified as Design Class I house the following equipment:

Ground Floor

1. Unit 1 Emergency diesel generators
2. Batteries
3. Air compressors
4. Auxiliary feedwater pumps
5. Cooling water pipes

The main feedwater pumps which are Class III are located in Class III* portions of the Turbine Building.

Mezzanine Floor

1. Unit 1 4160V and 480V safeguards switchgear

The above Class I designation applies to the walls, floors, ceilings, structural support and foundations of structures that isolate, support or are associated with the protection of Class I equipment.

The quality classification for all Design Class I and Design Class I* components listed in Table 12.2-1 are Type 1 and Type 3, respectively.

12.2.1.2 Design Codes

The design and construction of this plant has been in accordance with the following codes as applicable:

a. American Concrete Institute Codes; ACI 318-63, ACI 301-66, ACI 349-85 and other sections of the ACI Codes as applicable.

b. American Institute of Steel Construction “Specification for the Design, Fabrication and Erection of Structural Steel Buildings,” 1963 Edition. (Modifications to the plant since original construction have used more recent editions.)

c. American Welding Society Code D 1.0 “Standards for Arc and Gas Welding in Building Construction.”

d. International Conference of Building Officials “Uniform Building Code,” 1967 Edition.

e. Atomic Energy Commission publications TID 7024 “Nuclear Reactors and Earthquakes.”

f. American Society of Mechanical Engineers “Boiler and Pressure Vessel Code 1968” with the 1968 Winter Addendum. (Modifications to the plant since original construction have used more recent editions.)

g. Piping Code, USAS B 31.1.0 - 1967 with applicable N - code cases to ASA B31.1-1955. Note that ASME B31.1-1989 has been reconciled with the original code to allow its use for fabrication and erection.

h. Local Stresses in Spherical and Cylindrical Shells due to External Loadings, Welding Research Council Bulletin No. 107

i. Industrial Commission of Minnesota “Regulations Relating to Industrial Safety.”

j. Design codes for the D5/D6 Diesel Generator Building, and support systems are generally more recent versions than the above. Design Codes for D5/D6 addition are detailed in the SBO/ESU Design Report, Reference 43.

k. American Society of Mechanical Engineering “Boiler and Pressure Vessel Code 1998” through 2000 Addenda.

The system quality types are defined in Section 1.5, Criterion 1.

The applicable codes and NDT requirements for vessels, pipes, pumps, valves, heat exchangers and tanks for those in the Reactor Coolant Pressure boundary and those in the balance of plant piping systems are given in Table 12.2-2 and Table 12.2-3 respectively.

Table 12.2-2 which resulted from the October 14, 1969 DRL meeting was established as the minimum acceptable basis for the NDT requirements outlined in the specifications issued for reactor coolant pressure boundary piping, valves, pumps, heat exchangers and tanks.

Table 12.2-3 was established as the minimum acceptable basis for the NDT requirements outlined in the specifications issued for balance of plant piping systems.

Piping system components for balance of plant piping were ordered to incorporate “normal” non-destructive testing that is a part of the ASTM Spec to which the components were ordered. For instance, all piping was supplied with applicable ASTM material spec: ASTM A155, A358, A106, A312. Also all welded fittings ordered to ASTM A234 or A403 had radiographic examination (RT) of the welds performed and accepted per ASME VIII, Para. UW-51. All cast fittings were shop hydro-tested per their applicable material specification, ASTM-A216, A217, or A351. All shop and field welds were visually inspected and reported by the piping installer’s QC personnel.

In accordance with Northern States Power Company's normal practices on power plant component designs, additional NDT requirements were imposed in special cases.

Since the majority of the components for Unit 2 were purchased at the same time as Unit 1, the codes and NDT requirements for Unit 2 are the same as listed in Tables 12.2-2 and 12.2-3.

12.2.1.3 Design Loadings

All structures and components in this plant are designed to withstand various kinds and combinations of loads. The different kinds of loads treated in the design are described in the subsequent paragraphs.

The load combinations for the original plant design, including modifications and additions thereto, are given in Section 12.2.1.4 for structures and in Section 12.2.1.5 for components. The load combinations for the D5/D6 addition structures and components are given in Reference 43.

The non-safety related structures, systems and components (SSC), whose failure could result in damage to safety related SSCs, are designed to the criteria known as "Seismic Two over One" (II/I). This Seismic II/I criteria is as follows:

- Piping: Regardless of building, non-safety related piping between Design Class I piping and the first structural or equipment anchor are required to be designed to withstand a Design Basis Earthquake.
- D5/D6 Building: All non-safety related SSCs are required to be designed to withstand a Design Basis Earthquake or be located where they cannot collapse on and damage safety related equipment.
- Non-safety related SSCs installed in all buildings, since November 1995, are required to be designed to withstand a Design Basis Earthquake or be located where they cannot collapse on and damage safety related equipment.

12.2.1.3.1 Environmental Loads

These consist of wind and snow loads.

Snow load of 50 lbs per sq ft of horizontal projected area is used in the design of structures and components exposed to snow.

The design wind speed is 100 mph. Wind pressure, shape factors, gust factors and variation of winds with height have all been determined in accordance with the procedures given in the Transactions of the American Society of Civil Engineer's paper ASCE 3269 "Wind Forces on Structures", Volume 126, Part II (1961).

In some cases the design snow load and design wind loads are applied simultaneously with other loads. These are further clarified in Sections 12.2.1.4 and 12.2.1.5.

12.2.1.3.2 Tornado Loads

Tornado loadings used in the design consist of the following:

a. A pressure drop equal to 3 psi. This pressure is assumed to drop from normal atmospheric pressure in 3 seconds.

b. A lateral force caused by a funnel of wind having a peripheral tangential velocity of 300 mph and a forward progression of 60 mph.

c. The design tornado driven missile was assumed equivalent to an airborne 4″ x 12″ x 12′0″ plank travelling end-on at 300 mph, or a 4000 lbs automobile flying through the air at 50 mph and at not more than 25 feet above ground level.

   The plant site was examined for possible sources of other missiles including equipment parts which were evaluated to determine the potentially most damaging missile. The result of this study is reported in Section 12.2.1.4.3.1.4.

   The conclusion of this study was that no other missile was as damaging as the design missiles.

The tornado loading used in design of the D5/D6 Building consist of the following (Ref 43):

a. A lateral force caused by a funnel of wind having a rotational speed of 290 mph and maximum translation speed of 70 mph.

b. A pressure drop of 3.0 psi, the rate of pressure drop being 2.0 psi/sec.

c. The design tornado generated missiles as shown on Table 12.2-43.

12.2.1.3.3 Live Loads

Live loads are defined as equipment loads as specified on the Manufacturer’s drawings and floor loads which are based upon the intended use of the floor. Live loads for the various building elevations are shown on the plant structural drawings. Live loads for D5/D6 addition structures and components are given in Reference 43.

12.2.1.3.4 Dead Loads

Dead loads consist of the weight of structural steel, concrete, dead weight of the component, etc., as computed for each case.

12.2.1.3.5 Seismic Loads

Several different seismic loads are used in the design of this plant.

a. Operational Basis Earthquake (OBE)

   The Operational Basis Earthquake is based upon a maximum horizontal ground acceleration of 0.06g and the response spectra are given on Plate 4.5 in Appendix E.

b. Design Basis Earthquake (DBE)

   The Design Basis Earthquake is based upon a maximum horizontal ground acceleration of 0.12g and the response spectra are given on Plate 4.6 in Appendix E.

   The DBE, as defined above, is synonymous with the Safe Shutdown Earthquake (SSE) as defined in subsection d below for the D5/D6 Building. Note that the original plant (all plant structures except D5/D6 Building) building seismic criteria are described in terms of the OBE and DBE. For the D5/D6 Building, the seismic criteria are described in terms of the OBE and SSE as described in subsection d below.

c. Uniform Building Code Earthquake Loads

   The seismic loads for this category are in accordance with the requirements of the Uniform Building Code. This code specifies the location of the plant site to be in a “Zero” earthquake area. However, for conservatism earthquake loads applicable to Zone 1 areas are used in the design under this category.

d. D5/D6 Building Response Spectra

   Response spectra for the D5/D6 Building design are based upon Regulatory Guide 1.60 spectra for maximum ground acceleration (zero period acceleration) of 0.06 g (Operational Basis Earthquake) and 0.12g (Safe Shutdown Earthquake).

12.2.1.3.6 Design Basis Accident (DBA) Loads

The Design Basis Accident for emergency core cooling system and containment design for this plant is the instantaneous double ended rupture of the cold leg of the Reactor Coolant System (RCS). Based on the exemption provided in Generic Letter 84-04, NRC Safety Evaluation (Reference 47) and implementation of leak-before-break technology, a double ended break of the reactor coolant system primary loop piping is no longer the structural design basis for this plant. Refer to Section 4.6.2.3 for additional discussions.

12.2.1.3.7 Other Loads

In addition to all the above loads listed there are other loads used in the design wherever applicable. Among these are ice loads, jet forces, other pipe rupture loads, construction loads, earth pressure loads, groundwater pressure/design flood loads, etc.

12.2.1.4 General Design Criteria for Structures

12.2.1.4.1 Load Combinations

The load combinations applicable to Design Class I, Design Class I*, Design Class II, Design Class III*, and Class III Structures are given in detail in subsequent paragraphs and also listed in Table 12.2-4.

a. Class I Structures

   Class I Structures are analyzed for each of the following conditions of loading:

   1. Normal Operating Conditions

      The load combination consists of Dead and Live loads together with the Environmental Loads (wind and snow) as specified in Section 12.2.1.3.

   2. Operational Basis Earthquake Conditions

      The load combination consists of Dead, Live and DBA loads together with the greater of the OBE plus Snow or Wind loads.

   3. Design Basis Earthquake Conditions

      The load combination consists of Dead, Live, Snow, DBA loads together with the DBE loads.

   4. Tornado Condition

      The load combination consists of Dead and applicable Live loads together with the design tornado and tornado missile impact loads, if any. These loads are assumed not coincident with DBA or Seismic loads.

b. Class I* Structures

Those structures designated as Class I* are analyzed for each of the following conditions of loading:

1. Normal Operating Conditions

   The load combination consists of Dead and Live loads together with Environmental Loads (wind or snow).

2. Design Basis Earthquake Conditions

   The load combination consists of Dead, Live, Snow and the DBE loads.

c. Class II Structures

Class II Structures are designed for the greater of the following two combinations of loads:

1. Dead, Live and Environmental loads (wind or snow) or

2. Dead, Live and Uniform Building Code Earthquake loads specified in Section 12.2.1.3.5, which for the Prairie Island site is 0.05g.

d. Class III* Structures

Class III* Structures are designed for the greater of the two combinations of loads given above for Class II Structures.

e. Class III Structures

Class III Structures are designed for Dead and Live loads together with Environmental loads (wind or snow).

As a minimum, Class III Structures are designed in accordance with the applicable codes as listed in Section 12.2.1.2. In accordance with the company's normal policy for the design of steam-electric generating stations, certain items of power plant structures in the Class III category by definition are designed according to the requirements of a higher classification.

Essentially all of our structures irrespective of the class designation are designed to a minimum of the Zone I earthquake loadings according to the Uniform Building Code.

12.2.1.4.2 Stress Design Criteria

a. Normal Operation

The allowable stress design criteria that are applied for normal operating conditions are in accordance with the applicable code(s) listed in Section 12.2.1.2. Maximum allowable stresses or comparable structural design criteria are summarized in Tables 12.2-5, 12.2-6, 12.2-7 and 12.2-41 (D5/D6 Building).

b. Design Basis Accident (DBA) and Operational Basis Earthquake (OBE)

The allowable stress design criteria which are applied for the DBA Condition in combination with the OBE are that stresses remain within the allowable limits specified by the applicable codes(s) listed herein, except that allowable stresses are not increased for the earthquake condition as is permitted by some codes. Maximum allowable stresses or comparable structural design criteria are summarized in Tables 12.2-5, 12.2-6, 12.2-7, and 12.2-41 (D5/D6 Building).

c. Safe Shutdown

The design criteria for tornado missiles, the design tornado condition, and also for the DBA in combination with the Design Basis Earthquake are that the reactor can be safely shutdown and that there be no uncontrolled release of radioactivity. To meet these criteria, structures or components are examined for their function in the total system to assure a safe and orderly shutdown.

These criteria as applied to (i) tornado winds, (ii) to the DBA condition in combination with DBE loads, permit some permanent deformation but do not permit loss of structural function. In this sense structural function is defined to mean that structures remain intact and continue to support their normal operating loads after an earthquake and/or DBA but may need repair or replacement for future continued use.

Permanent deformations have been allowed locally in structures resisting tornado missiles and turbine missiles. These structures include slabs, beams, removable steel barriers, walls and steel louvers (in D5/D6 Building walls) which are in a location that could be impacted by the above missiles. Also designed to have some permanent deformation are shields for internally generated missiles and some of the contact points of pipe restraints inside and outside of containment.

Plastic deformation has been used as part of the energy absorbing system because of impracticability of the designing within elastic limits. Failure of the structure in each case is prevented by using design stresses limited to less than yield for the overall structure and limiting the plastic deformation for energy absorption to local areas.

Tornado missiles may result in large local deformations but the criteria does not permit the missile to breach the barrier and jeopardize essential safeguard functions.

12.2.1.4.3 Structural Design Basis

12.2.1.4.3.1 Class I Structures

12.2.1.4.3.1.1 Seismic Design

For dynamic analysis, an equivalent multi-mass mathematical model was constructed to approximate the structural system. The effect of the foundation soils was included in the model by means of equivalent springs. Then the spectral method was used to determine the maximum response of each mass point for each mode, using the Operational Basis Earthquake (Plate 4.5 in Appendix E) and the damping values given in Table 12.2-8 as input. The total response for each point was determined by the root-mean-square method. From this a set of curves were developed which show the variation with height in the structure of the maximum translational accelerations, displacements, shears and moments. All of the above work was performed by John A. Blume & Associates and is reported in detail in a separately submitted Topical Report JAB-PS-02 (Reference 6) (hereinafter referred to as “The Blume Report”). Vertical acceleration equal to two-thirds the horizontal ground acceleration was applied to the structure.

Topical Report JAB-PS-02 was prepared by John A. Blume & Associates using the modal analysis response spectra method of seismic analysis to determine the maximum response of the structures. These response values were used in the design of the Class I (seismic) structures (See Appendix B of Reference 6 for a description of the analytical procedures.)

For the design of components, “Response Acceleration Spectra” (Topical Report JAB-PS-04 - Reference 8) was prepared by John A. Blume & Associates using a ground acceleration time-history developed from the ground response acceleration spectra specified for the Prairie Island Plant as shown in Plates 4.5 and 4.6 of Appendix E.

A description of the seismic analysis for Class I mechanical and electrical equipment is given in Section 12.2.1.5.

The Class I structures were found to be rigid vertically and therefore did not contribute to the amplification of the vertical motion. The vertical motions of the structures were computed by considering the flexibility of the foundation soils. Corresponding vertical response spectra were generated for the design of piping and equipment contained within the buildings. (These spectra are presented in Figures 26 and 28 of Reference 8).

The mathematical model of the structure as shown in Figures 10, 11, and 12 of the Blume Report (Reference 6) is a discrete mass system which represents the structures of the combined Reactor-Auxiliary-Turbine Building. Axi-symmetric modelling techniques were not appropriate in this case and were therefore not used. Analyses of the foundation material were performed using finite element techniques and using elastic half-space equations to determine the base springs shown on the mathematical model of the structure. For the finite element analysis, the foundation was modeled as a network of plane-strain elements and the building foundation as a rigid plate. By applying unit static displacements, the force-deflection relationship of this model was determined. The finite element results were adjusted to account for three-dimensional effects and compared with the half-space results. Based on these comparisons and on judgment and experience, the values for the spring constants presented in Blume Report (Reference 6) were selected for use in the analyses.

Because of the basic theoretical differences, the time history method and the response spectrum method cannot be expected to produce identical results. Thus, an exact check of the analyses cannot be made by comparing the results of the two methods. However, such a comparison should indicate whether results are consistent, that is within the same general range. The following is a comparison of accelerations, in g units, computed by the two methods for selected points in the structure:

	N-S Earthquakes		E-W Earthquakes	
Nodal Point*	Response Spectrum	Time-History	Response Spectrum	Time-History
1, 1A	0.23	0.29	0.23	0.29
4, 4A	0.14	0.17	0.14	0.17
9, 9A	0.18	0.21	0.18	0.21
12, 12A	0.12	0.13	0.12	0.13
20, 20A	0.06	0.08	0.06	0.08
29	0.07	0.10	0.07	0.10
31	0.06	0.08	0.06	0.09

*See Figures 10, 11, 12, JAB-PS-02

It can be seen from the above comparison that the results of the two methods are consistent.

The structural material properties were selected in accordance with reference 4 of the Blume Report (Reference 6). Sufficient conservatism was employed in the analysis so that variation in structural material properties in the order of twenty percent would not cause structural responses significantly greater than those reported in the two John A. Blume reports (References 6 and 8).

The effects on the floor response spectra of expected variations in structure response were accounted for by allowing a ten percent change in the expected location of the peaks of the spectra (which corresponds to about a twenty percent variation in structural material properties). That is, the very sharp and narrow peaks of the spectra as determined directly from the computer analysis were widened to produce the broader smooth spectra as shown in the Blume Report (Reference 8).

a. Operational Basis Earthquake (OBE)

Using the data presented in the Blume Report, stresses were computed in various parts of the plant structures. The stresses resulting from both horizontal and vertical acceleration are combined to obtain the total earthquake stresses. Earthquake stresses are then added linearly and directly to stresses caused by DBA, snow, dead loads, and the appropriate operating loads to obtain the total stresses. The total stresses are reviewed to assure that they are within the maximum stress limits as established in Table 12.2-5 and Table 12.2-6.

The shears, moments, deflections and accelerations were computed independently for each mode excited by the hypothesized ground motions as represented by the response spectra for the operating basis earthquake (Plate 4.5 in Appendix E). Since the absolute addition of all modal responses would be unrealistic and overly severe, the maximum response is obtained by computing the squareroot of the sum of the squares of the individual modal responses.

For the plant buildings, the closely spaced modes are in most cases for different structures and for different directions of these structures. For example, modes 18 thru 21 have natural periods of 0.153 seconds. These four modes represent the north-south and east-west directions of the two containment vessels (Unit No. 1 and Unit No 2). The north-south modes are primarily excited by the north-south earthquake, and the east-west modes by the east-west earthquake. In these cases the responses are independent and are not combined even though the modal periods are numerically equal. In the remainder of the cases the modal responses were all combined by the root mean square method.

Direct stress superposition is not used where materials are designed to be stressed beyond the elastic range. Materials are designed to be stressed beyond the yield point only for missile impact and at the contact points of pipe rupture restraints.

For pipe restraints, small areas of metal or short pieces of small diameter pipe or rod are placed at the impact points to lessen the impact force on the restraint structure. These contact buffers are designed to function elastically after they have deformed plastically to accommodate mismatching of the surfaces.

b. Design Basis Earthquake (DBE)

The forces for the Design Basis Earthquake are taken to be two times the forces as determined by the spectral analyses for the Operational Basis Earthquake. Stresses are combined as before and established that they are within limits as indicated in Table 12.2-5 and Table 12.2-6.

12.2.1.4.3.1.2 Seismic Design of D5/D6 Building

a. General

The building was modeled as a three dimensional finite element lumped mass model. The soil was represented by six spring elements. The computer program STARDYNE was used for modal integration to obtain floor response, acceleration time histories and subsequent generation of the floor response acceleration spectra. The spectral acceleration was used for the design of the floor slabs and the procurement of equipment based on their design classification. The modal responses obtained from the spectral analysis were combined in accordance with the Regulatory Guide 1.92. All three components of seismic accelerations, two horizontal and one vertical component, were considered to act simultaneously. The spatial components were combined by the Square Root of Sum of Squares (SRSS) method.

b. Design Input Response Spectra

Input horizontal and vertical seismic spectra for the seismic design of the D5/D6 Building were based on the Regulatory Guide 1.60 Rev. 1, July 1981 spectra for a maximum ground acceleration (zero period acceleration) of 0.06g for the Operating Basis Earthquake (OBE) and 0.12g for the Safe Shutdown Earthquake (SSE).

c. Time Histories

Acceleration time histories for the three directions of motion (two horizontal and one vertical) were developed using the reverse shock spectrum feature (DYNRE 7) of the STARDYNE MICRO computer program. These time histories were developed independently for the three directions using randomly generated input values by iteratively adjusting the frequency content so as to match the resulting response spectrum with the design response spectrum for a chosen damping value. Comparison of the resulting spectra for various damping values with the corresponding OBE input design response spectra curves was done to ensure acceptability of the final time histories. The requirements of NUREG-0800 SRP 3.7.1 for the enveloping criteria was satisfied. No separate SSE time histories were developed. The OBE response spectra values were doubled to obtain corresponding SSE values.

These time histories were used as input motion for the mathematical model to generate response spectra for all floors.

d. Critical Damping Values

The percentage of critical damping values for the OBE and SSE levels used in the analyses was in accordance with Reg Guide 1.61, October 1973. These values for systems and equipment are given in Table 12.2-42. The values for material and radiation damping within the soil medium were taken as ten percent of critical damping for the translational (three directions), rocking and torsional motions of the building structure. These values are the same for both OBE and SSE levels.

e. Soil-Structure Interaction

Soil-structure interaction was accounted for in the seismic analyses through the use of soil springs representing the six degrees of freedom (three translations and three rotations) of the rigid foundation mat. Soil spring constants were computed using the formulas given in “Design Procedures for Dynamically Loaded Foundations” by R.V. Whitman and F.E. Richart, Journal of the Soil Mechanics and Foundations Division, ASCE, November, 1967. Material properties used in these formulas were taken from Reference 6. The set of six springs were placed at the mass center of the foundation slab. The ground motion time histories were applied directly at the base of the foundation slab.

f. Interaction With Other Structures

The D5/D6 Building foundation is independent from the adjacent Turbine Building and Auxiliary Building foundations using compressible materials between them to achieve a specified spacing.

Similarly, the D5/D6 Building walls and floors are independent from the adjacent Turbine Building and Auxiliary Building walls. The only exception to this is along G-line wall of the Turbine Building between column lines 15 and 16.6 between elevations 695′ and 705′, where the D5/D6 Building stairwell partition wall has some contact with the Turbine Building flood wall.

The above mentioned spacing between structures, and points of contact, have been analyzed to show that they provide adequate seismic independence of all the D5/D6 Building from existing structures.

g. Mathematical Model

The dynamic model consisted of several “sticks” representing the shear walls with nodes at each floor level. Each of these nodes was connected by rigid link, representing the rigid diaphragm action of the floor slab.

The shear areas and moments of inertia of the concrete walls between floors were used to calculate the stiffness characteristics of the elements between nodes of the individual wall “stick”.

The foundation slab was treated as a rigid member for in-plane and out-of-plane motions.

In order to incorporate the effect of the actual floor slab stiffness on the vertical response spectra, vertical spring mass systems were attached at the center of mass of each floor to represent the vertical flexibility of the slab at that elevation. The slab mass along with the equipment loads represented the mass of the spring mass system. This was done by solving the problem twice, once with the lowest flexibility of the slab and second with the highest flexibility. This provided an enveloping spectrum. Composite behavior of the concrete slab and steel beams were considered in the computation of the spring constants, where appropriate.

In order to account for possible variations in mass/stiffness characteristics of the structural members, the horizontal and vertical spectra values were increased by 10%. Also, the E-W and N-S horizontal response spectra are enveloped to give the design horizontal response spectra.

h. Spectral Peak Broadening

The spectral accelerations obtained from the STARDYNE program were smoothed and its frequency at peak acceleration broadened (± 15%) in accordance with the requirements of Regulatory Guide 1.122, Revision 1.

12.2.1.4.3.1.3 Tornado Winds

Structures are analyzed for tornado and missile loads. Tornado loads are combined with dead loads and the appropriate operating loads to obtain total loads. Structural design criteria are as specified in Table 12.2-5 for the original plant and Table 12.2-41 for the D5/D6 Building.

12.2.1.4.3.1.4 Design for Missiles

Many missiles were considered, but only the most damaging missiles were used for design. Missiles were assumed to be generated by explosive injection due to pressure differential, by building component failure, and by aerodynamic lifting, each resulting in an airborne or free falling missile. Only objects with large surface to weight ratios would remain airborne long enough to attain high horizontal velocities. Table 12.2-9 lists the tornado generated missiles considered and the maximum velocities that would be attained by each. Table 12.2-10 lists internally generated missiles. Table 12.2-43 lists the tornado generated missiles considered in the design of the D5/D6 Building.

All tornado generated missiles are assumed to impact end on at 90 degrees to the surface being impacted and all areas of Class I structures exposed to either falling or horizontally flying tornado missiles are investigated. The tornado missiles are assumed to come from stored material, destruction of lower class structures, offsite construction, etc. For the D5/D6 Building, local damage of barriers (penetration, spalling and scabbing effects) and their overall response was investigated in accordance with SRP 3.5.3. Also, where elasto-plastic behavior of steel barriers was relied upon, a maximum ductility ratio of 20 was used.

The only potential missile sources within the D5/D6 building are the diesel generators themselves. The diesel generators are each surrounded by reinforced concrete barriers ranging from 12 to 20 inches thick.

The method of analysis used to evaluate the impact stresses of tornado missiles on Class I structures is described in the following paragraphs of this section. The method of analysis used to evaluate the effect of the turbine missile on the Class I structures is described in Section 12.2.7.

The missile shield structures inside of Containment were analyzed using the conservation of momentum method as described in “Systems Standard Design Criteria, Protection Criteria Against Dynamic Effects Resulting from Pipe Rupture” by Westinghouse Electric Corporation. The overall response and damage prediction of the reactor vessel missile shield was performed by demonstrating that deformations satisfy maximum acceptable ductility ratio of 10 using elasto-plastic response evaluations. The penetration depth was calculated by the Ballistic Research Laboratories Formula for steel and concrete.

Missile protection against tornado for Class I structures consists of reinforced concrete slabs and walls and steel barriers capable of intercepting and stopping the missiles. 01425577

Class I structures inside of containment were protected in two ways:

a. Missile shields were analyzed as a barrier to the missile.

b. Items which could become potential missiles were arranged so that their missile path was directed into a strong wall or slab.

The concept in analysis and design for missiles considered impact to be a plastic collision between the missile and the structure struck by the missile.

The design procedure for tornado missiles consists of equating the energy of the missile to the energy used up in deforming the material (with the plastic deformation of concrete slab and wall reinforcing steel limited to a strain of 5%, concrete beam deformations limited to a ductility factor of six, and steel barrier deformations limited to a maximum ductility factor of 20). 01425577

For pipe restraints, small areas of metal or short pieces of small diameter pipe or rod are placed at the impact points to lessen the impact force on the restraint structure. These contact buffers are designed to function elastically after they have deformed plastically to accommodate mismatching of the surfaces.

Tornado missiles generally are of an intermediate energy level. Their total kinetic energy is dissipated by energy absorption by the affected structure as a whole. This results from the elastic and plastic response of the structure to the impact force, energy absorption by the missile itself due to plastic deformation of the missile, and by the building structure missile barrier member due to local plastic deformation.

A missile barrier of reinforced concrete will react to missile impact as a combination of non-ductile concrete and ductile reinforcing steel. The mode of concrete failure will be brittle fracture such as might result from punching shear. Shear cracks will occur at the impact area perimeter and progress outward as concentric parametric rings of fracture. A reinforced concrete member will respond elastically and plastically as a moment resisting reinforced concrete element up to the point of brittle fracture of the concrete, then the reinforcement will respond as tensile strands in membrane action, elongating plastically to absorb the kinetic energy.

Unlike concrete, the failure of a missile barrier of steel construction is ductile, warranting a larger ductility limit of 20. The steel structure will behave elastically up to the elastic limit, at which point significant plastic deformation will occur to absorb the kinetic energy of the missile. 01425577

The problem of establishing a missile barrier can be subdivided according to the behavioral response of the characteristic structural element, i.e., slab, wall, beam and column.

Slabs and walls can respond by perforating or shear failure, plastic bending and finally forming a tensile membrane as described above.

A comparison was made of various penetration formulae such as the Army Corps of Engineers, Ballistic Research Laboratory and Modified Petry before selection of the Modified Petry formula as being the most commonly used and best fitting the controlling conditions. None of the available formulae developed from empirical ballistic information are particularly suited to tornado missile problem solutions.

In using the Petry formula, the usual rule is to make a slab or wall of a thickness at least twice the penetration determined by the second Modified Petry formula for concrete of finite thickness, and this is done assuming all deformation to occur in the concrete (indestructible missile). A correction factor is applied to steel missiles on non-circular or open cross-section such as steel girts and steel pipe so that the area used in the Petry formula to determine the theoretical penetration of an indestructible missile is 3 times the net cross-sectional area of the steel.

Assumption of an indestructible missile leads to very high peak loads and shear stresses when making an analysis for impulse loading. Experiments of limited scope were therefore performed which verified that almost all of the local plastic deformation would occur in the wood for a wood missile impacting on concrete, and that steel missiles would enter a plastic range while penetrating concrete.

To provide a workable solution for applying the Petry formula to a wood missile, a “K” value predicated on the plastic deformation (or destruction) of the wood is used to determine the “penetration” or deceleration path and from this a peak load is obtained. In the case of the steel missile, the peak load is limited by the short duration yield strength of the steel.

The peak loads associated with the various missiles as shown in Table 12.2-9, are as follows:

horizontal flying wood plank, 400 kips;

vertical falling wood plank, 288 kips;

steel girts, 197 and 257 kips;

steel pipe, 180 kips;

automobile, 182 kips.

Reinforced Concrete Methodology

Using the peak load, reinforced concrete slabs and walls are analyzed for their response in shear (approximately at the ultimate strength of the concrete, in shear) and ability to develop plastic hinges and a tensile membrane of reinforcing steel. After the shear failure of the concrete slab or wall, the plastic deformation of the longitudinal reinforcing steel is calculated not to exceed a strain of 5%. 01425577

Reinforced concrete beams in a horizontal plane are analyzed for impulse loading. A rectangular forcetime curve is assumed so that the methods contained in Reference 1 (See Section 12.2.7) can be used. The dynamic system is established including boundary conditions, size of member, member characteristics, reinforcing, loading, span, etc., to determine the natural frequency and plastic strength of the member. From the peak load previously found and the plastic resistance, the ductility factor is determined which is conservatively limited to 6. The dynamic reactions are calculated for the elastic or plastic strain range, as required, and combined with other loads (Dead Loads, etc.). A minimum value of missile impact reaction of 300 kips is used in order to provide a minimum shear strength capability for missiles impacting near a support.

The allowable shear stresses used are:

$4\phi\sqrt{f'_c d}$	for un-reinforced beam webs
$6\phi\sqrt{f'_c d}$	for d/2 stirrup spacing
$10\phi\sqrt{f'_c d}$	for d/4 stirrup spacing

where ϕ = 0.85, d = 1.25, and f'_c= ultimate compressive strength of concrete, and using a minimum web reinforcement of 0.15% bs (beam width, b x bar spacing, s). Stirrup stress is limited to .85 times 1.25 fy (fy=yield strength of reinforcing bars) and bond stress is limited to $.15f'_c$ with .85 of the summation of the perimeter of bars.

Beams designed by this procedure have very minimal plastic deflection under tornado missile impact. Beams which are too small to be made to comply with the above requirements are investigated for the capability to hang from adjacent slabs as a thickened portion of the slab.

Columns are designed for a 300 kip missile impact load centered on top combined with all applicable loading. The 300 kip load was chosen to establish a minimum strength in columns subject to missile impact and exceeds the dynamic reaction from the beams.

The stress level in columns under the above loading is limited to 1.5 times the ACI code allowable to provide a higher factor of safety in the columns than are used in beam and slab design.

Steel Barrier Methodology

Similar to Reinforced concrete, steel missile barriers are designed to intercept tornado-generated missiles preventing perforation of the barrier by the missile. Unlike concrete, which is prone to fracture, steel barriers are highly ductile and primarily use plastic deformation to absorb the energy of the missile impact. Members are checked for perforation and ductility. The dynamic system is established including boundary conditions, size of members, material properties, loading, span, etc. to determine the members' response to a missile impact. The ductility ratio of the member is limited to a maximum ratio of 20. A minimum value of missile impact reaction of 300 kips (or the peak missile impact forces previously found) is used in order to provide a minimum shear strength capability for missile impacting near a support. 01425577

The listed procedures are conservative and provide for missile barriers that can absorb sufficient missile energy to reduce the missile velocity to zero without physical breach of the barrier, and keep cracking and plastic deformation within acceptable levels.

12.2.1.4.3.1.5 Design for Cathodic Protection

The installation of a plastic membrane beneath the Reactor Building, Containment Vessel, Auxiliary and Turbine Building has the effect of insulating these structures from the environment. Because of the mildly corrosive earth at the site, the concrete encasement and the plastic membrane, a very favorable environment has been provided for these structures and a long life can be anticipated.

Buried steel structures such as fuel oil storage tanks, circulating water lines, fire hydrant lines, emergency service water intake pipe, screen house and well water pump houses are protected by a cathodic protection system. This Cathodic System is based on general accepted practices for protecting buried structures. A rectifier with negative potential connected to the protected structure maintains the anodes and the surrounding environment at a positive potential with respect to the structure. Test points are provided to periodically check the effectiveness of the system.

12.2.1.4.3.2 Class I* Structures

The design of Class I* Structures is similar to the design of Class I structure for seismic loads only as detailed in Section 12.2.1.4.3.1.

In all other respects the design of Class I* Structures is identical to the design of Class III Structures as detailed in Section 12.2.1.4.3.5.

12.2.1.4.3.3 Class II Structures

Structures in this class are designed for the conditions of loading specified in Section 12.2.1.4.1 and Table 12.2-4 and in accordance with the design methods and allowable stresses specified in the codes listed in Section 12.2.1.2. Stresses are combined as before and reviewed to assure that they are within the limits set forth in Table 12.2-7.

12.2.1.4.3.4 Class III* Structures

The design of Class III* Structures is similar to the design of Class II Structures for the condition of loading specified in Section 12.2.1.4.1 and Table 12.2-4.

In all other respects the design of Class III* Structures is identical to the design of Class III Structures as detailed in Section 12.2.1.4.3.5 of this section.

12.2.1.4.3.5 Class III Structures

Structures in this class are designed for the conditions of loading specified in Section 12.2.1.4.1 and Table 12.2-4, and in accordance with the design methods and allowable stresses specified in the codes listed in Section 12.2.1.2. Stresses are combined as before and reviewed to assure that they are within limits set forth in Table 12.2-7.

12.2.1.4.3.6 Mixed Classification Structures

A Class I area located in a lower class structure is treated as a Class I structural system within the lower class structure. Components of the Class I structural system which are required to meet the total structural function of this system may extend into the lower class area and are analyzed for their Class I function. These components include related foundations, supporting structures and overhead structures.

The mathematical model used by John A. Blume in the dynamic analysis of the plant structures (see John A. Blume report JAB-PS-02) embraced all contiguous plant structures. The resultant accelerations, shears, moments, displacements and torques were used in the design of the main frames of the lower class structures and their bracing. This design was made in accordance with the parameters listed in Tables 12.2-4, 12.2-5 and 12.2-6. For this reason, the main frame of the Auxiliary Building outside the Class I area, and the main frame of the Turbine Building are classified as Class I*. At joints where seismic separations between adjacent structures were required, a gap equal to twice the sum of their respective displacements was provided.

Portions of the siding and the roof decking have been attached with pressure relief fasteners to vent the building from tornado pressures and forces. This will prevent the stresses in the main structural frames from exceeding the allowable limits established in Table 12.2-5 thus preventing their collapse onto Class I structures.

Floors and other structural elements of the lower class structures whose failure would not endanger Class I structures, systems or components are designated Class II, Class III* or Class III. These elements are designed for a Uniform Building Code, Zone I earthquake using the design parameters listed in Tables 12.2-4 and 12.2-7.

The design assures that a failure of adjacent lower class structures due to earthquake, tornado winds or missiles will not cause a loss of function to the Class I structure by direct, or indirect, failure of structural components.

12.2.1.5 General Design Criteria for Components

This section describes the General Design Criteria for all mechanical, electrical and I&C components, used in the plant design.

The seismic design method described in this section is referred to as the Seismic Qualification method. This original licensing basis method is based on the analysis and/or testing of specific components and meeting applicable acceptance criteria. An acceptable alternative method for verifying seismic adequacy is described in section 12.2.14. This alternative method applies to new, modified or replacement Design Class I mechanical and Class 1E electrical equipment. Restrictions on the use of the alternative method are also discussed in Section 12.2.14.

12.2.1.5.1 Load Combinations

The load combinations applicable to Class I, Class I*, Class II, Class III*, and Class III Components are given in detail in subsequent paragraphs and also listed in Table 12.2-11. The load combinations applicable to safety related equipment in the D5/D6 Building are given in Reference 43. The term Live Loads, when used on components, consists of thermal and pressure loads.

a. Class I Components

   Class I Components are analyzed for each of the following conditions of loading:

   1. Normal Operating Condition

      The load combination consists of Dead and Live loads together with Environmental loads (wind or snow) wherever applicable.

   2. Normal & Operating Basis Earthquake Condition

      The load combination consists of Dead and Live together with the greater of the OBE or Wind loads.

   3. Normal & Design Basis Earthquake Condition

      The load combination consists of Dead and Live together with DBE loads.

   4. Normal & Pipe Rupture

      The load combination consists of Dead, Live, and DBA loads (or other pipe rupture loads whichever is greater).

5. Normal & Design Basis Earthquake & Pipe Rupture

   The load combination consists of Dead, Live, and DBA loads (or other pipe rupture loads whichever is greater) together with DBE loads.

b. Class I* Components

Those components designated as Class I* are analyzed for each of the following conditions of loading:

1. Normal Operating Conditions

   The load combination consists of Dead and Live loads together with Environmental loads if applicable.

2. Design Basis Earthquake Conditions

   The load combination consists of Dead, Live and DBE loads.

c. Class II Components

Class II Components are designed for the greater of the following two combination of loads:

1. Dead, Live and Environmental Loads if applicable, or,

2. Dead, Live and Uniform Building Code Earthquake loads specified in Section 12.2.1.3.5.

d. Class III* Components

Class III* Components are designed for the greater of the two combinations of loads given above for Class II components.

e. Class III Components

Class III components are designed for Dead and Live loads together with Environmental loads if applicable.

As a minimum, Class III components are designed in accordance with the Applicable Codes as listed in Section 12.2.1.2. In addition, in accordance with the company's normal policy for the design of steam - electric generating stations, certain components of the power plant in the Class III category by definition are designed according to the requirements of a higher classification.

12.2.1.5.2 Design Criteria

12.2.1.5.2.1 Class I Vessels, Piping and Supports

In addition to the loads imposed on the system under normal operating conditions, the design of piping, vessels and supports requires that consideration also be given to abnormal loading conditions such as seismic and pipe rupture. Two types of seismic loadings are considered: Operating Basis Earthquake (OBE) and Design Basis Earthquake (DBE).

The allowable stress limits for each of the possible combinations are limited to those specified in Tables 12.2-12, 12.2-13, and 12.2-14. (Note that portions of the RCGVS & RVLIS piping is designed to ASME Section III Limits.) Interim criteria may be applied for piping and supports as delineated in Reference 35. The normal as well as abnormal loads must be considered singly and in combination as shown in Table 12.2-15.

The reasoning for selection of the above mentioned loading combinations and stress limits is, as follows: For the Operating Basis Earthquake the nuclear steam supply system is designed to be capable of continued safe operation or return to power operations.

In the case of the Design Basis Earthquake it is only necessary to ensure that critical components do not lose their capability to perform their safety function, i.e., shut the plant down and maintain it in a safe condition. This capability is ensured by maintaining the stress limits as shown in Tables 12.2-12, 12.2-13, and 12.2-14 for a faulted condition. The stresses induced in Class I piping by the Design Basis Earthquake are limited so that they do not cause a rupture of that pipe.

The seismic analysis and design of Class I piping has employed one of two methods:

a. Most systems such as the main steam, feedwater, residual heat removal and cooling water systems have been subjected to a dynamic multidegree of freedom modal analysis using the spectral method and the applicable floor response spectra. The stress, deflections and accelerations at each point were determined by the square root of the sum of the squared values for each mode. Valves and valve operators were considered as lumped masses in the pipe.

b. Some piping systems have been designed with sufficient restraints to remove the fundamental frequency of the system from the resonant range of the supporting structure. For such systems the floor response spectra at the attachment points were used to determine accelerations for design and stress analysis purposes.

   Seismic effects of piping systems due to the relative displacement of structures have been analyzed utilizing the results of a dynamic multidegree of freedom modal analysis of the structures using the spectral method.

The location of seismic supports and restraints for Class I piping systems were determined on the basis of a dynamic analysis of the piping systems as implied above. The design information derived from the dynamic analysis and data on thermal expansion, dead weight, and combined stress analysis (forces, moments, location and type of seismic support) were transmitted to the hanger vendor who, using this information, designed the required support, restraint, etc. The vendor designs were then reviewed to assure that the design location and characteristics of these supports and restraining devices were consistent with the assumptions made in the dynamic analysis of the system. To insure proper fabrication and erection, distribution of approved vendor designs were made to the field and to the hanger vendor. After installation a check is made for proper location and support type as indicated on the distributed drawing.

All Class I piping is isolated by structural or equipment anchors from piping for which Class I analysis is not required. Class II pipe which is connected to Class I pipe is analyzed as Class I pipe up to a structural or equipment anchor which provides a means for isolating the Class I piping from the Class II piping.

The dynamic seismic analysis of all Class I piping systems includes a flexibility analysis which accounts for the differential seismic movements at points of piping attachment to the structure. These movements are imposed as boundary conditions in the seismic analysis and are calculated by considering both the relative movement within a structure and the relative movement between structures. All movements are superimposed in such a manner so as to yield the maximum relative displacements. The resulting seismic induced stresses were combined with the stresses due to the other loading conditions as given in Table 12.2-13. Stresses were maintained within allowable values in accordance with the ANSI B31.1.0 code for Power Piping.

Stresses for safety related piping systems in the D5/D6 Building were maintained within allowable values in accordance with the requirements of ASME Section III, Class 3 1986 edition, with exceptions highlighted in the Design Report (see Reference 43).

Seismic effects on buried Class I piping are expected to be insignificant. For buried pipe, seismic displacements and resultant stresses are limited to the forced deformation of the pipe by the seismic shear waves in the soil and the displacement of the structural base mat at the point of entry into the structure. Analysis of piping bending stresses due to seismic shear wave deformation has shown these stresses to be negligible (less than 2,000 psi). Maximum calculated OBE base mat displacements are less than 0.1″ during the seismic event. The surrounding graded and compacted granular fill material is not expected to move differentially with respect to the base mat at the interface during an earthquake and will not undergo significant settlement relative to the base mat subsequent to a seismic occurrence. Additionally, seismic analysis considering the relative displacements of terminal and support points of non-buried piping, which are greater than those for buried piping, consistently show piping stress levels less than 2,000 psi.

Reaction forces at points of piping attachment to the structure are developed in the piping analysis. These reaction forces were used in an analysis of the supporting structure and the resultant stresses were maintained within the appropriate code allowable values.

Careful design and thorough quality control during manufacture, construction and periodic inspection during plant life, ensures that the independent occurrence of a reactor coolant pipe rupture is extremely remote. If it is assumed that a reactor coolant pipe ruptures, the stresses in the unbroken leg will be limited to those for a faulted condition in Tables 12.2-12, 12.2-13, and 12.2-14.

The probability of the simultaneous occurrence of the Design Basis Earthquake and a reactor coolant pipe double-severance is practically zero. However (for the initial design), for this extremely remote event, the design of Class I piping and components, excluding the broken leg, is checked for no loss of function, i.e., the capability to contain fluid and allow fluid flow. This is assured by limiting the various stress combinations within the limits specified for faulted condition. Subsequently, leak-before-break (LBB) was approved for Prairie Island (refer to Sections 4.6.2.3 and 4.6.2.4 for details). With LBB it is not necessary to assume a complete severance of the piping approved for LBB for the dynamic considerations discussed in Section 4.6.2.3. However, new designs of Class 1 piping and components, excluding the broken pipe, are checked for no loss of function assuming the severance of piping not approved for LBB application.

The design limit curves that give the allowable stresses for stainless steel for faulted conditions are developed by using the approach presented in WCAP 5890 (Reference 3). This report develops limit curves by using 50% of the ultimate strain as the maximum allowable membrane strain. Subsequent to the submission of WCAP 5890, the allowable membrane strain was limited to 20% of the ultimate strain. Design limit curves are developed by using the following procedure:

i. Use material data to develop stress-strain curves.

   Stress-strain curves of Type 304 stainless steel, Inconel-600, and SA-302B low alloy steel at 600°F have been generated from tests using graphs of applied load versus cross-head displacement as automatically plotted by the recorder of the tensile test apparatus. The scale and sensitivity of the test apparatus recorder assure accurate measurement of the uniform strain.

   For other materials, stress-strain curves are developed by conservative use of pertinent available material data (i.e., lowest values of uniform strain and initial strain hardening). Should the available data not be sufficient to develop a reliable stress-strain curve, three standard ASTM tensile tests of the material in question will be performed at design temperature. These data will be conservatively applied in developing a stress-strain curve as described above.

ii. Normalize the ordinate (stress) of the stress-strain curves to the measured yield strength. (See Figure 12.2-1)

iii. Use 20% of uniform strain as defined on the curve developed under Item (i) as the allowed membrane strain.

iv. Establish the normalized stress ratio at 20% of uniform strain on the normalized stress ratio-strain curves developed under item (ii).

v. Establish the value of the membrane stress limit.

   Multiply the normalized stress ratio in Item (iv) by the applicable code yield strength at the design temperature to get the membrane stress limit. As an alternate, the actual physical properties, as determined from standard ASTM tensile tests on specimens from the same heats, is used to determine the membrane stress limit.

vi. Develop limit curves for the combination of local membrane and bending stresses.

   The limit curves are developed by using the analytical approach presented in WCAP-5890, Rev. 1 and the stress-strain curve up to the membrane stress limit as developed under Item (v). In addition dynamic and stability analyses are performed, where required.

Examples of design limit curves developed are given in Figures 12.2-2A and 12.2-2B.

The Operating Condition categories are defined as follows (see also Tables 12.2-12, 12.2-13, 12.2-14 and 12.2-15):

a. Normal Condition - Any condition in the course of system startup, operation in the design power range, and system shutdown, in the absence of Upset, Emergency, or Faulted Conditions.

b. Upset Condition - Any deviations from Normal Conditions anticipated to occur often enough that design should include a capability to withstand the conditions without operational impairment. The Upset Condition includes those transients caused by a fault in a system component requiring its isolation from the system, transients due to a loss of load or power, and any system upset not resulting in a forced outage. The estimated duration of an Upset Condition should include the effect of the specified earthquake for which the system must remain operational or must regain its operational status.

c. Emergency Condition - Any deviations from normal conditions which require shutdown for correction of the conditions or repair of damage in the system. The conditions have a low probability of occurrence but are included to provide assurance that no gross loss of structural integrity will result as a concomitant effect of any damage developed in the system. The total number of postulated occurrences for such events shall not exceed twenty-five (25).

d. Faulted Condition - Those combinations of conditions associated with extremely low probability postulated events, whose consequences are such that the integrity and operability of the nuclear energy system may be impaired to the extent where considerations of public health and safety are involved. Such considerations require compliance with safety criteria as may be specified by jurisdictional authorities. Among the Faulted Conditions may be a specified earthquake for which safe shutdown is required.

The design stress limits associated with faulted conditions were applied to all Class I systems and components where pipe rupture effects could occur. The bases for these applications is given above while the loading combinations and associated design stress limits are given in Tables 12.2-11 through 12.2-15. The design stress limit for Class I components not delineated in the tables is 90% of the material yield strength.

12.2.1.5.2.2 Class I & Class I* Equipment

12.2.1.5.2.2.1 Reactor Vessel Internals

a. Design Criteria for Normal Operation

The internals and core are designed for normal operating conditions and subjected to loads of mechanical, hydraulic, and thermal origin. The response of the structure under the Operation Basis Earthquake is included in this category.

The stress criteria established in Section III of the ASME Boiler and Pressure Vessel Code, Article 4, has been adopted as a guide for the design of the internals and core with the exception of those fabrication techniques and materials which are not covered by the Code, such as fuel rod cladding. Seismic stresses are combined in the most conservative way and considered primary stresses.

The stress criteria of ASME Section III, are based on a limit design theory with the assumption that the material behavior is perfectly plastic with no strain - hardening. The criteria are chosen so that the structure has a sufficient margin against the limit load for primary stresses and that shakedown to elastic behavior is assured for secondary stresses.

The members are designed under the basic principles of: (1) maintaining distortions within acceptable limits, (2) keeping the stress levels within acceptable limits, and (3) prevention of fatigue failures.

To study the seismic response of the reactor internals, a dynamic, elastic study is performed. The maximum stresses are obtained by combining the contributions from the horizontal and vertical earthquakes by adding components. These stresses are then superimposed on the normal operating stresses. The following paragraphs describe the horizontal and vertical contributions for the standard 2-loop, 12-foot core, reactor internals.

b. Design Criteria for Abnormal Operation

The abnormal design condition assumes blowdown effects due to a pipe break combined either by square root sum of the squares or a more unfavorable manner with the effects associated with the Design Basis Earthquake.

For this condition the criteria for acceptability are that the reactor be capable of safe shutdown and that the engineered safety features are able to operate as designed. Consequently, the limitations established on the internals for these types of loads are concerned principally with the maximum allowable deflection criteria for critical structures under normal operation, plus the maximum potential earthquake and blowdown excitation.

For the blowdown accident, the assumption of a perfectly plastic material with yield stress equal to the initial yield stress of actual material at temperature is too conservative. Therefore, for this case we are in agreement with the NRC developed stress criteria, which is based on the same limit analysis concept as the criteria of Section III, but takes credit for the strain hardening capabilities of the materials. The allowable stress values are based on the actual stress-strain curve of 304 SS at 600°F.

12.2.1.5.2.2.2 Engineered Safeguards Systems

For mechanical components of Engineered Safeguards Systems, analyses have been performed on a worst plant basis to determine the response in the frequency range of interest. Modifications were made as necessary considering potential for resonance responses. The component was then analyzed using seismic loads as obtained from building response calculations to show that stresses and deflections are within allowable limits and will not result in loss of function. The equipment was considered in the operational as well as static mode.

The standard Westinghouse 2-loop analysis uses an envelope of response acceleration spectra which is more conservative than those presented in the Blume Reports (References 6 and 8).

Seismic Criteria for Westinghouse Furnished Equipment

1. Equipment specifications to vendors required that Westinghouse supplied Seismic Class I Auxiliary Pumps be designed by the vendor to operate during horizontal and vertical acceleration of 1.0g and 0.67g, respectively and simultaneously. The sum of the primary stresses shall not exceed Section III of the ASME Code for pressure containing members and other critical components.

2. Seismic Class I tanks were designed to withstand the simultaneous horizontal and vertical forces resulting from the amplified ground acceleration response spectrum curves for the DBE.

3. Seismic Class I valves were designed by the vendor to withstand seismic loadings equivalent to 3.0g in the horizontal direction and 2.0g in the vertical direction.

The typical Westinghouse supplied Class I mechanical components for Prairie Island which require a seismic analysis are checked for seismic adequacy by employing the following procedures:

The manufacturers drawings were reviewed to classify the component.

a. If the component fell within a category which has been previously analyzed using a multi-degree-of-freedom model and shown to be relatively rigid, then the equipment specification for that component was checked to ensure that the response spectra was smaller than the specified values.

b. If the component could not be categorized as similar to previously analyzed components, then a seismic modal analysis was performed as described below.

Stresses and deflections are checked to ensure that they will not result in loss of function.

Typical Westinghouse supplied Class I mechanical equipment of the Engineered Safeguards Systems including heat exchangers, pumps, tanks and valves are seismically analyzed using a multi-degree-of-freedom modal analysis. All contributing modes are considered. In addition, it should be pointed out that a sufficient number of masses are included in the mathematical models to insure that coupling effects of members within the component are properly considered. The results of these analyses indicate that the models contain more masses than necessary, and that future analyses of comparable equipment could be considerably simplified by considering fewer masses or merely a simple static approach. The method of dynamic analysis uses a proprietary computer code called WESTDYN. This code uses as input, inertia values, member sectional properties, elastic characteristics, support and restraint data characteristics, and the appropriate seismic response spectrum. Both horizontal and vertical components of the seismic response spectrum are applied simultaneously. The modal participation factors are combined with the mode shapes and the appropriate seismic response spectra to give structural response for each mode. The internal forces and moments are computed for each mode from which the model stresses are determined. The stresses are then summed using square-root-sum-of-squares method.

The horizontal and vertical floor response spectra submitted by architect-engineer are considered in the analysis. An umbrella spectrum is used in the analysis which encompasses the floor response spectra.

Typical Class I Engineered Safeguards tanks supplied by Westinghouse, e.g. Boric Acid, Accumulator, and Surge, are analyzed using the above method, for horizontal and vertical seismic excitation occurring simultaneously, and in conjunction with normal loads. Hydrodynamic analyses of these tanks is performed using the methods described in Chapter 6 of the “U. S. Atomic Energy Commission - TID 7024”.

Typical Class I heat exchangers supplied by Westinghouse associated with the Engineered Safeguards Systems, e.g. Residual Heat Removal, are analyzed using the above mentioned method.

Seismic Design Criteria for Balance of Plant Class I (Seismic) Mechanical Components and Electrical Equipment

1. For the operating basis earthquake, the mechanical components and electrical equipment are designed to be capable of continued safe operation within normal design limits when subjected to the combination of normal loads and OBE loads.

2. For the design basis earthquake, the mechanical components and electrical equipment are designed so that the deflections or distortions resulting from the combination of normal loads and twice the OBE loads shall not prevent their proper functioning, shall not endanger adjacent or attached equipment, and shall not cause the equipment to operate in an uncontrolled manner.

In order to meet these seismic design criteria the following measures were taken for seismic design and restraint:

1. The mechanical component or electrical equipment and its supports were designed to be sufficiently rigid so that its natural frequency or frequencies will be out of the range of resonance with the building structure where it is located.

2. The maximum stresses induced from the combination of normal loads plus OBE loads were maintained below the allowable stress limit of the material as given in the applicable codes. Subsequent re-analysis of motor operated valves to determine the subcomponents weak links uses 0.60Sy as the acceptance criteria for tensile allowables for non-pressure retaining components for Normal and 0.80Sy for Upset conditions.

3. The maximum stresses induced from the combination of normal loads plus twice the OBE loads were maintained at less than 90% of the yield strength of the material under consideration, and the deflections or distortions will be so limited that they would not affect power functioning of the equipment.

The analytical or testing methods utilized to verify the adequacy of the above are described as follows:

1. Analytical methods

    a. Determine the natural frequency or frequencies of the component or equipment under consideration by the use of a proper mathematical model.

    b. For a single-degree-of-freedom model, use the natural period to determine the horizontal and vertical response accelerations from the structural floor response acceleration spectra. Apply these accelerations at the center of mass of the component simultaneously and analyze the system statically.

    c. For a multiple-degree-of-freedom model, use the modal superposition method to determine the responses of the dynamic system.

    d. For those components for which the natural frequency cannot be determined, apply the peak values of the structural floor response accelerations multiplied by the maximum torsional acceleration factor at the center of mass of the component and determine the seismic responses.

    e. For the DBE, the response acceleration values are twice those used for the OBE.

2. Testing methods

   a. Continuous Test

      The test is executed at frequencies incremented within the range of significant structural response of the applicable structural response spectra. The test consists of the application of a continuous sinusoidal motion corresponding to the maximum structural acceleration for which the equipment is qualified and for an appropriate length of time. The equipment is properly mounted during testing so as to reflect the field installed condition and energized if required.

   b. Sine Beat Test

      Natural or resonant frequencies are detected by scanning from the lowest practical frequency to 25 Hz. The test at resonant frequencies consists of the application of sine beats of peak acceleration values corresponding to that for which the equipment is qualified. The duration of the beat for each particular test frequency is chosen to most nearly produce a magnitude of equipment response equivalent to that produced by the particular floor acceleration with proper damping ratio. The equipment is properly mounted during testing so as to reflect the field installed condition and energized if required.

      Seismic input values used for analysis or testing purposes to verify the adequacy of Class I (seismic) components were obtained from the John A. Blume Report (JAB-PS-04)(Reference 8). Seismic input values used for analysis or testing purposes to verify seismic components in the D5/D6 Building addition were obtained from Regulatory Guide 1.60 response spectra as discussed in Reference 43.

12.2.1.5.2.2.3 Instrumentation and Control Systems

The design bases for protection grade equipment (Class I) with respect to earthquakes is that for an Operational Basis or Design Basis Earthquake, the equipment is designed to ensure that it does not lose its capability to perform its function; i.e., shut the plant down and/or maintain the unit in a safe shutdown condition. For the Design Basis Earthquake, the capability of the protection equipment to perform its function is maintained.

If a seismic disturbance occurs subsequent to an accident, the instrumentation and electrical equipment associated with emergency core cooling will not be interrupted during this disturbance.

Initial evaluation of Protection System equipment for its ability to withstand the seismic condition is typically done by actual vibration type testing of typical protection grade equipment. Mathematical models derived from empirical tests are not normally used for seismic design evaluation of instrumentation. However, in the absence of empirical test data, such as may be the case for very large equipment (for example, control room panels), evaluation may be supported by mathematical analysis or some combination of mathematical analyses and prototype testing. (See Reference 7 for discussion and documentation of some test program results.)

12.2.1.5.2.3 Class II and Class III* Components

Components in this class are designed for the conditions of loading specified in Table 12.2-11 and in accordance with the design methods and allowable stresses specified in the codes listed in Section 12.2.1.2. Stresses are combined as for Class I above and reviewed to assure that they are within the limits set forth in the applicable codes.

12.2.1.5.2.4 Class III Components

Components in this class are designed for the conditions of loading specified in Table 12.2-11 and in accordance with the design methods and allowable stresses specified in the codes listed in Section 12.2.1.2.

As discussed in Section 10.4.1.2.4, there are several piping runs in the cooling water system that are not analyzed for seismic loads and the lines are not isolated from the safety related (seismic) headers following a seismic event. The hydraulic analysis of the cooling water system that demonstrates functionality following a seismic event assumes a complete failure (rupture) of a non-seismic pipe at the worst-case location. The intent of assuming a complete failure of one pipe is to bound possible cases where partial failures (cracks) may occur in multiple lines. The size of the non-seismic piping break can be reduced using stress analyses techniques of the non-Class I cooling water piping for seismic loads (References 63, 64, and 65). In lieu of the acceptance criteria in Tables 12.2-13 and 12.2-14, the stress analysis may use acceptance criteria in ASME, Section III, Sub-Section ND, Level D Service Limits. CL non-seismic piping that is analyzed (stress analysis) in order to change the assumed pipe rupture location will be included in the scope of the Generic Letter 89-13 program.

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12.2.2 Reactor Containment Vessel Design

12.2.2.1 Design Loadings

12.2.2.1.1 Dead Loads

Dead loads consist of the dead weight of the Reactor Containment Vessel and its appurtenances, the weight of internal concrete, and the weight of structural steel and miscellaneous building items within the Reactor Containment Vessel.

Weights used for dead load calculations are as follows:

a.	Concrete:	143 PCF
b.	Steel Reinforcing:	489 PCF using nominal cross section areas of reinforcing bar sizes as defined in ASTM A 615
c.	Steel Containment Vessel:	489 PCF

12.2.2.1.2 Design Basis Accident (DBA)

This load was determined by analysis of the transient pressure and temperature effects that could occur during the Design Basis Accident. (See Section 14).

12.2.2.1.3 Operating Loads

Operating loads include the following:

a. Gravity loads from all equipment and piping;

b. Weight of water in the refueling cavity;

c. Weight of crane;

d. Loads resulting from the restraint of the free movement of the Vessel at the line of embedment in concrete;

e. Piping reactions at nozzles resulting from thermal movement.

Equipment loads used were those specified on the drawings supplied by the manufacturers of the various pieces of equipment. Floor loadings for the design of internal slabs are consistent with their intended use.

12.2.2.1.4 External Pressure Load

During normal operation, annulus pressure is essentially ambient barometric pressure. The Reactor Containment Vessel's shell plates are of suitable thickness to meet the specified internal pressure requirements and are capable of withstanding an external pressure differential of 0.8 psi in accordance with paragraph UG-28 of Section VIII of the ASME Boiler and Pressure Vessel Code.

Automatic vacuum relief devices are used to prevent the Reactor Containment Vessel from exceeding the external design pressure in accordance with the requirements of the ASME Boiler and Pressure Vessel Code, Section III, Article 17. The design and analysis of the vacuum breaker capability is discussed in Section 5.2.

During and following the Design Basis Accident (LOCA), the annulus pressure will not exceed 1-inch water column positive or 5-inches water column negative. These values include results of analysis examining effects of a single failure within the Shield Building Ventilation System.

The containment vessel is designed for a maximum external operating pressure of 0.8 psi in conjunction with the vessel dead loads and horizontal and vertical seismic loads. There are no local impact loads on the vessel since all piping has been adequately restrained to prevent pipe whip and missile shield have been provided to exclude missiles striking the vessel.

The criteria that were used to determine the maximum allowable external pressure that was specified for the vessel are given in the ASME Boiler and Pressure Vessel Code, Section III, N-1111 and Section VIII, UG-28, UG-33(c), and Appendix V.

In accordance with rules of Paragraph UG-28 of the code, the vessel design indicates margins for a maximum allowable external pressure of approximately 25 percent greater than the maximum external pressure which the vessel is specified to withstand.

The maximum membrane meridian and circumferential stresses for an external pressure of 0.8 psi together with the vessel dead loads and including horizontal and vertical earthquake effects are listed in the following table.

MAXIMUM STRESSES AND DEFORMATIONS IN CONTAINMENT VESSEL SHELL UNDER LOADINGS OTHER THAN INTERNAL PRESSURE

	Membrane Stress (PSI) 0.8 psi External Pressure Plus Dead Load Plus Horizontal and Vertical Earthquake Loads		Buckling Factor		Radial Deformation (Inches)	
Type of Stress	OBE	DBE	OBE	DBE	OBE	DBE
Meridian	-1119	-1340	.34	.39		
Circumferential	-337	-337			.007	.007

It is to be observed that the radial deformation resulting from the circumferential stresses of the table above is very small and well within the elastic range of the material, being on the order of 0.500t. The formulae used to calculate these stresses are generally accepted to be valid for radial deformations in the range of up to 0.5t.

Since stress is proportional to deformation, it is more meaningful for our purposes to consider stress limits rather than deformation limits in the evaluation of the margins provided against unstable deformation.

The basis and references for the stress evaluation given in the following discussion are found in the Welding Research Council Bulletin No. 69, “Biaxial Stress Criteria for Large Low-Pressure Tanks” by J. J. Dvorak and R. V. McGrath.

Under the conditions of load stated, the vessel shell is in a condition of unequal biaxial compressive stress within the elastic range.

For this condition of unequal biaxial stress the following two criteria are imposed.

$$\frac{\text{Smaller Stress}}{0.9 \times 10^6 \times \frac{t}{R}} + \frac{\text{Difference in Stress}}{1.8 \times 10^6 \times \frac{t}{R}} \leq 1.0 \qquad (1)$$

$$\frac{\text{Smaller Stress}}{0.9 \times 10^6 \times \frac{t}{R}} \geq 1.0 \qquad (1)$$

(2)

Where:

t = Thickness of Shell in Inches

R = Radius of Cylinder (630 in.)

The calculated factor determined by the sum of the terms of equation (1) is referred to as the “Buckling Factor” in the table above. The significance of this “Buckling Factor” is that it represents that portion of the total available allowable elastic buckling stress that can be attributed to the actual biaxial stress condition that exists under the loads being considered. From this, it is demonstrated that we are experiencing only 34 and 39 percent of the maximum allowable biaxial stress due to external pressure and dead loads together with OBE and DBE respectively.

It is also to be noted that additional design margins have been provided in equations (1) and (2). The stresses represented by the denominator terms of these equations include a safety factor of 2.0 over that reported in the tests of prototypes.

12.2.2.1.5 Seismic Loads

Seismic loads were computed using the following:

a. Operating Basis Earthquake seismic ground acceleration of 0.06g (See Appendix E).

b. Design Basis Earthquake seismic ground acceleration of 0.12g.

A vertical component of 2/3 of the horizontal ground acceleration is applied simultaneously with the horizontal acceleration.

The Reactor Containment Vessel earthquake design included the seismic effects of the inertial mass of the air locks and equipment door, and the seismic effects of the airlocks vibrating as an independent system. The independent vibration effects are considered to act in two directions:

a. along the longitudinal axis of the air lock;

b. in a rotational direction about the point of support of the Vessel shell.

The seismic effects of the inertial mass of the crane is included in the Reactor Containment Vessel earthquake design.

During an OBE or DBE the steel containment vessel is subjected to dynamic vertical and horizontal loads and dynamic overturning moments are transmitted into the concrete foundation and subfoundation through the combined action of friction and bearing on the shell bottom.

The lateral and vertical loads caused by earthquake are of such a minimal nature that the friction is neglected to simplify the analysis but conservative approximations of the bearing loads are considered.

The seismic overturning moment of the containment vessel will produce vertical reaction pressures on the shell bottom. The horizontal seismic forces produce lateral pressure reactions on the shell bottom. Since the forces are minimal, a conservative, simplified mode of lateral load transfer for analysis of steel and concrete is made.

First, the load analysis consists of using John A. Blume's modal analysis seismic acceleration curves for the containment vessel. The calculation of the vessel shears, membrane, and surface stresses is by the application of the Kalnin's program to shells. The most critical area of stress in the steel shell under these conditions occurs in the discontinuity zone at the line of the external embedment in the concrete. The calculated stresses and allowable stresses at this line are given in Table 12.2-16. The allowable stresses are based on the criteria of Table 12.2-17.

Next, the total lateral seismic load of the containment vessel is conservatively assumed to be carried by tangential shear above the embedment line into the vessel plates whose planes are almost oriented in the direction of the seismic loads. These loads are then carried by membrane action into a hoop band of limited width at or near the mezzanine floor line. This hoop band, under membrane tension, bears against the mass of the internal concrete at the mezzanine floor line as an integral internal member, thereby transferring this lateral load to the diametrically opposite surface of the internal member, from where it is transferred by concrete bearing through the shell plate which in turn bears on an expanded area of the concrete grout foundation external to the vessel.

To further simplify the analysis this band is conservatively assumed to be 20 in. wide and the resulting stresses and allowable stress criteria are shown in Table 12.2-18.

The plots of the seismic response spectra are shown in Appendix E. The classification of plant structures and equipment, and the applicable damping factors are shown in Tables 12.2-1 and 12.2-8 respectively.

12.2.2.1.6 Foundation Deformation Loads

During grouting, while the Vessel was supported on temporary columns, deformations of the base slab due to the weight of grout were not imposed on the Reactor Containment Vessel. Deformations of the base slab at this time were accommodated by the manner in which the grout was placed.

The bottom internal deck structure that was fitted to the inside contour of the Vessel bottom, the grout base that confines the outside of the Vessel bottom, and the heavily reinforced slab foundation mat form a stiff integral structural system capable of transmitting all internal building loads directly into the supporting soil with negligible relative deformation of the system.

12.2.2.1.7 Internal Test Pressure

The Containment Vessel was designed to be internally pressure tested on temporary supports. The Vessel can also be tested to 46 psig at any time during its service life.

Prior to reactor operation the Vessel can be tested to an over-pressure test of 51.8 psig.

12.2.2.1.8 Thermal Stresses in Steel Shell Due to Temperature Gradients

The steel shell in the knuckle region was designed for the combined pressure and temperature gradients present.

12.2.2.1.9 Pipe Reaction and Rupture Forces

Pipe ruptures were postulated in the high pressure portions of all piping systems and the resulting jet forces considered in the design of the containment vessel and vessel penetrations.

In the design of the vessel shell, consideration was given to potential hazards from jet impingements resulting from ruptures of adjacent piping. The force of the jet impinging upon the vessel was computed as a function of distance from the hypothetical rupture. All high pressure piping within containment was examined to assure that the selected routings imposed no potential hazard.

The combinations of loading which are to include pipe rupture forces and the associated stress limits, are given in Table 12.2-17.

The analysis of piping reaction forces acting on the steel shell penetration nozzles is based on the algebraic summation of the loading and movements derived from the analysis of the containment vessel, interior and exterior structures, and the attached piping systems.

This analysis included:

a. Piping systems under dead load, live load, thermal, and seismic conditions.

b. Containment vessel and interior and exterior structures under dead load, live load, thermal and seismic conditions.

The analysis considered the interactions between the exterior and interior structures and the containment vessel and included any reversal of penetration reaction forces that could occur.

Major piping systems are not anchored to the containment shell, but are allowed to move freely through the shell penetration. Indirect piping loads on the penetration bellows that are used to seal these systems at the containment boundary have been included in the containment vessel penetration analysis and design.

Piping systems that are directly attached to the containment shell (without expansion bellows) are of small-size pipes and by geometry and design are flexible, imposing minimal loads on the containment shell.

The moment, shear, and radial thrust structural capabilities of all of the vessel shell penetrations have been analyzed by the Chicago Bridge and Iron Co. and have been provided and documented in the Containment Vessel Stress Report on file at the site.

The appropriate combinations of loads, moments, shears, and radial thrusts have been applied to the vessel penetrations in conjunction with the operating load parameters of the piping system analysis and the resulting stresses are found to be within suitable margins of the specified design criteria to preclude any damage to the steel shell or any anchorage system.

12.2.2.2 Design Codes and Classification

The design, fabrication, inspection, and testing of the Reactor Containment Vessel complies with the requirements of the ASME Boiler and Pressure Vessel Code, Section II, Materials; Section III, Nuclear Vessels, Subsection B, “Requirements for Class B Vessels”; Section VIII, Unfired Pressure Vessels; and Section IX, Welding Qualifications Appendix IX.

The Reactor Containment Vessel is a Class B vessel as defined in the ASME Boiler and Pressure Vessel Code, Section III, Nuclear Vessels N-132. Its design and construction meets all the requirements of state and local building codes.

The Reactor Containment Vessel is code stamped for pressures of both 46 psig and 41.4 psig in accordance with Paragraph N-1500.

The design internal pressure for the Reactor Containment Vessel is as specified in the provisions of the “Winter 1965 Addenda” to Section III of the ASME Boiler and Pressure Vessel Code.

The design requirements for Class B vessels are contained in Paragraphs N-1311 through N-1314 of the addenda. Paragraph N-1312 states that the design internal pressure may differ from the maximum internal pressure but may not be less than 90 percent of the maximum containment internal pressure. A maximum internal pressure of 46 psig and a design internal pressure of 0.9 x 46 = 41.4 psig have been specified.

The Reactor Containment Vessel has been pressure tested for acceptance of the Vessel, air locks, equipment door and all Vessel penetration nozzles in accordance with the rules of Section VIII, UG-100 and Section III, N-1314 (d). The maximum test pressure was 1.25 times the design internal pressure of 1.25 x 41.4 = 51.8 psig.

12.2.2.3 Materials

The Reactor Containment Vessel is fabricated from SA516 Grade 70 steel plate meeting SA 300 requirements except that impact test requirements are as specified in the ASME Boiler and Pressure Code, Section III, N-1211 (a) for a minimum service metal temperature of 30°F.

Charpy V-Notch specimens (ASTM A370 Type A) used for impact testing materials of all product forms were in accordance with the requirements of the ASME Boiler and Pressure Vessel Code, Section III, N-330. All material except austenitic stainless steels or non-ferrous metal associated with the Reactor Containment Vessel has a NDT temperature of at least 0°F when tested in accordance with the appropriate code for the material.

During reactor operation and during pressurization for pressure and leak-rate testing, the Reactor Containment Vessel metal temperature is maintained above 30°F.

12.2.2.4 Design Stress Criteria

The Reactor Containment Vessel retains the capability to restrict leakage to the acceptable specified level under all conditions of loading that might occur during its lifetime. The vessel is designed to exhibit a general elastic behavior under accident and all earthquake loading conditions. No permanent deformations due to primary stresses are permitted in the design under any conditions of loading.

For the Operational Basis Earthquake designed as Loading Condition 1, in Table 12.2-17, the Reactor Containment Vessel is capable of continued safe operation during a DBA. For this loading condition, the structure functions within the normal design limits specified by Section III of the ASME Boiler and Pressure Vessel Code, Figure N-414 and as listed in Table 12.2-17. Loading Condition 1 provides the design basis upon which the Reactor Containment Vessel is code stamped.

For the Design Basis Earthquake designated as Loading Condition 2, the margin provided in the design assures the capability to maintain the Vessel in a safe operating condition. For this loading condition, the basic design was reviewed to insure that the Reactor Containment Vessel and its components retain their capability to perform their containment function.

Primary stress intensities are conservatively limited to 90% of the yield strength of SA 516-GR70 carbon steel plate at the accident temperature. The application of this criteria to Table N-414, ASME Boiler and Pressure Vessel Code, Section III is shown under Loading Condition 2 of Table 12.2-17.

Earthquake stresses are added linearly and directly to stresses caused by the Design Basis Accident, dead loads, and the appropriate operating loads to obtain the total stresses. These total stresses are within the maximum stress limits allowed by the design criteria, listed in Table 12.2-17.

Prior to the special ruling of ASME Code Case 1392 (Reference 2), the interpretation of the ASME Code for the design of containment vessels was to treat all vessel configurations and loading under the design rules of Section III (b) and satisfy the basic stress limits of paragraphs N-414.1, N-414.2, N- 414.3 and N-414.4 of Section III.

The interpretation presently permitted by this code case is to accept the design provisions of Section VIII in the absence of substantial mechanical or thermal loads other than pressure.

The design rules of Section VIII are satisfied for all configurations and loadings explicitly treated by Section VIII, using “Sm” (See N-1314 (b)) in place of “S” in the various formulas.

The requirements of ASME Code Case 1392 are applicable to the Reactor Containment Vessel. The minimum of the bottom configuration is predicted on the design rules of Section VIII.

In consideration of the large diameter of the vessel, the shell bottom was analyzed by the “Yale Shell Program” (Reference 4).

Circumferential compressive stresses resulting from internal pressure forces were calculated and held below the critical buckling stress by a margin of safety compatible with good pressure vessel design practice.

The design was reviewed to assure that any resulting deflections or distortions would not prevent the proper functioning of the structure or pieces of equipment and would not endanger adjacent structures or components. The ASME Pressure Vessel Code provisions for out-or-roundness tolerance was not considered appropriate for the cylindrical vessel of the magnitude of the Reactor Containment Vessel. It was therefore specified to have an out-of-roundness tolerance of not greater than one half of the ASME Boiler and Pressure Vessel Code Permissible tolerance, i.e., one-half percent of the normal diameter. For the 105 foot diameter containment vessel, the requirement results in a maximum permissible out of roundness of 6.30 inches. The remaining tolerances were specified to be the same as the requirements of the ASME Code, Section VIII.

The maximum out of roundness of the as-built Unit No. 1 containment vessel is 5-1/4 inches, and for the Unit No. 2 containment vessel it is 5-3/8 inches. Thus, both vessels are within specified tolerance and the functional safety of the system is not affected.

All other applicable tolerances of fabrication and erection as specified in the ASME Boiler and Pressure Vessel Code, were applied to the Reactor Containment Vessel.

The ellipsoidal bottom of the Reactor Containment Vessel is bonded to and in intimate contact with the support grout under the Vessel bottom. It is noted that those surfaces internal and external to the vessel that are in contact with concrete are not readily available for inspection.

This concept of Reactor Containment Vessel support and internal concrete construction is recognized by the B & PV Code and has been licensed on numerous nuclear power plants.

The successful application of this method of support is predicted on the inherent built-in quality of construction associated with pressure vessels and the test requirements of acceptance and code certification.

All weld seams on the bottom are fully radiographed and have been soap bubble-tested at 5 psig and also at the design internal pressure.

12.2.2.5 Design Review and Analysis

The Reactor Containment Vessel was designed, fabricated, constructed, and stamped in accordance with the rules of Section III, Subsection B, of the ASME Boiler and Pressure Vessel Code.

The bottom head of the Reactor Containment Vessel was designed using the formula of Section VIII of the ASME Code as allowed by Paragraph N-134 (a) (1) when substantial mechanical or thermal loads other than pressure are not present.

The containment vessel was built in one-stage construction namely, the vessel was built in one continuous construction sequence and the bottom was not covered with concrete until the entire vessel was completed and pneumatically tested and all weld seams inspected during pneumatic testing. Therefore, the rules of paragraph N-1411 of the 1965 edition of Section III of the ASME B&PV Code which applies to two-stage construction where the lower part of a vessel is built and tested first and then covered with concrete after which the upper part of the vessel is constructed do not apply to the one-stage construction used on the containment vessel for this plant.

Temporary stiffeners were added to protect the structure during the overload pressure test, and prevent the possibility of any damage to the Vessel during construction. Following the overload pressure test, the temporary stiffeners on the bottom head were removed to prevent damage to the internal concrete or the Vessel that could result from movements of the Vessel.

The removal of the temporary columns and stiffeners in the knuckle area was performed in accordance with Section VIII of the ASME B&PV code. Removal was accomplished by arc air gouging the metal to be removed to within 1/4″ of the vessel plate surface, then grinding the remaining metal smooth to the surface of the vessel.

The minimum plate thickness tolerance is specified as minus 0.01 in. maximum in the ASME B&PV Code, Section VIII, Paragraph UG-16. Gages were used in grind areas to assure that the minimum thickness was not below the specified tolerance. All grind areas were magnetic particle inspected.

The Reactor Containment Vessel’s concrete fill support under the ellipsoidal bottom was placed using a concrete placement and grout technique as follows:

a. The concrete fill was placed in a predetermined sequence of pours. The size, location and timing of placement of the individual pours were all considered in determining the pattern and sequence of pours.

b. The placement of low-viscosity chemical grout proceeded in sequence during the above operation after the concrete grout had completed its cycle of hydration and shrinkage. The chemical grout was pressure injected through grouting ducts. Injection continued at each duct until the chemical grout appeared around the periphery of the pour. This method ensured full bearing under every part of the containment vessel’s ellipsoidal bottom.

After embedment in concrete, the bottom head was subject to both thermal and pressure loads not considered by the formulas of Section VIII and a detailed thermal study and stress analysis was performed to show compliance with the stress intensity limits of N-414.1, N-414.2, N-414.3 and N-414.4 as required by N-1342 (a)(2). This analysis is of the type normally required for Class A vessel.

The containment vessel has been specified to have charpy V-notch temperature requirements not higher than 0°F. Charpy V-notch test data which demonstrate the nil-ductility transition temperature which is at least as low as 0°F, are to be found in the voluminous record of impact test data on file at the site.

The Shield Building, as described in Section 5.3, protects the entire containment vessel from direct exposure to the outside atmosphere. An analysis of temperature gradients across the shell itself, the bulk air within the annulus, and the Shield Building (assuming an outside temperature of - 20°F) shows that an internal containment air temperature of 38°F results in a shell temperature of 30°F. Containment air temperature of 38°F or less during normal operation is not considered credible for the following reasons:

a. The containment mean air temperature is continuously monitored and is normally greater than 80°F. Should the air temperature drop significantly below this value, the motor operated gate valves in the fan coil cooling water supply line can be closed manually by positioning the control room switches. These valves can also be operated to reduce water flow by a local handwheel in case of motor failure.

b. The heat load from internal equipment within containment is approximately 4×10^6 Btu/hr, while the heat loss to the surroundings at -20°F is 4.7×10^5 Btu/hr. This suggests that the containment air could be heated at a rate of 3.8°F/min if the fan coil units are not removing heat. Thus the operating equipment provides an excellent heat source for assuring an adequate operating temperature of the containment vessel shell.

c. The containment air purging system heaters are designed to supply 75°F air at a rate of 33,000 CFM. The equivalent net heat input of this system is 1.3×10^6 Btu/hr with respect to the lowest permissible containment air temperature of 38°F. Since this heat input is almost a factor of 3 higher than the heat loss of 4.7×10^5 Btu/hr, the containment air purging system heaters are capable of maintaining containment air temperature above 38°F even if the fan coil units are in operation.

d. Normal maximum and minimum daily temperature at St. Cloud are about 20°F and 0°F in January. Therefore, the assumption of constant outside temperature at -20°F is conservative, in spite of a few hours of extreme minimum at -40°F being recorded in January.

In the event of a LOCA, the initial shell temperature will exceed 50°F, as specified under the definition and requirements for containment operability.

For any break which releases the stored energy of the reactor coolant, the air temperatures in the containment and in the annulus will remain above NDT considerations until long after the shield building ventilation system is in the full recirculation mode.

Beyond this time, the containment air temperature can readily be maintained above 50°F by not deliberately over-cooling the containment atmosphere with the post-accident heat removal systems. During eventual containment purge, the purge air heaters can be used if the outside air required warming.

Calculations based on -20°F outside air and 32°F cooling water temperatures, heat input only from the containment, 200 cfm of annulus inleakage, and a combined convective-radiative heat transfer coefficient of 1.5 B/hr ft°F yield the following values:

Annulus air temperature	25°F
Containment air temperature	50°F
Minimum penetration temperature (Fan-coil coolant inlet)	32°F

These values assure that the containment shell NDTT limit of 0°F is not approached or breached.

The lower portion of the shell which is embedded in concrete is also afforded significant protection by the Shield Building.

Of the entire lower shell, the portion in the transition zone between El. 694′-0″ and El. 708′-3″, as shown in Figures 1.1-15 and -16, is the most critical because of its minimum distance to the outside surface. Analysis indicated that the shell between these two elevations would be at 30°F or higher if the outside temperature is constant at -20°F and the inside temperature is 34°F or higher. As previously discussed, the inside temperature will always be maintained above 34°F. Therefore, the containment vessel temperature in this region could never be below 30°F in normal operation.

The reactor will not be operated unless the containment vessel air temperature exceeds 50°F. This assures a minimum 12°F margin above the highest critical temperature of 38°F.

12.2.2.5.1 Containment Vessel at Embedment Region

In its final configuration, the containment shell knuckle is embedded by the internal concrete to a point 2 feet 3 inches above the shell tangent line and externally to a point approximately 2 feet 9 inches below the tangent line. A transition zone exists in a region that extends from top of the internal concrete to some point on the shell below the temporary stiffeners. This condition is shown on Figure 12.2-3.

The joints between the steel and concrete were completely sealed so that no air or moisture can enter and cause corrosion. The sealant is a silicone rubber elastomeric sealing compound. This arrangement is such that it is not physically possible to monitor for corrosion of the steel containment without removing structural concrete. Therefore, corrosion protection is dependent on the integrity of resilient material caulking-seal at the concrete floor level. These seals were inspected for any physical disturbance after containment pressurization for leak testing and are visually inspected at each major refueling outage to monitor for age-deterioration of the sealant material as part of normal plant surveillance. No specific technical specification is felt to be required.

Embedment of the Reactor Containment Vessel knuckle in concrete produced bending stresses, resulting from thermal and pressure expansion, at the interface between the encased and non-encased portions of the shell. These bending stresses are minimized by providing a smooth transition between the part of the shell which is free to expand and the part which is fixed in concrete.

When the Reactor Containment Vessel is pressurized, in operational configuration, it exerts a pressure on the internal concrete in the knuckle region as the shell attempts to deform inward. Also, the vessel exerts a pressure on the concrete outside of the vessel where the elliptical head is tending to deform outward. These reactions on the concrete are due to the tendency of the elliptical head to become hemispherical in shape when pressurized.

The analysis of the concrete-steel shell interaction in the embedded zone is basically a flexibility method of analysis of the concrete-steel structural systems. The primary objective of the analysis is to determine stresses in the steel shell and the contact pressures on the concrete from the steel in the embedment region.

Three structures are used in the analysis: (1) internal concrete, (2) steel shell, (3) external concrete.

The model of the ellipsoidal bottom shell structure is divided into carefully selected segments to best represent the shape of the model. The restraint of the concrete on the steel shell is modeled using “Analogous Springs” and a distributed pressure. The “Analogous Springs” are represented by a flexibility model of both the internal and external concrete supplied by Pioneer Service & Engineering Company. This model is in the form of load per unit length of circumference per unit of concrete displacement and is given for various positions on a meridian line of the shell bottom. The pressures applied at the selected loading points of the shell bottom are equal to the algebraic sum of the internal pressure plus the reaction pressure of the concrete. The pressure distribution between loading points specified in the program is assumed to be linear.

Continuity and interaction are established by assuring that the deflections are equal at the points where concrete is in contact with the steel shell. Equations for the internal and external contact points are constructed and solved for redundants. A trial and error procedure is used for the solution since the equations corresponding to the redundants with negative contact pressure must be eliminated.

“Analysis of Shells of Revolution Subjected to Symmetrical and Non-Symmetrical Loads” by A. Kalnins (Reference 4), published in the September 1964 issue of the Journal of Applied Mechanics is used to develop the equations to be used in the following analyses; i.e., deflections, shears, and loads.

The pressure on the concrete on the inside and outside of the shell was found by applying a distributed pressure to the mathematical model. This pressure was then varied until the radial shear at the ends of the parts was negligible.

The stiffnesses of the concrete are used in the calculations as discontinuous, feasible loads at the end of each part. These discontinuous loads are in the form of a stiffness matrix. The stiffness matrix is as follows:

$$\begin{bmatrix} Q \\ N \\ M \end{bmatrix} = \begin{bmatrix} K_{1,1} & K_{1,2} & K_{1,3} \\ K_{2,1} & K_{2,2} & K_{2,3} \\ K_{3,1} & K_{3,2} & K_{3,3} \end{bmatrix} \begin{bmatrix} W \\ U \\ B \end{bmatrix} + \begin{bmatrix} C_1 \\ C_2 \\ C_3 \end{bmatrix}$$

The term $K_{1,1}$ of the stiffness matrix at the end of a part of the model is set equal to the elastic modulus of the concrete foundation at location K times the distance between the midpoints of the two adjoining parts. All other terms in this 3 x 3 stiffness matrix are set equal to zero. Therefore, the deformations at the point in question are independent of the loads at any other point.

The term, C_1, of the constant vector is computed such that the applied shear is equal to zero when the deflection at the point is equal to the assumed deflection. The assumed deflection is found by dividing an assumed pressure on the concrete by the elastic modulus of the concrete. The term C_1 is used to minimize the shear being applied to the shell in order to speed up the convergence of the solution. The terms C_2, C_3 are set equal to zero. The above matrix reduces to the following expression:

$$Q = K_{1,1} \, x \, W + C_1$$

The pressure which is applied to the shell is equal to the algebraic sum of the internal pressure plus the reaction pressure on the concrete. The pressure distribution between the ends of the parts is assumed to be linear. The resulting deflections of the shell are checked at all output points to verify compatibility between the applied pressures and the resulting deflections.

The output of the vessel shell analysis program includes pressure forces that must be provided by internal and external concrete. These pressure forces maintain knuckle stability and the necessary external support when the vessel is pressurized.

The pressure distribution and C_1 are varied until the shear being applied at the end of all parts is equal to zero. The change in pressure from one trial to the next is found by dividing the difference in transverse shear between two adjoining parts by the distance between the midpoints of these parts. If the shear difference is directed outward from the axis of revolution, the resulting change in pressure is taken as a positive internal pressure.

The pressure loads then become the input of a separate Kalnins program. A model developed by Pioneer Service and Engineering Company (PS&E) was used for the analysis of the internal concrete back-up shell using this input. The loading for the concrete model utilized the output of the steel shell analysis. The program outputs of the PS&E concrete model are moments, shears, and membrane forces in the concrete. The internal concrete shell is designed for these moments, shears, and membrane forces in accordance with the rules of the ACI Code.

In summary, the compatibility of deformations of the structures is amply demonstrated when the solution of the flexibility model is carried to the point where the deflections of structures in contact are equal. This has been done and the final internal and external contact pressures of the concrete on the steel are in equilibrium with the structures as a free body for axisymmetric loads. There are no non-axisymmetric loads considered acting on the bottom in this analysis.

12.2.2.5.2 Concrete and Steel Interface Design

The containment vessel plate is sandwiched between internal and external concrete structures and is designed to act as an independent structural steel shell member in its function to resist containment pressure. No-leak monitoring channels are provided at the welds, since the welds and plate are designed as an integral structural system exclusive of adjacent concrete structures. No leaks are credible once the leak-tight integrity of the bottom welds has been established.

Prior to placing any concrete either inside or outside of the vessel bottom, the following measures were taken to detect any possible leakage through the bottom plates and welded seams:

a. All butt weld seams were 100 percent radiographed, inspected per ASME Section VIII and approved procedures using double film exposure.

b. Soap film test of all weld seams at 5 psig (Pneumatic).

c. Overload pressure test for one hour at 51.8 psig (Pneumatic).

d. Soap film test of all weld seams at 46 psig (Pneumatic).

e. 72 hour leak rate test at 46 psig (Pneumatic).

The final stress report demonstrated that the Reactor Containment Vessel, as specified, analyzed, and designed, meets all applicable requirements of the ASME Code.

As a further verification of the design, a second overpressure test (1.25 times design pressure), not required by ASME Code, was performed at a pressure of 51.8 psig with the temporary vessel stiffeners removed and the internal concrete support system in place, but before fuel is loaded. The maximum pressure was maintained only long enough to verify the pressure level. The final preoperational integrated leak rate test provides yet another check on the leak tight integrity of the vessel.

A chemically inert concrete grout is tightly bonded to the outside of the vessel shell and the supporting concrete. The chemical grout used is a two- component epoxy polysulfide material of low viscosity with a monomolecular bond attraction. It consists of epoxy resin, organic amine, and polysulfide. It contains no solvent, does not shrink, and is chemically inert to concrete and steel. The grout has no potential for corrosive action on the steel plates and seam welds.

In the presence of the tight dead weight pressure-bonded contact with age cured dry internal concrete, and the tight bonded contact with a chemically inert concrete grout externally, it is implausible to postulate conditions that would be conducive to corrosion of the vessel bottom plates and seam welds. Therefore, no special corrosion detection systems were provided to inspect the vessel bottom once it had been covered with concrete.

12.2.2.5.3 Non Axisymmetric Loading Due to Concrete Shielding at Fuel Transfer Tube

An area 43.5 ft high and about 24 ft wide of the inside of the containment vessel is covered by interior concrete walls. This area is referred to as the “cold spot”.

The analysis discussed in this section was based on a containment pressure curve shown in USAR, Appendix G, Figure G.3-20 (originally FSAR, Figure 14.3.23). Subsequent containment pressure and temperature analyses (some performed prior to initial plant licensing - FSAR, Section 14C) showed that the peak temperature and pressure could occur much later than that predicted in Figure G.3-20. (Note that G.3-20 is strictly historical). The later occurrence of the peak temperature and pressure results in more heat being transferred to the containment shell; which increases the temperature difference between the heated shell and the “cold spot”. However, the analysis of the stresses in the “cold spot” in this section was not updated to reflect the subsequent containment pressure and temperature analyses. An assessment was performed of the affect of the subsequent containment pressure and temperature curves (Appendix K, Figure K-18) and determined that the stresses due to the “cold spot” would not exceed the acceptance criteria (Reference 72). Thus, the containment is considered operable given the results from the analysis of record. As the analysis was not re-run for this assessment, the discussion of the analysis in this section is not being replaced or revised.

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The design basis for the steel containment vessel is that after a LOCA the internal pressure will peak at 46 psi at about 10 seconds, and the internal temperature will peak at 268°F, also at about 10 seconds. After that, both pressure and internal temperature will decline (Based on Appendix G, Figure G.3-20).

During the first 10 seconds after a LOCA the temperature of the inner surface of the containment vessel shell rises from 120°F to 155°F, and the outer surface rises from 120°F to 129°F. Thereafter, the shell temperature continues to rise to 165°F at the inner surface and 155°F at the outer surface. This increase does not occur in the portion of the shell protected by the internal concrete thus producing the “cold spot” which results in additional stresses in the steel shell. These stresses are classified as Thermal Stresses.

The area behind the “cold spot” is assumed to not be subject to the 46 psi internal pressure. Thus, the pressure around the circumference of the steel shell is unbalanced giving rise to local pressure stresses.

Kalnins’ Static Shell Program was used to determine the effects of the missing temperature rise and the missing pressure increase at the “cold spot”. For each investigation, 21 Fourier harmonics were combined to represent the sudden changes at the edges of the “cold spot”.

Additional stresses due to missing temperature at the “cold spot”, after 10 seconds, were obtained and are given in Table 12.2-19.

The circumferential stresses are the maximum stresses at the three elevations listed.

In determining the effect of the missing pressure, cognizance was taken of the fact that the increase in pressure and temperature of the steel shell after a LOCA would cause an increase in diameter. This would result in a theoretical finite gap between the steel shell and the exterior face of the concrete walls. However, the unbalance in forces around the circumference of the shell, due to missing pressure against the portion of the shell protected by the concrete wall, would cause the steel shell to press against the concrete and thus prevent the gap from opening. At the maximum internal pressure of 46 psi, a pressure differential of 3 psi is required to close the gap. Thus, the steel shell will then press against the concrete with a pressure of 43 psi. The additional stresses in the steel shell were calculated on this basis and are given in Table 12.2-20.

The additional stresses given above must be added to the stresses which would exist if no “cold spot” were present. These stresses are summarized in Table 12.2-21.

The total combined stresses in the containment vessel steel shell due to the presence of the ”cold spot” are summarized in Table 12.2-22. These stresses occur 10 seconds after the start of LOCA. The stress intensity for each loading condition is also shown in this table. The stress intensity is calculated in accordance with Paragraph N-413 of Section III of the ASME Boiler and Pressure Vessel Code (1968 Edition).

The allowable stresses in the containment vessel steel shell were established by the criteria listed in Table 12.2-17. For the Operating Basis Earthquake condition, this table refers to Figure N-414 of Section III of the ASME Boiler and Pressure Vessel Code (1968 Edition). The limits of stress intensities in Figure N-414 are based upon the allowable stress intensity Sm. For Class B vessels, this allowable stress intensity is obtained from Section VIII of the ASME Code, and is 17,500 psi for A-516 Grade 70 steel. This allowable stress intensity is applicable to general membrane stress intensities when local membrane and bending stresses are not present. When the latter stresses are present, the allowable stress intensity is increased to 1.5Sm or 26,250 psi. When secondary stresses due to differential thermal expansion are included, the allowable stress intensity is increased to 3.0Sm or 52,500 psi.

Allowable stress criteria for the Design Basis Earthquake are given in Table 12.2-17 under loading condition 2. The allowable stress intensities computed in accordance with this criteria are:

General membrane	= 1.16 (Sm)	= 20,300 psi
General membrane plus bending	= 1.16 x (1.5 Sm)	= 30,450 psi
All stresses including secondary stresses	= 3 Sm	= 52,500 psi

The allowable stresses intensities are also shown in Table 12.2-21.

It can be determined from an examination of Table 12.2-22 that the presence of the “cold spot” does not result in a condition, in which actual stress intensity exceeds the allowable stress intensity.

After the first ten seconds following a LOCA the internal pressure drops and the temperature differential between the two surfaces of the steel shell also declines, but the average temperature of the shell increases another 18°F to 160°F. The change in stresses due to the first two effects is greater than that caused by the third effect. The net result is a decline in total stresses after about ten seconds.

It has previously been pointed out that at ten seconds after a LOCA, the steel shell exerts a 43 psi pressure against the outer surface of the concrete wall. At the same time there is an internal pressure of 46 psi against the inner surface of the concrete wall. The 3 psi pressure differential is carried by the concrete wall.

During the first ten seconds following a LOCA the pressures do not increase linearly, so the pressure differential could, briefly, exceed 3 psi during this interval. For instance, at one second the interior pressure is 12 psi. This 12 psi pressure was used to calculate stresses in the concrete wall on the conservative assumption that the steel shell had not had time to react and press against the concrete.

STRESS computer program was used to calculate moments and shears, and Pioneer Service & Engineering Co. program S-020 was used to determine the required reinforcing steel. The amount of reinforcing steel thus determined, did not exceed 35 percent of the steel required by other loading conditions. Thus, the internal concrete structure will not be overstressed by the loads caused by the pressure of the “cold spot”.

In conclusion, it has been demonstrated that all stress combinations which include those computed on the basis of non-axisymmetric loading at the “cold spot” meet the criteria for allowable stress for the steel containment vessel and the interior concrete construction.

12.2.2.5.4 Testing of the Reactor Containment Vessel on Temporary Supports

The Vessel was erected on a temporary support system. To fulfill the requirements of the ASME Code and provide a basis for acceptance of the Vessel, the acceptance overpressure test was made while the Vessel was supported on the temporary support system. To provide additional safety margin for those engaged in the erection and testing of the vessel, internal stiffeners were provided as a precautionary measure.

The temporary internal stiffeners consisted of four flat bars that were attached by fillet welds to the inside surface of the shell in the knuckle area. These bars are circumferential rings spaced about two feet apart. The stiffener bars are shown in Figure 12.2-3. These stiffener bars were installed to provide lateral stability of the knuckle plate during pressure testing. This was necessary because the knuckle plate is subject to circumferential compression stress when it is pressurized for testing. After the vessel was acceptance tested, the temporary stiffeners were removed and the required lateral support is provided by internal concrete structures.

The vessel was temporarily supported on fifteen laterally braced pipe columns shaped to the contour of the vessel. The columns were welded to the shell plate immediately below the tangent line at the knuckle and also to the horizontal web of an external horizontal tee member. This horizontal tee ring-girder developed the lateral stiffness of the temporary bracing. These temporary supports are shown in Figure 12.2-4.

The temporary structure supported the weight of the vessel (2300 tons) and specified construction loads. In addition, it resisted lateral loads, such as wind, which were imposed during construction. The vessel loads were then transferred to the grout support. The temporary legs, bracing, and external tee member have been removed by cutting to within 1/4″ of vessel face and then ground smooth.

12.2.2.5.5 Wind Analysis

The Reactor Containment Vessel and Penetrations associated with primary containment were completely enclosed by the Shield Building and are therefore not directly subjected to the forces and effects of wind and tornadoes.

12.2.2.5.6 Piping Penetrations Analysis

Piping Penetrations Analysis is discussed in Section 5.2.2.2.2 under Piping Penetrations.

12.2.2.5.7 Seismic Analysis

The seismic analysis of the Containment System, critical appurtenances, structures, and equipment were analyzed based on the ground response acceleration spectra, the building response spectra (Reference 6) and the floor response acceleration spectra (Reference 8). Figure 12.2-5 shows the location of seismic instrumentation.

12.2.2.5.7.1 Building Response Spectra

A building response spectra analysis (Reference 6) was performed based on Operational Basis Earthquake (0.06g) and Design Basis Earthquake (0.12g). The analysis developed values for maximum translational accelerations, displacements, shears and moments and maximum torsional accelerations, moments and rotations of the building structures.

The mathematical model considered the three major structures (Reactor, Auxiliary and Turbine Building Structures) in a combined idealized three-dimensional model with 63 degrees of translational freedom for symmetrical elements, and both translational and torsional rotation for unsymmetrical and irregular elements.

12.2.2.5.7.2 Floor Response Acceleration

The floor response acceleration spectra analysis (Reference 8) is based on Operational Basis Earthquake (0.06g) and Design Basis Earthquake (0.12g). The analysis developed the generated acceleration time-history response spectra at mass points designated for the seismic analysis of critical equipment and piping located throughout the structural complex. The mathematical model utilized for this analysis is the same as used for the building response spectra analysis.

12.2.2.5.8 Air Lock Seismic Analysis

The specification for the Reactor Containment Vessel requires that the design for earthquake include the effects of the air locks vibrating as an independent system. The Reactor Containment Vessel contractor, Chicago Bridge & Iron Company, developed a design method for analysis of the air lock system under seismic loading. Their design method has been reviewed and approved by an independent seismic consultant, John A. Blume & Associates. The calculations have been performed according to this method, and the results show that the stresses in the air locks do not exceed the allowable stresses given in Table 12.2-17.

In this method of analysis, the horizontal earthquake is assumed to act in one of two directions as shown in Figure 12.2-34. Case I results in seismic forces and moments from the air lock being applied to the Containment Vessel in the circumferential direction. Case II will result in a radial thrust being applied to the Containment Vessel.

12.2.2.5.8.1 Dynamic Constants for Air Locks

The first step in the dynamic analysis of an air lock is to determine its dynamic constants. For this determination, the air lock and its associated reinforcing insert plate are considered as a single rigid system. Since the airlocks are completely separated from the concrete Shield Building and adjacent concrete structures by an air space, there is no interaction between airlock structures and concrete structures. The procedure used to calculate these constants for each case is shown below.

Case I - Vertical and Circumferential Directions

The spring constants for vertical and circumferential directions are determined by applying a unit moment (1000 in-lbs) at the shell and determining (Reference 9).

$$K = \frac{M}{\Theta}$$

Where: K = Spring constant of shell

$$\omega = \left[\frac{K}{I}\right]1/2$$

ω = Angular frequency of lock

I = Mass moment of inertia of lock about point of support on shell

$$T = \frac{2\Pi}{\omega}$$

M = Unit moment at shell

Θ = Rotation at shell to insert junction

T = Fundamental period of lock

Case II - Radial Direction

The spring constant for the radial direction is found by applying a unit thrust (1000 lbs) at the shell and determining w (Reference 10):

$$K = \frac{P}{w}$$

Where: W = Weight of lock plus insert

g = 386.4 (in/sec^2)

P = Unit Load

$$T = 2\Pi\left[\frac{W}{Kg}\right]1/2$$

w = Deflection at shell to insert junction

12.2.2.5.8.2 Seismic Forces on Air Locks

The vibration driving force on the air locks was determined from accelerations derived from the response acceleration spectra prepared by the seismic consultants, John A. Blume & Associates (Reference 8). Using the fundamental period of the air lock (T) and 1% damping (as recommended by John A. Blume & Associates), the acceleration (in percent of gravity) is obtained from the Blume report. The air lock dead loads were then multiplied by this acceleration to obtain the seismic forces acting on the air lock.

12.2.2.5.8.3 Seismic Stresses

Stresses in the shell due to the air locks vibrating as an independent system from a horizontal and vertical earthquake were determined by the use of Welding Research Council Bulletin 107.

Stresses were limited to the allowable stress criteria as set forth in Table 12.2-17.

A horizontal earthquake acting perpendicular to the air lock (Case I) results in a circumferential shear and moment being applied to the shell. An earthquake acting parallel to the air lock (Case II) subjects the shell to a radial thrust.

Stresses were checked at three locations: at the neck-to-insert junction, at the insert-to-shell junction, and at a distance of 0.5 $(Rt)^{1/2}$ from any local stress area. (R = radius of curvature and t = thickness).

Stresses in the reinforcing insert and in the shell due to the applied earthquake loads were calculated. Pressure stresses were added directly to the earthquake stresses, an equivalent stress intensity was calculated (per maximum shear theory) and compared to allowable values in Table 12.2-17.

The applicable ASME Section III allowable stress intensities for stresses due to the pressure and applied loads at the containment vessel shell to airlock nozzle junction (See Table 12.2-17) are calculated as follows:

<u>For Operating Basis Earthquake (OBE)</u>

Membrane Stress:

PL = (1.5) (17500) = 26250 psi

Surface Stress:

PL + Q = (3) (17500) = 52500 psi

For Design Basis Earthquake (DBE)

Membrane Stress:

PL = (1.16) (1.5) (17500) = 30450 psi

PL + Q = (3) (17500) = 52500 psi

A summary of the membrane and surface stresses at the connecting weld of the containment vessel shell plate to the airlock barrel nozzle plate for the two earthquake loading conditions is given in Table 12.2-23.

The designs are based on the Operational Basis Earthquake and are reviewed for the case of Design Basis Earthquake to assure that the stress limits for this loading were not exceeded.

12.2.2.5.8.4 Vertical Earthquake

A vertical earthquake acceleration applied to the vessel and air locks was assumed to act simultaneously with the horizontal earthquake.

12.2.2.5.9 Postulated Pipe Failure Analysis Outside of Containment

The analysis as discussed in Appendix I presents the final design analysis of postulated pipe failure outside of containment structure, including the rupture of a main steam or feedwater line.

The design analysis provided information regarding the final design of modifications to the high energy lines outside of containment. The final design incorporates an encapsulation sleeve over the design basis break of the main steam pipe that occurs in the auxiliary building. The sleeve restricts the steam discharge rate from the postulated design basis break.

The final design also incorporates redundant isolation dampers in each ventilation duct that penetrates those compartments designated as steam exclusion zones. These zones include the Class I corridor in the turbine building and the compartment in the auxiliary building that contains the major components of the residual heat removal system, the component cooling system, and equipment required for this postulated accident.

12.2.2.5.10 Seismic Analysis for As-Built Safety Related Piping Systems

Pursuant to the requirements of I.E. Bulletin No. 79-14, a seismic re-evaluation of safety-related piping systems was performed for Unit 1 and Unit 2. The re-evaluation program consisted of physical and analytical inspection of accessible piping. The physical inspection included the actual verification of as-built drawings by direct measurement and comparison. Inspection elements involved were piping geometry, support/restraint design and location, support/restraint function/clearances, valve and valve operator locations, and pipe attachment details.

The analytical inspection portion involves the verification of the input information into the seismic analysis. Included in this review were the valve weights, any offset center of gravity concerns, and materials of construction from the quality assurance records. All accessible piping in the plant which falls under the scope of this bulletin were inspected physically and analytically. Discrepancies found during the course of the program were reported to the NRC and subsequently corrected. Where inconsistencies exist with the piping geometry used as inputs, re-analyses were made to ensure no unacceptable stress levels exist.

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12.2.3 Shield Building Design

12.2.3.1 Design Loadings

12.2.3.1.1 Dead Load

Dead load consists of the dead weight of the Shield Building, the Reactor Containment Vessel, grout under the Containment Vessel, the foundation slab, and the weight of concrete, structural steel, equipment, and miscellaneous building items within the Reactor Containment Vessel.

The basis for calculation of weights is given in Section 12.2.2.

12.2.3.1.2 Design Basis Accident (DBA)

The DBA load is determined by analysis of the pressure and temperature transients in the annulus during a Design Basis Accident (See Section 14).

12.2.3.1.3 Live Load

Live loads consist of snow loads on the dome applied uniformly to the top surface of the dome at 50 pounds per horizontal square foot (PSF).

12.2.3.1.4 Wind Load

Wind loading for the Shield Building is based on Figure 1 (b) ASCE Paper 3269, "Wind Forces on Structures," using the fastest wind speed for a 100-year period of recurrence. This results in 100-mph basic wind at 30 feet above grade. In addition, this paper was used to determine shape factors, gust factors and variation of wind velocity with height.

12.2.3.1.5 Tornado Load

The structure was analyzed for tornado loading (not coincident with accident or earthquake) on the following basis:

a. The Shield Building is designed for an internal pressure of 3 psi acting outward and uniformly over the entire inside surface of the building;

b. Lateral force on the Shield Building assumed as the force caused by a "funnel" of wind having a peripheral tangential velocity of 300 mph and a forward progression of 60 mph.

The design tornado that is assumed, corresponds to a large funnel of arbitrary radius with a band of maximum velocity wind (300 mph) which is at least 150 feet wide and which extends from the ground surface up at least 200 feet.

The forces induced in the Shield Building by the external pressure of the above hypothetical tornado are considered in two cases. In the first case the structure is considered as an annular cantilever beam loaded with maximum wind pressure over the entire horizontally projected area of the structure. This case gives the maximum overturning moments, membrane shear and axial stresses.

In the second case, it is assumed that the exterior pressure at any horizontal section varies from a maximum at the sub-wind centerline of the building to zero at each end of the diameter which is perpendicular to this centerline. This pressure pattern is assumed to be the same at all elevations. This case gives the maximum ring moments and shears.

12.2.3.1.6 Uplift Due to Buoyant Forces

Uplift forces which are created by the displacement of ground water by the structure are accounted for in the design of the structure. The uplift force associated with the probable maximum flood (703.6 ft) is equal to the volume of water displaced multiplied by the unit weight of water. This upward force is 6640 Kips. The dead weight of the structure, (Shield Building) is 93,700 Kips. This results in a factor of safety of 14 against uplift. As shown in Figures 1.1-15 and -16 the lower portion of the Shield Building is primarily massive concrete and, therefore, stresses in structure due to water pressure are insignificant.

For other Class I structures, the factors of safety against uplift due to buoyant forces created with high water at elevation 703′-6″ are listed in Table 12.2-24.

12.2.3.1.7 Earthquake Loads

Seismic loads were computed using the following:

a. Operational Basis Earthquake seismic ground acceleration of 0.06g. (See Appendix E.)

b. Design Basis Earthquake seismic ground acceleration of 0.12g.

A vertical component of 2/3 of the horizontal ground acceleration is applied simultaneously with the horizontal acceleration. The plots of the seismic response spectra are shown in Appendix E. The classification of plant structure and equipment and the applicable damping factors are shown in Table 12.2-1 and Table 12.2-8 respectively. The general procedure for seismic analysis is treated in Section 12.2.2.1.5.

A computer program based on methods developed by K. Hetenyi was used to determine the stresses in a section through the two Shield Buildings and the Spent Fuel Pool which are supported on a common foundation.

Eight different cases were investigated: -

a. Shield Buildings out of phase, both accelerating away from the Spent Fuel Pool (DL+LL+OBE)

b. Same as (a), except (DL + LL + DBE)

c. Shield Buildings out of phase, both accelerating toward the Spent Fuel Pool (DL+LL+OBE)

d. Same as (c), except (DL+LL+DBE)

e. Shield Buildings in phase, one accelerating away from and one toward the Spent Fuel Pool (DL+LL+OBE)

f. Same as (e), except (DL+LL+DBE)

g. Shield Buildings in phase, one accelerating away from the Spent Fuel Pool and the other Shield Building and the Spent Fuel Pool accelerating toward one another (DL+LL+OBE)

h. Same as (g), except (DL+LL+DBE)

In all cases the stresses were less than those listed in Tables 12.2-25, 12.2- 26, 12.2-5 and 12.2-6 for the appropriate cases.

12.2.3.1.8 External Missiles

The design tornado missile was chosen to be equivalent to an airborne 4″ x 12″ x 12′-0″ long wood plank traveling at 300 mph. Other possible sources of tornado missiles were evaluated as reported in Section 12.2.1, but none were as destructive as the above wood plank.

The turbine missile is discussed in Section 12.2.7.

12.2.3.2 Codes

12.2.3.2.1 Design Codes

Concrete structures are designed in accordance with the ACI Code 318-63 "Building Code Requirements for Reinforced Concrete." The specifications for the construction of the Shield Buildings (ACI 347) which require a tolerance of ±1/2 inch in the thickness of the wall, and that the total deviation of any point on the interior surface of the cylindrical wall should not vary more than one inch in 50 ft of height.

The as-built dimensions of the Unit No. 1 Shield Building did not conform to the specified requirements. A complete report entitled "Slip Form Construction of Unit No. 1 Reactor Shield Building Including Explanations of Construction Details and a Structural Analysis", dated, December 4th, 1968, has been submitted to the AEC. This report gives actual dimensions of the completed structure, and demonstrates that the dimensional variations will not result in an overstress condition and will not impair the functional safety of the system.

The as-built dimensions for Unit No. 2 Shield Building indicate that there is a slight dimensional variation from the specified tolerance, but not as much as occurred in the Unit No. 1 building. Therefore, it can be concluded that there will be no impairment of the functional safety of this building.

Structural steel is designed in accordance with the "AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings."

Welding is in accordance with the ASME Boiler and Pressure Vessel Code and AWS "Standard Code for Arc and Gas Welding in Building Construction."

In addition to the above, the Shield Building is designed and constructed in accordance with applicable state and local building code requirements.

12.2.3.2.2 Material

Specifications and working drawings for materials and their installation are of such scope and detail that the quality of work assured the desired integrity to the Shield Building as constructed.

Basic specifications for these materials include the following:

a. Concrete

   All concrete work is in accordance with ACI 318-63 "Building Code Requirements for Reinforced Concrete" and ACI 301 "Specification for Structural Concrete for Buildings."

The concrete was tested to assure a minimum compressive strength of 4000 psi at 28 days. Where the rate of heat of hydration was not a factor, the type of cement used was Portland cement Type I conforming to Specification ASTM C150. Type II cement is used for its lower heat of hydration and a slower rate of heat generation.

The testing of concrete was accomplished by Twin Cities Testing and Engineering Laboratory, St. Paul, Minnesota, whose primary business is to perform such testing and show proof of the required knowledge and facilities to perform the specified tests and report accurate results. Testing of aggregates and cement, design of concrete mixes, taking of samples and making of required field tests were performed by the testing laboratory.

When concrete was bucketed and/or conveyed by belt, slump and compression cylinder samples were taken at the truck discharge into the bucket or conveyor. When concrete was pumped, samples were taken at the truck discharge until July, 1970, when this was changed to the point of pump discharge. A comparison of the test results indicated no changes, that would be attributed to variations in sampling locations for all design mixes used.

Standards and specification for concrete materials, testing and construction methods are as follows:

ACI 301	Specifications for Structural Concrete for Buildings
ACI 306	Recommended Practice for Winter Concreting
ACI 347	Recommended Practice for Concrete Formwork
ACI 605	Recommended Practice for Hot Weather Concreting
ACI 614	Recommended Practice for Measuring, Mixing and Placing Concrete
ASTM C31	Making and Curing Concrete Compression and Flexure Test Specimens in the Field
ASTM C33	Specifications for Concrete Aggregates
ASTM C39	Test for Compressive Strength of Molded Concrete Cylinders
ASTM C87	Test for Effect of Organic Impurities in Fine Aggregate on Strength of Mortar
ASTM C98	Specification for Ready-Mixed Concrete
ASTM C143	Test for Slump of Concrete

ASTM C150 Specification for Portland Cement

ASTM C172 Method of Sampling Fresh Concrete

ASTM C175 Specification for Air-Entraining Portland Cement

ASTM C227 Test for Potential Alkali Reactivity of Cement Aggregate Combinations

ASTM C231 Test for Air Content of Freshly Mixed Concrete by the Pressure Method

ASTM C260 Specification for Air-Entraining Admixtures for Concrete

ASTM C289 Test for Potential Reactivity of Aggregates

ASTM C295 Petrographic Examination

ASTM C350 Fly Ash for use as a Concrete Admixture

b. Reinforcement

All reinforcing is new billet steel and specified as follows:

Type & ASTM Spec No.	Grade Designation	Minimum Tensile - PSI	Minimum Yield - PSI	Minimum Elongation in 8² Spec
A-15	Intermediate	70,000	40,000	7 - 12% *
A-408	Intermediate	70,000	40,000	10%
A-432	------	90,000	60,000	7%

ASTM A305 Specification for Deformations of Deformed Steel Bars for Concrete Reinforcement

AWS D12.1 Recommended Practices for Welding Reinforcing Steel, Metal Inserts and Connections in Reinforced Concrete Construction

* Varies with size of bar

All reinforcing steel is shipped to the job site in bundles bearing a tag identifying its size, grade and code number keyed in heat numbers. This information was verified by certified mill test reports which accompany each shipment of reinforcing steel.

All reinforcing steel bars are clearly identified with markings legibly rolled into the surface of each bar at the producers plant showing the point of origin, size, designation and type of steel. High-strength steel bars are further identified with the minimum yield strength rolled into the surface of each bar.

Bond and anchorage provisions of reinforcing bars are in accordance with Sections 918, 919, 1300 and 1301 of the ACI Building Code Requirements for Reinforced Concrete, (ACI 318-63). Allowable bond stresses for deformed bars per ASTM-A615 are:

For Tension Bars:

$$\text{Top bars} - \frac{3.4\sqrt{f_c}}{D} \quad \text{but not more than 250 psi.}$$

$$\text{Bars other than top bars} - \frac{4.8\sqrt{f_c}}{D} \quad \text{but not more than 500 psi.}$$

For Compression Bars:

$$\frac{6.5\sqrt{f_c}}{D} \quad \text{but not more than 400 psi.}$$

Where loading conditions are such that allowable stresses are 1-1/2 times ACI allowable of .75 f′c as indicated in Tables 12.2-26 and 12.2-5, allowable bond stresses have been increased by 1.5 and 1.67 for the above two conditions, respectively.

In addition, field tests of the concrete indicated that the actual compressive strength of the concrete is considerably higher than the values of f′c used for design purposes.

All reinforcing steel was tested in accordance with ASTM Specifications. Tests included one tension and bend test per heat of mill shipment, whichever was less, for each diameter bar.

Full size bar samples were used for user tests of reinforcing bars. Yield strengths were in accordance with ASTM-A615.

The Quality Control Engineer followed construction to assure that steel bars as specified on the drawings were placed in their proper location and were of the designated size and strength.

The fabricator was required to furnish assurance that the steel as detailed, fabricated and shipped to the job site, bearing identification tags, was the same as that received from the mill.

Cadweld splices were not used in Class I structures except for repairs made to reinforcing bars approved on a case by case basis by the architect-engineer. Cadwelds were made and inspected in accordance with the manufacturers recommended procedures. Operators were qualified. Welding electrodes used in the containment vessel were controlled in accordance with the manufacturers approved procedure which conforms to the requirements for Section III Class B vessels.

Reinforcing steel bars are generally lap-spliced except where the design indicates that welded splices are structurally required, because the lap splice cannot adequately meet the joint requirements within practical limits or where welded splices are economically advantageous. Welded splices are made by fusion welding and only A-15 reinforcing bars are used, no 14S or 18S bars were used. Welded splices conform to Figure 3 (b) and (c) of AWS D 12.1- 61, Welding Reinforcing Steel Metal Inserts and Connections in Reinforced Concrete Construction and provide weld throat and angle iron sections that exceed the cross section of the mating bars. Qualification tests were made and in each case the strength of the splice exceeded the bar strength. Approximately 97% of these splices were magnetic particle inspected to ensure absence of defects. None were observed.

Whenever the integrity of structure is dependent upon such welding, the weld splicing of A-15 bars was as follows:

a. A fully prequalified, written welding procedure was used.

b. All welders were prequalified by tests;

c. The chemistry of the bars to be welded was determined, and no bar with a carbon content greater than 0.35% was welded;

d. Welded splices in adjacent bars were staggered at least 24 inches;

e. An extra ring of bars was added and was in addition to those required by the design;

f. Not less than one in twenty-five welds were tested radiographically. Each weld was inspected visually.

Shield Building wall reinforcement in each direction is governed by the structural requirements of tornado pressure loadings, wall temperature gradients, and missile loading. The structural requirements are far in excess of the minimum percentages of reinforcement required by ACI-318. The design indicates that the minimum percentage of reinforcement in any case is not less than 0.4%.

Figures 12.2-10 and 12.2-11 show the typical arrangement of the reinforcing steel patterns for the base slab and at the lower portion of the wall. The reinforcement in the discontinuity zone of the wall with the base is shown on Figure 12.2-10.

Figures 12.2-12 and 12.2-13 show the typical arrangement of the reinforcing steel pattern for the dome and the upper portion of the wall. The reinforcement in the discontinuity zone of the wall with the dome is shown on Figure 12.2-12.

Figure 12.2-14 shows the typical arrangement of the reinforcing steel pattern at a typical large opening. In this illustration the opening was provided for an airlock. It is to be noted that where the general pattern of wall reinforcement has been interrupted by an opening, additional perimeter beam reinforcing has been added.

Since the Shield Building is not subjected to any significant internal pressure, reinforcement bar anchorage will generally not occur in tension zones.

12.2.3.3 Design Stress Criteria

The reinforced concrete and structural steel of the Shield Building and foundation were designed by the working stress method and are based on allowable stresses as set forth in Table 12.2-25.

Earthquake stresses were added linearly and directly to stresses caused by the Design Basis Accident, snow, wind, dead loads, and the appropriate operating loads, to obtain the total stresses. These total stresses are within the maximum stress limits allowed by the applicable design criteria.

The summation of loads as stated in Conditions 1 and 2 of Table 12.2-25 provided the design basis for the Shield Building.

The summation of loads as stated in Condition 3 provides the design basis to maintain the integrity of the Shield Building so that a proper shutdown can be made during the ground motion having twice the intensity of the Operating Basis Earthquake. The design was reviewed to assure that resulting deflections or distortions did not prevent the proper functioning of the structure or piece of equipment and would not endanger adjacent structures or components.

Adequate reinforcing is placed in the concrete walls and dome to control cracking due to concrete shrinkage and temperature gradients.

The summation of loads as stated in Condition 4 of Table 12.2-25 provided the basis for a design review to assure that the Shield Building will suffer no loss of shielding or containment function due to a 300-mph design tornado.

The allowable stress values shown in Table 12.2-25 assure an elastic behavior of steel reinforcement during a Design Basis Earthquake, thus minimizing the cracking of concrete and the impairment of leak-tight integrity. The allowable stresses in reinforced concrete for loading Condition 3 indicated in Tables 12.2-25, 12.2-26 and 12.2-29 are all based on working stress design.

It is demonstrated by the stress criteria of Table 12.2-26 that during the maximum conditions of earthquake loading, the allowable stress values of Table 12.2-25 provide adequate margins within the elastic range of the material to assure the elastic behavior of the structure.

12.2.3.4 Design Review and Analysis

12.2.3.4.1 Concrete Cracks in Walls of Shield Building During Post-Earthquake Conditions

It is expected that only negligible amounts of leakage could occur through cracks during post-earthquake conditions. Accordingly, leakage due to an earthquake has been indicated as “negligible” in the Table 5.3-1. This is primarily a consequence of the combination of low shear stresses and the predominant effect of the dead load of the structure on the moment and shear stresses produced by earthquakes.

The maximum expected tangential shear stresses on a horizontal cross section of the Shield Building wall at the base of the structure during a Design Basis Earthquake will be of the order of 90 psi. This shear stress will vary from zero to a maximum and will be distributed over the cross section as outlined in Section 12.2.3.4.5.

The ratio of the structure dead load to the Design Basis Earthquake moment uplift forces is in the order of 1:0.95, with a dead load stress of about 220 psi and a maximum moment uplift stress of 210 psi at the outer fibers of the cross section where the shear stresses are zero.

The structure dead load acts as a prestress on the structural system and modifies the trajectories of tensile and compressive principal stresses from their characteristic curved lines to near straight orthogonal lines.

For example, where the horizontal tangential shear stress is zero, the principal stress trajectory is vertical and there is no normal horizontal stress and associated cracking of concrete. This condition occurs in the walls at the extremities of a diameter normal to the neutral axis. A second limiting condition occurs where the horizontal tangential stress is a maximum (90 psi) at the extremities of a diameter coincident with the neutral axis. The vertical dead load stress (220 psi) combines with the shear stress to generate a principal stress trajectory that is oriented near vertical (approximately 20 degrees to the vertical) with a normal principal tensile stress of 30 psi.

It is evident that earthquake conditions cannot produce sufficient horizontal forces to initiate and open vertical shear cracks to any appreciable degree. It is further evident that whatever cracks might form would only open and be exposed to in-leakage for an average time length of only one-half of the duration of an earthquake, since the elastic action of the structural system alternately opens and closes stress cracks during earthquake stress cycling and finally remain closed after the earthquake has subsided. In addition, the stresses due to an earthquake vary from a maximum at the base of the structure to zero at the very top, thereby further minimizing any leakage. Finally, it can be concluded that earthquake forces will not provide any stresses of appreciable magnitudes that might contribute to any significant cracking and associated in leakage to the Shield Building ventilation system.

12.2.3.4.2 Concrete Cracks and Leakage at the Springline of the Shield Building

The construction of the Shield Building roof consisted of two stages. First, a thin reinforced concrete dome roof was placed (about 5″ thick). This dome was supported by the Reactor Containment Vessel shell dome by means of temporary construction shores. When this concrete dome had sufficient strength, the temporary shores were removed and the balance of roof shielding concrete was placed.

The design and construction of the dome roof and walls at the springline are monolithic. The dome roof and walls are reinforced in a meridional direction for structural discontinuity, moments and shears. The dome roof and walls are also reinforced in a circumferential direction to resist the hoop tensile stresses caused by springing of the dome roof at its point of support. The hoop reinforcement is proportioned in accordance with the discontinuity strains of the concrete shell near the springline to achieve an efficient use of the steel for crack control. The concrete dome roof is under continual compression due to dead load; therefore, cracking in the dome is negligible.

It is estimated that the thrust of the dome roof will result in a maximum tensile stress of 120 psi in the hoop concrete section at the springline. This stress is well within the limiting stress at which cracking in concrete can be expected to occur. However, in keeping with a conservative approach the concrete is assumed to crack. The resulting inleakage through these cracks is shown in the Table 5.3-1.

The wall and dome surfaces are provided with a minimum of 2″ concrete cover over the reinforcing bars as required by ACI-318, to provide an adequate protection of reinforcement against freezing and thawing.

12.2.3.4.3 Foundation Slab and Support and Adjacent Building Construction

The foundation slab of the Reactor Building has structural continuity with the foundation slab of the adjacent building.

All construction joints below grade, for structures housing Class I equipment, have been provided with polyvinylchloride waterstops. In addition, the ground exposed surfaces of the base slab and exterior walls of the main powerhouse structure consisting of the Administration, Turbine, Service, Auxiliary and Shield buildings have been completely enveloped with an impermeable membrane which consists of continuous sealed 40 mil thick polyvinylchloride sheets. This membrane terminates in a wall reglet located six inches above grade.

The design provides for the relative static settlements that could occur between structures as dead load is placed on the slab during construction. The design assumes a zero relative settlement between structures during an earthquake. The design takes into account local foundation deformations during an earthquake due to the dynamic action of a rocking structure.

The structures are supported on controlled recompacted soils as described in Appendix E.

Structures adjacent and exterior to the Shield Building walls are designed with provisions for the movements of the Shield Building during an earthquake.

Walls originating in the Auxiliary Building that abut the Shield Building wall are isolated from the Shield Building by an adequate physical separation to prevent damage to either structure due to hammering during an earthquake.

Floors in the building adjacent to the Shield Building wall are not supported by the Shield Building wall but are cantilevered from columns. These floor slabs and beams are separated from the Shield Building wall by a flexible expansion joint that permit independent building movements that might occur during an earthquake. Adequate physical separation between these floor slab terminal edges and the Shield Building walls are provided to prevent hammering.

The Shield Building and Auxiliary Building (part of which is secondary containment) were dynamically analyzed for seismicity by John Blume & Associates. The results of these two analyses were correlated to determine the separation required between the two structures to assure that there would be no physical contact between the structures under earthquake conditions.

Flexible expansion joints have been provided between the Shield Buildings and the Auxiliary Building above the ground floor (elevation 695′-0).

The expansion joint consists of a foam plastic filler used to form the joint and sealed with a sealing compound, which adheres tenaciously to concrete or steel. The sealant is not subject to adverse environmental effects and requires no maintenance.

This seal is of a type that provides for movements that extend or contract the separation joint and for movements in the two other coordinate axes of movement. Calculated movement between the Auxiliary Building and Shield Building, with the buildings accelerating away from, or toward one another, is 0.5 inches. Figure 12.2-15 indicates the type of joint used.

The linear feet of joint between the Auxiliary Building Special Ventilation Zone Boundary and the Shield Building Wall is approximately 444 ft. Since most of the joint is filled in with caulking compound or by roofing material, gross leakage is determined by the Auxiliary Building Ventilation system tests per Technical Specifications. 01436159

Periodic auxiliary building special ventilation tests, as required and described in Technical Specification Section 3.7.12, will indicate any excessive leakage into the Auxiliary Building.

12.2.3.4.4 Analysis and Design for Missiles

The Shield Building is designed to withstand the tornado missile without loss of function. It is also designed to intercept the turbine missile and prevent it from damaging the Reactor Containment Vessel. Details of design analysis methods and criteria are given in Section 12.2.1.4.3.

12.2.3.4.5 Design Analysis

12.2.3.4.5.1 Dynamic Shell Analysis of Shield Building

The mathematical model used by John A. Blume & Associates in their dynamic analysis of the plant structures (See Figure No. 10 and 11 in Reference 6) considered the Shield Building as a vertical beam. When the subsequent structural analysis was done, the Shield Building was analyzed as a shell, using Kalnin's computer program for axi-symmetric shells. The output of Blume's dynamic analysis included accelerations of the Shield Building in a N-S and E-W direction (Figure No. 13 in Reference 6).

These accelerations were used to determine the horizontal seismic forces which occur at each level of the Shield Building. The forces for the N-S earthquake and for the E-W earthquake were considered separately and not combined. These seismic forces were used as input to Kalnin's Program which gave, as output, the axial forces, moments and shears in the Shield Building. These were then converted to stresses in the concrete and in the reinforcing steel.

Because the Shield Building is a very rigid cylindrical structure with stiffening members at the bottom (foundation and grout) and at the top (dome), the seismic effect on reinforcing steel is very small. In conjunction with LOCA, which produces a steep temperature gradient (outside -35°F and inside +165°F) the thermal effect far exceeds the seismic effect.

In the dome the combination of loads with OBE produced a maximum stress in the reinforcement of 13.30 ksi in the meridian direction and 14.63 ksi in the hoop direction, with these stresses occurring at the springline in the discontinuity zone. In the walls the maximum stress is 17.43 ksi in the meridian direction and 19.12 ksi in the hoop direction.

In the construction stage, the load of the Shield Building walls and the Containment Vessel act on the periphery of the base, which, in its conical form, spans 120 ft. diameter and produces heavy ring tension forces in the wall which require heavy reinforcement. Although these stresses are relaxed with construction of the internal concrete, in the analysis they were assumed as fully existing during the operational stage. The maximum stresses are 21.05 ksi in the meridian direction and 21.23 ksi in the hoop direction.

The stresses in the Shield Building in combination with DBE showed little change, staying far below the allowable stresses of 1.5 times the code working stresses, with the maximum stress in the dome of 14.62 ksi in the meridian direction and 17.69 ksi in the hoop direction, the stresses in the cylindrical part were 28.92 ksi in the meridian direction and 27.57 ksi in the hoop direction, and near the base of the walls, the stresses were 28.44 ksi in the meridian direction and 32.38 ksi in the hoop direction.

Tornado loads do not cause substantially higher stresses in the dome, the maximum being 29.04 ksi in the meridian direction and 34.85 ksi in the hoop direction, except at the dome springline the maximum stresses are 36.3 ksi in the meridian direction and 26.29 ksi in the hoop direction. The walls are stressed locally to 50.12 ksi in the meridian direction and 51.87 ksi in the hoop direction, with these maximums occurring at different elevations. At the base the stresses are 38.53 ksi in the meridian direction and 27.75 in the hoop direction. The allowable stress for loading combined with tornado is 54 ksi (90% of yield strength).

The shear stresses caused by OBE, DBE and tornado are within the allowable for concrete and no shear reinforcement is required. The shear reinforcement used in the walls and base was required for loads created by construction and temperature conditions.

12.2.3.4.5.2 Dynamic Finite Element Analysis of Shield Building (Lobar Motion)

An additional dynamic analysis of the Shield Building dome and wall was made considering them as a part of an axi-symmetric shell of revolution. The shell structure was analyzed by the finite element method utilizing the general Structural Analysis Program (SAP) developed by Professor E. L. Wilson of the University of California at Berkeley.

A finite model was set up with a plane of symmetry passing through the diametrical section of the structure with only half of the structure considered. The finite elements are shown in Figures 12.2-6, 12.2-7 and 12.2- 8 with a total of 351 quadrilateral elements and 381 node points. Model selected includes all deformations including lobar motion.

A Raleigh-Ritz procedure was used in the computer program to determine the natural frequencies and mode shapes of the shell structure. The number of degrees of freedom was condensed to 54. Frequencies and mode shapes were calculated for the first 20 modes.

The response spectra approach was used to analyze the dynamic response of the shell structure. The structure was assumed to be fixed at the base foundation when the horizontal response spectra representing the ground motion for a maximum horizontal ground acceleration of 6% of gravity (OBE) as shown in Appendix E, (Plates H4.5 and H4.6) is applied as computer input. Damping was assumed to be 2% of critical.

The computer output gave translational and rotational displacements of all node points as well as stresses and moments of all elements for the first 20 modes. A square-root-sum-of-the-squares combination of stresses and moments of all elements was computed and tabulated (See Table 12.2-27). The tabulated stresses are equivalent concrete stresses and are negligible when converted to tensile steel stresses.

The tabulated results show that the combined stresses are generally compressive over the inside face of the entire dome except in the following locations:

a. The maximum hoop stress on the dome near the juncture with the wall is 134 psi. This was anticipated and is included in the design.

b. Meridional stress near the juncture with the wall is 5.29 psi.

In element ’253’ of the analysis, a small tensile meridional stress (5.29 psi) occurs at the inside surface. This is the result of a combined dead-load and square-root-sum-of-squares stresses which represent the upper limit of stress in absolute value. In reality, there is no tensile meridional stress at the inner face. This has been verified by the stress condition of Modes 1, 11 and 17. Even at this stress level, the stress in the provided reinforcing steel is no more than 1,400 psi which is quite insignificant. Therefore, no danger of cracking or splitting under earthquake conditions exists.

According to the computer output of the dynamic response spectral analyses, (Figure 12.2-9, Sheets 1 through 4) the first mode gives the most significant contribution to the response, followed in descending order of participation factors by the second, eleventh, twelfth, seventeenth and tenth modes. The non-symmetrical mode of the dome occurs at the 11th and 17th modes. (In Reference 11, non-symmetrical modes occur in modes 3 and higher). Comparing with the illustrative example of Reference 11, we see that the first mode frequency is in the same order of magnitude (4.10 cps vs. 3.65 cps). Due to the use of different types of elements, our analysis using quadrilateral plate elements gives more modes than that using ring elements. Our mode No. 11 showing antisymmetrical bending configuration of the dome (f=15.48 cps) is corresponding to mode No. 4 (f=21.92 cps) of the sample while our mode No. 17 (f=22.02 cps) is similar to their mode No. 5 (f=23.76 cps). Mode No. 3 and some extra modes in our analysis show the ovalling action (lobar mode) of the cylindrical shell.

To further verify the results of the dynamic analysis, the structure was also analyzed statically by the use of equivalent static loads to simulate the action of the specified earthquake (see Table 12.2-28). Four cases of loading were considered in a finite element analysis using the SAP program.

a. Horizontal
b. Vertical up
c. Combined horizontally and vertically up
d. Combined horizontally and vertically down

Another loading condition was considered by use of Kalnin's program for shells; combined horizontal and anti-symmetrically vertical, i.e., half up and half down.

All these static analyses show that the stress level due to the assumed dynamic excitation is very low compared with the dead load stresses. Therefore, the dynamic effect on the structure is insignificant and buckling or crushing of the dome is incredible.

Reference 12 illustrates a method of determining the effect of the transverse earthquake shear waves across a building of sizable dimensions, but the given information is for square or rectangular buildings. Assuming it is applicable to a round building, calculations were made based on the natural frequency of the Shield Building as a cantilever fixed at the foundation slab and the shear wave velocity given in Appendix E. These resulted in a value of accidental eccentricity “e” of 46% which would cause a torsional shearing stress in the walls of 56 psi at the foundation level. The dead load of the Shield Building dome and wall, reduced by vertical earthquake at this level causes a compressive stress of 175 psi, therefore, the torsional shear is pure shear and cannot result in diagonal tension.

The resulting stresses from individual loadings are combined as indicated in Table 12.2-25.

The Shield Building is subjected to two distinct types of shear action, i.e., (a) a radial shear normal to the building walls developed by discontinuity conditions, and tornado internal pressure and (b) a tangential shear and accompanying longitudinal shear taken on a radial plane through the walls induced by the flexural action of the structure under the effects of wind or earthquake loads (Reference 17).

The containment atmosphere is generally maintained greater than 80°F during normal modes of operation. The outside temperature in winter time could be at -20°F for extended periods of time.

The steady-state temperature distribution in the Shield Building was determined, using the conservative assumption that the containment vessel and the annulus air are both at 120°F and the outside temperature is a -35°F. The calculated inside and outside wall temperatures are 99°F and -7°F, respectively.

The transient temperature distributions following a LOCA were calculated by superimposing on the steady-state temperature profile, the effects of radiative and convective heat transfer to the wall from the annulus air and the containment vessel. The maximum wall temperature is 117°F on the inside surface at 45.2 minutes following a LOCA, and temperature changes at a depth beyond 10 inches are then negligible. The physical properties as well as the assumptions used to calculate the steady and transient temperature profiles are listed in Table 12.2-30.

The total concrete and steel stresses, including thermal stress due to the transient thermal condition are also presented in the table, and these are below the allowable limits.

The calculation described is conservative in most respects, rather than realistic. The LOCA conditions are those associated with a double-ended break, and calculations of outer containment wall temperatures following smaller breaks indicate very little difference in heat input to the annulus for these cases. The dominant thermal effect is the gradient resulting from initial temperatures on opposite sides of the wall and these temperatures have been chosen conservatively.

The Shield Building is a structure of a conventional reinforced concrete building design and construction. The Shield Building is designed to resist the internal pressure of 3 psi due to tornado loading. However, this does not subject the structure to significant membrane forces that are ordinarily associated with pressure vessels. This low internal pressure minimizes the cracking of concrete and assures that concrete will function as a shear carrying member. In addition, the structural discontinuity stresses are small since no significant pressure forces are present. Radial shears are therefore very low.

The shear provisions of the ACI Code are applicable to concrete members under the combined action of moments and applied axial compressive or tensile loads. These provisions are applicable to the transverse and tangential shear actions noted above.

The tangential and longitudinal shear distribution over the annular cross section and at any point is given by:

$$v_c = \frac{V}{\Pi rt} \cos\theta$$

The tangential and longitudinal shear varies from zero over a thickness of wall located at the extremes of a diameter normal to the neutral axis to a maximum on a wall thickness located on both extremes of a diameter coincident with the neutral axis and is given by:

$$v_{max} = \frac{V}{\Pi rt}$$

V = Total shear on the annular cross section.

v_c = Unit tangential or longitudinal shear on the annular cross section

r = Mean radius of Shield Building wall.

t = Thickness of Shield Building wall.

θ = Polar angle measured on either side of the neutral axis locating the point at which the unit shear is to be determined.

The maximum unit shear is limited in the design by the allowable values of the ACI 318-63 Code, Chapter 12.

12.2.3.5 Construction

The reinforced concrete vertical cylindrical wall of the Shield Building was constructed using the slip-form method as described in Chapter 5 of ACI Code 357-68.

Several minor changes were made to the fixed-form designs to facilitate slip-forming. The changes affected the following:

a. Length of forms for block-outs, plate inserts and embedded items were shortened for easy placement in forms.

b. Spacing of form supporting yokes required shorter lengths of reinforcement.

c. Concrete slump increased to 6 inches and maximum aggregate size was limited to 3/4″.

d. Form to be tapered to avoid adhesion as form is raised.

During the early slip-forming stages of operation, the Quality Control Engineer inspection detected that the Shield Building wall showed slight deviation from true plumb. The contractor made corrections by revising the jacking rate at selected locations to bring the wall back to plumb. While making this correction, a slight out-of-roundness developed and again corrective measures were undertaken to regain the circular cross-section.

After completion of construction, the Shield Building wall was surveyed to measure the deflection from plumb and out-of-roundness. These deflections were used as input to a structural analysis computer program to determine the effect on stresses. The as-built structure has a slight increase in stresses due to out-of-shape, however stresses do not exceed the maximum allowable as established for the Shield Building. For complete detailed study of slip-form construction and structural analysis, refer to Northern States Power Company and Pioneer Service & Engineering Co. report dated December 4, 1968, and submitted to AEC Division of Compliance, Region III on December 4, 1968.

12.2.4 Internal Structures

12.2.4.1 Design Criteria

The reactor cavity, the reactor coolant system compartments, and all equipment rooms housing Class I components or systems as shown in Table 12.2-1 are designed as Class I structures. The design criteria for structures as described in Section 12.2.1.4 is applicable with the exception of environmental loads such as snow, wind and tornado which do not affect containment internal structures.

The design has been performed in accordance with the applicable portions of the following design codes:

American Concrete Institute Code 318 "Building Code Requirements for Reinforced Concrete," 1963 Edition.

American Concrete Institute Code 349 "Code Requirements for Nuclear Safety Related Concrete Structures," 1985 Edition.

American Institute of Steel Construction "Specification for the Design, Fabrication and Erection of Structural Steel Buildings," 1963 Edition.

American Welding Society Code D 1.0 "Standards for Arc and Gas Welding in Building Construction."

International Conference of Building Officials "Uniform Building Code," 1967 Edition.

Atomic Energy Commission publication TID 7024 "Nuclear Reactors and Earthquakes."

The design pressure differentials across walls and slabs of the enclosed compartments in the internal structure have been calculated for the hypothetical ruptures of the Reactor Coolant System within the compartments.

Section 4.6.2.3 discusses the basis for the elimination of large primary loop pipe rupture as a structural design basis. Based on the acceptance by the NRC, as documented in a Safety Evaluation Report (Reference 47) and as stated in NUREG-1061, Volume 3, pipe whip, pipe break reaction forces, jet impingement forces, and vessel cavity or subcompartment pressurization including asymmetric transient effects may be excluded for primary loop pipe rupture only. The original design included these dynamic effects, as described below, but should no longer be considered part of the plant design basis. Note that the dynamic effects due to steam and feedwater line breaks described below are not excluded by the "leak-before-break" analyses and continue to be part of the plant design basis.

A four volume model was used to calculate the peak differentials for the steam generator vault. The volumes represented were the dome, the base compartment, and the two steam generator vaults. Time dependent equations of conservation of mass, conservation of energy, conservation of momentum and state were used in the calculation. Flow inertia effects between the volumes were calculated. This model calculated critical flow conditions for application under high pressure differentials. The model assumed 100% entrainment of the water which emerges from the break. The time step used up to the time of the peak differential was one millisecond.

The pressure in the pipe annulus and reactor annulus were established through a steady state analysis of the peak mass and energy flow in the region. This study utilized two phase critical flow relations. The pressure in the compartment below the reactor was established by addition of a fifth volume, representing this compartment, to the model used to calculate the peak differentials of the steam generator vaults. A steam and energy flow was input to this new volume to calculate the resultant peak differential. The time step in use when the peak pressure was calculated was four milliseconds, although time steps of one millisecond and two milliseconds were used earlier in the transient. The lengthening of the time step is due to the low rate of change of the blowdown after the initial flow rise through the break. The shorter time steps are used to follow the rapidly changing portion of the blowdown.

The peak differential pressure in the steam generator vaults for each unit have been calculated for a double ended rupture of a reactor coolant pipe. One steam generator vault had a volume of 21,740 ft^3 and vent areas of 360 ft^2 and 111.3 ft^2, the other vault has a volume of 24,500 ft^3 and vent areas of 394 ft^2 and 106 ft^2.

The reactor cavity region has been analyzed for the effects of a longitudinal split inside the concrete pipe sleeve of area equivalent to the cross- sectional area of a reactor coolant pipe. A circumferential failure of the pipe at this location would result in a much smaller flow discharge area because the vessel, pipe, and sleeve arrangement is such that no significant relative movement can take place. The compartments analyzed are the pipe sleeve, reactor vessel annulus, and the compartment below the reactor. The volume of the pipe annulus is 117.3 ft^3 and the vent area is 23.5 ft^2. The volume of the reactor vessel annulus is 223 ft^3 and the vent area is 43.9 ft^2. The volume of the compartment under the reactor vessel is 4709 ft^3 and the vent area is 33.2 ft^2. Figure 12.2-16 shows the volume and flow path schematic of the reactor cavity region.

The absolute pressure achieved in a steam generator vault at the time of the peak differential pressure is 30.0 psig. The maximum differential between the steam generator vault and the dome is 28.6 psi and the maximum differential between the steam generator vault and the base compartment is 24.8 psi.

The peak pressure calculated in the pipe annulus is 365 psia which corresponds to a peak differential of 350 psi. The peak pressure in the reactor annulus is 99 psia, or, a differential of 84 psi. Finally, the peak pressure in the cavity below the vessel is 25.2 psig and the peak differential pressure is 24.4 psi.

The jet forces from rupture of various pipes are:

Break Location	Jet Force
Primary loop hot leg	1800 kips
Primary loop cold leg	1600 kips
Crossover (pump suction leg)	2250 kips
Steam line (Unit 1 / Unit 2)	813 kips / 700 kips
Feedwater line (Unit 1 / Unit 2)	277 kips / 231 kips

The structural design capability of the steam generator vault and reactor cavity compartments to withstand these differential-pressure and jet forces are discussed in the following paragraphs.

12.2.4.1.1 Steam Generator Vault Compartment Structural Design

The steam generator vault design depends primarily on steel reinforcement to carry the loads induced by compartment differential pressures, jet impingement forces and steam supply equipment reaction forces resulting from pipe rupture. Loads from compartment pressure differentials represent only a minor part of the total design load. The wall thickness was determined from biological shielding requirements and the resulting thickness exceeds that required by the loading configuration. The horizontal and vertical steel reinforcing bars carry the loads associated with the hoop stresses and longitudinal stresses respectively. Steel reinforcing bars formed into stirrups carry the shear load. Figure 12.2-17 shows the orientation of the stirrups.

The simplified analysis initially used in the design of the compartments was a quasi-folded plate method. This method exaggerates the loading on the transverse and longitudinal frames examined during the analysis, because some actual loads are accounted for twice. This repetition leads to a structure that is very conservatively designed. The finite element analysis subsequently performed, demonstrated the degree of conservatism of the initial analysis.

12.2.4.1.1.1 Structural Analysis of Steam Generator Compartment

12.2.4.1.1.1.1 Model

The finite element code Structural Analysis Program (SAP) was used for the analysis. The structure for the compartment around the Unit 1 Steam Generator 1A was modeled. The compartment configuration is shown in plan view in Figure 12.2- 18, and in section on Figure 12.2-19. The model consists of vertical steam generator and reactor coolant pump shield walls, mezzanine floor, missile shield slabs, knuckle wall, and steam generator support steel beams.

The structural model consists of four different type of elements connected by 2151 nodal points. The number and type of elements are:

a. 967 three dimensional brick elements

b. 40 three dimensional beam elements

c. 42 elastic shell elements

d. 111 boundary elements.

12.2.4.1.1.1.2 Loads

Structural integrity was analyzed for the 24 loading cases listed in Table 12.2-31. The loads are defined in the following discussion.

The maximum pressure differential load was determined from the compartment pressure analysis described above. The maximum pressure differential was 27.4 psi at 0.17 sec. after the break. A differential pressure of 28.6 psi was used in the confirmatory finite element analysis.

The seismic loads are determined from the analysis provided by References 70 and 71. The Operational Basis Earthquake (OBE) has been defined as having ground acceleration of 0.12g. The Design Basis Earthquake (DBE) has been defined as having ground acceleration of 0.24g. For this analysis an equivalent static load method was used.

Pipe rupture forces, namely jet impingement and pipe reaction forces, were developed from a detailed analysis of the steam generator supports and calculations done expressly for this analysis.

Load Combinations

Loading combinations for design and stress investigation of the compartment structures are defined in Table 12.2-32. (See Notes in Table 12.2-32)

a. Load Combination 1: Normal Operation

Stresses are very low during normal operations since the operating loads are extremely low when compared to accident induced loads. The massive self supporting walls and missile slabs are designed to restrain equipment and piping of Reactor Coolant System from lateral movements during hypothetical accidents.

Similarly, compartment structures are also designed to provide biological shielding from radiation, and this requires wall thickness in excess of those required for structural integrity.

b. Load Combination 2: Design Basis Accident

This load combination includes maximum calculated internal differential pressure plus Operating Basis Earthquake. The wall acceleration used was 0.12g.

c. Load Combination 3: Safe Shut-Down

Load Combination 3 is similar to Load Combination 2 above except that the earthquake loading is the Design Basis Earthquake. The wall acceleration used was 0.24g.

d. Load Combination 4: Local Effects Consideration

This combination includes operating load and DBE in addition to an appropriate superposition of the time dependent pressure differential loads, pipe rupture reaction loads and jet impingement loads.

The highest loading results from a double-ended crossover pipe break. The peak load occurs at the time when the pressure differential load is 0.766 of its maximum value.

Load Combination 4 has also been modified to include hypothetical pressure differential loads of 50 psi and 75 psi.

12.2.4.1.1.2 Results of Finite Element Analysis

Concrete walls are modeled with three dimensional brick elements. The computer output consists of stresses at the element faces with the general convention described in Figure 12.2-20, and as indicated shear stresses are part of the unique solution for each loading case.

The most highly stressed elements of each load combination were selected for further detailed examination. The stresses on the brick elements were converted to axial forces, shear forces and moments, at each face. Then the stresses in the steel reinforcing bars and stirrups and concrete were calculated. These stresses were compared with Code allowable values as given in Table 12.2-32.

Because most sections are subjected to axial loads, the shearing capacity of the concrete was computed by

$$V_c = 1.75\sqrt{f_c}\left(1.0 + 0.004\,N/Ag\right)$$

where N = axial force (if N is tension, negative sign reduces concrete shearing capacity), for Load Combinations 2 and 3. For Load Combination 4 the shearing capacity of the concrete was computed by

$$V_c = 2\left(1.0 + 0.002\,N/Ag\right)\sqrt{f_c}$$

The stresses for the critical sections are summarized in Table 12.2-33. For Load Combination 4a the calculated stresses are below the stress limits. The assumed allowable concrete stress design limit of 3400 psi is well below the actual measured 28-day compressive strength of 5000 psi.

Further, in regard to the reinforcing bars specified by code, minimum yield strength is 60,000 psi but it has a tested average strength of 70,000 psi.

The margin of the compartment structural capability is indicated by the results for Load Combination 4c where, even for a hypothetical pressure differential load of 75 psi, the critical concrete stress is below both the specified and actual measured strength and the reinforcing steel stresses are below the specified and measured yield limits. Essentially no permanent deformation would be expected to occur.

Initially only the compartment walls of Unit 2 were analyzed by a finite element analysis. In this case reinforcing provided was 40% more than the analysis required.

12.2.4.1.2 Deleted

12.2.4.1.3 Structural Analysis of Reactor Vessel Cavity Compartment

The load carrying ability of the reactor vessel cavity compartment is provided primarily by the 7.33-ft. thick steel reinforced concrete walls. Steel supports for the reactor vessel are also encased in the concrete structure.

The dead load consists of the weight of the walls, equipment and water in the refueling pool. Thermal loads result from the neutron attenuation-induced thermal gradient in the wall. The long term design differential pressure for the cavity is 100 psi; however, a short term differential pressure of 185 psi and an upper bound pressure differential of 480 psi have been considered for analysis purposes.

Seismic loads are determined from Reference 6. The highest structural stress and design limits for different load combinations are given in Table 12.2-34.

12.2.4.1.4 Nozzle Cavity

The only applicable design load for the nozzle cavity is the differential pressure load due to pipe rupture. The most critical stresses and the associated stress limits for the design differential pressure of 475 psi and an upper bound differential pressure of 1400 psi are given in Table 12.2-35.

Neutron absorption by the concrete biological shield structure at the level of the reactor core results in a temperature of 124°F on the inside surface of the concrete facing the reactor vessel and an internal concrete temperature which peaks at about 195°F two (2) feet inside of the concrete. Thermally induced stresses in the concrete shield structure caused by neutron absorption were structurally analyzed using the "IBM M003 FLEXIBILITY ANALYSIS" program.

The concrete and reinforcement design of the concrete shield structure is based on the "Working Stress Design Method" of the ACI-318-63 Code. Figure 12.2-21 depicts the reinforcing steel patterns for the reactor vessel biological shield. The design pressure for the biological shield of the reactor cavity is 100 psi.

To provide generous margins in the design of the six structural steel columns that support the reactor vessel and that are embedded in the concrete of this region, the structure was analyzed as an independent vertical support system and was assumed to carry the total reactor vertical load without relying on the surrounding concrete. The embedded structural steel members were designed in accordance with the requirements of the AISC, "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings".

The design was reviewed to assure that any resulting deflections or distortions do not prevent the proper functioning of the structure or piece of equipment, do not endanger adjacent structures or components, do not allow the uncontrolled release of radioactive material, and do not prevent the safe shutdown and isolation of the reactor. Stresses resulting from earthquake were compared with stresses implied by the damping values used to assure that the analysis was consistent.

The reactor cavity is designed to contain safety injection water up to the level of the reactor nozzles.

Pipe-whip restraints are provided as required to prevent consequential damage to the Reactor Coolant System, engineered safety features systems, containment isolation systems, and the Reactor Containment Vessel as the result of a double-ended rupture of high pressure piping within the containment.

The design of the internal structures has been examined to assure that credible missiles from high-pressure equipment will not cause a loss of function within the structure, the Reactor Coolant System, the containment isolation systems or the Reactor Containment Vessel. The principal barriers against missiles are the reinforced concrete biological shield, the shield walls around the Reactor Coolant System compartments, and miscellaneous local barriers. A steel barrier is located above the reactor vessel head to block any missile that could be generated by a failure of the control rod drive mechanism.

12.2.4.1.5 Reactor Coolant System Compartments and Refueling Cavity

There are two Reactor Coolant System Compartments, each housing one loop of the primary nuclear steam supply components; namely, pressurizer, (1 loop only), steam generator, and reactor coolant pump. The compartments are vertical vaults of reinforced concrete construction with flat walls having an irregular geometric wall configuration.

12.2.4.1.5.1 Loads

The vault compartment structure and refueling cavity are designed to carry the vertical and horizontal live and dead loads of adjacent floors and structures that are attached to it as well as its own live and dead loads and horizontal loads imposed on the compartment walls from the equipment components contained within and external to the compartments.

Also included in the design are: -

a. Jet Load

   Jet impingement loads on compartment surfaces that are postulated to occur in the event of a pipe rupture.

b. Internal Pressure

   Compartment internal pressure buildup that would occur during a LOCA blowdown. These loads are defined, Section 12.2.4.1.

c. Pipe Rupture Reaction Load

   These are reaction forces on equipment and structures caused by a postulated pipe rupture.

d. Thermal Load

   These are stress loads from thermal gradients through compartment walls that are caused by a LOCA.

e. Residual Construction Loads

These are internal residual construction stress loads resulting from the shrinkage of the various levels of construction pours of concrete.

f. Refueling Cavity Pool Water Loads

Vertical pool water load and lateral hydraulic water pressure loads from water in the refueling cavity pool. These loads are treated as live loads since they will only occur during reactor shut-down.

g. Seismic Loads

Seismic loads are determined from Reference 8 for the appropriate mass points under consideration.

12.2.4.1.5.2 Method of Analysis

The general structural model consists of a composite of the compartment walls, the adjoining biological shield, the attached floor systems and structures, and the refueling cavity pool.

The analytical model is broken down into two distinct interacting models: (1) laminar horizontal frames analyzed by STRESS programs and (2) vertical structural elements analyzed to carry unbalanced load reactions to the base by the box girder action of the compartment walls.

This latter model could more aptly be described as a modified folded plate structure.

The local effect of forces are analyzed by considering loading conditions on simplified but conservative models limited to the immediate and closely adjoining areas of loading.

The above described analysis results were later confirmed by an independent finite element method of analysis that indicated conservative margins in the results of the original analysis of the order of 30 percent and greater.

12.2.4.1.5.3 Design Codes

Concrete walls are designed in accordance with the working stress design methods of the ACI-318 Code.

Concrete is specified to attain a minimum strength of 4000 psi in 28 days. Reinforcement is ASTM-A615, Grade 60. No bar sizes greater than No. 11 are used. No welding of reinforcement was permitted.

In those portions of concrete walls subject to dynamic loadings and having heavy tensile reinforcement, particular attention was given to enhance the ductility of the concrete by the addition of stirrup systems (Reference 13).

12.2.4.1.5.4 Design Criteria

The design stress limits for the various design condition categories are shown on Table 12.2-32. This criteria is in accordance with Section 12.2.1.4.

Criteria and stress limits are shown for reinforced concrete and structural steel. Structural steel criteria and stress limits applies to those members that are a functioning integral part of the compartment structure system, i.e.; embedded items, structural wall-tie members, etc.

Pipe rupture reaction forces and resulting jet impingement loads act on limited extents of the structures. Because of the instantaneous nature of these loads, they can produce very high local stress concentrations on parts of the structures. Accordingly, these loads require a somewhat different treatment than categories 1, 2 and 3. Tables 12.2-29 and 12.2-32 include category-4, “Pipe Rupture Reaction Loads and Jet Loads”.

This category includes the local effects of pipe rupture reaction loads and jet forces. The criteria and increased stress limits presented in this category take into account the self-equilibrating nature of these localized stresses. This is analogous to the techniques employed in the design of pressure vessels. The design stress limits indicated are consistent with the design margins used throughout the plant.

Because of the time related effects of the reaction forces induced by a pipe rupture when considered simultaneously with internal compartment pressure buildup from a LOCA blowdown, two limiting loading conditions were established for category-4, as shown in Table 12.2-32.

12.2.4.1.6 Structural Supports for Steam Generators, Reactor Coolant Pumps, and Reactor Vessel

12.2.4.1.6.1 Loads

12.2.4.1.6.1.1 Steam Generators and Reactor Coolant Pump Structural Supports

The structural support system for the steam generators and reactor coolant pumps is designed for the following loads:

a. Operating Loads. Consists of dead loads, live loads, and stresses caused by thermal expansion.

b. Seismic Loads. Seismic loads are determined from J.A. Blume Response Acceleration Spectra Curves (Reference 8) for the appropriate mass points under consideration.

c. Pipe Rupture Reaction Loads. These are reaction forces on equipment and structures caused by a postulated pipe rupture.

12.2.4.1.6.1.2 Reactor Vessel Support Structures

The ventilated support structures located on top of the reactor vessel support columns are designed for the following loads:

a. Operating Loads. Operating loads consist of: (1) Reactor vessel vertical dead loads, (2) Lateral radial friction loads caused by radial thermal expansion of the reactor vessel, (3) Internal loads within the ventilated support structures caused by local temperature gradients.

b. Seismic Loads. Seismic loads are determined from J.A. Blume Response Acceleration Spectra Curves (Reference 8) for the appropriate mass points under consideration.

c. Pipe Rupture Reaction Loads. These are reaction forces on equipment and structures caused by a postulated pipe rupture.

12.2.4.1.6.2 Method of Analysis

12.2.4.1.6.2.1 Steam Generator Method of Analysis for Supports 01406855

The RSG lower lateral supports are modeled explicitly in the RCS structural model. The support wide flange beams are modeled with proper section properties and the bumpers are connected to those using single direction gapped springs. The fixity applied to the supports simulates the actual fixity of the system. For example, the column supports are modeled as pin-pin beams, and the upper lateral supports are springs (snubbers) in the hot leg direction.

The reactor coolant pump supports are modeled explicitly in the structural model. The tie rods are represented by springs using the appropriate stiffness properties for their cross sections. The RCP vertical columns are modeled as pin-pin beams. Each of the RCP tie rods only acts in the direction (tension or compression) that is appropriate for that tie rod, ensuring the support system behaves as it was designed.

The RV supports are modeled explicitly in the RCS structural model. The air boxes are represented by elements with the same properties as the air box shell. The nozzles are free to slide in the radial direction but are prevented from moving in the transverse direction. Each of the six supports is terminated at their connection to the internal concrete structure.

Each of the support systems is incorporated into the overall Reactor Coolant System model containing the replacement steam generators. This model is analyzed for static (i.e., deadweight, thermal expansion, and pressure expansion) and dynamic (seismic and HELBA). The gapped supports are given two different support schemes representing the active supports for positive and negative horizontal motion of the reactor coolant loop. The High Energy Line Break Analyses are done using a time history analysis and the gapped supports are evaluated at each interval to determine their reactions.

12.2.4.1.6.2.2 DELETED 01406855

12.2.4.1.6.3 Design

The structures are of heavy welded construction. The materials are specified to be ASTM-A588 high strength low-alloy steel except for the coolant pump horizontal linkage bars, which are ASTM-A517 and the connecting bolts to the steam generators and the pumps, which are high strength Maraging alloy steels except for Unit 1 and Unit 2 SG connecting bolts, which are high strength alloy steel bolting per ASTM-A193 Gr. B7 or ASME-SA193 Gr. B7. Anchor bolts are specified to meet ASTM-A540 requirements.

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The design and fabrication is specified in accordance with the AISC Specifications for the "Design, Fabrication, and Erection of Structural Steel for Buildings" and selected provisions of the ASME Boiler and Pressure Vessel Code. Welding is in accordance with Section IX of the ASME Code.

In general, the final design sizing of members and welds as determined by the specified design criteria have been increased approximately 40 percent above the minimum design requirements to provide adequate margins in the design.

All anchor bolts carrying dynamic loads of equipment and that are subject to the forces of pipe rupture reactions are designed to be positively anchored by means of embedded anchor plates.

All plate material is specified to be ultrasonically tested. Steel plates, structural shapes and anchor bolts are specified to meet charpy impact test requirements of 15 ft-lb average and 12 ft-lb minimum at 40°F. Anchor bolts are magnetic particle inspected.

All welding procedures are prequalified. All welders are qualified for the project work regardless of previous qualifications. All welds are either ultrasonically inspected or magnetic particle inspected in progressive steps. All welds over 1-1/2 inches in thickness are stress relieved.

All work is specified to meet at least the minimum equivalent quality assurance requirements of Appendix IX of the ASME Boiler and Pressure Vessel Code.

12.2.4.1.6.4 Design Criteria

Because of the impulse nature of pipe rupture dynamic loads and the short duration of these loads, on the order of 10 milli-seconds or less, there will be high strain-rate loadings present in the response of the structures.

This factor of the dynamic material behavior has been taken into account in establishing the design criteria since the results of tests have shown that the stresses associated with the initiation of yielding have been found to increase with increased strain-rates (Reference 14).

Localized plastic deformation was permitted in secondary component parts of the support structure members provided that: (1) the stresses in the support structures, equipment, and system piping did not exceed the allowable stress limit criteria, (2) the stability of parts or the whole of the support structures was not affected.

Table 12.2-36 indicates the design stress limits that were used for the various design condition categories. The stress limits that are indicated for concrete apply to the anchorage design of the support embedments.

12.2.4.1.7 Reactor Cavity Biological Shield

The biological shield is a massive reinforced concrete structure that surrounds the reactor vessel and all its nozzles and immediate piping. The shield structure is supported in the vicinity of the containment vessel bottom head. It supports the reactor vessel by composite action with the embedded support columns. The top of the shield forms the floor of the refueling cavity pool. Its interconnection with the steam generator compartments and floor systems integrates its design function into the total interior structure complex.

12.2.4.1.7.1 Loads

The shield structure is designed to carry the vertical and horizontal live and dead loads of adjacent floors and structures that are attached to it as well as its own dead load.

Also included in the design are:

a. Pressure forces within the confines of the structure that are postulated to build-up in the event of a LOCA. These pressures are identified in Section 12.2.4.1.

b. Stress load from neutron absorption. This is treated in Section 12.2.4.1.3.

c. Water pressure loads from the pooling of safety injection water in the reactor cavity up to the level of the nozzles.

d. Reactor vessel dead and live loads.

e. Refueling cavity pool water load. This load is treated as a live load and will only occur during vessel shut-down.

f. Residual Construction Loads. These are internal residual construction stress loads resulting from shrinkage of the various levels of construction pours of concrete.

g. Seismic Loads. Seismic loads are determined from Reference 8 for the appropriate mass points under consideration.

h. Pipe Rupture Reaction Load. These are reaction forces on equipment and structures caused by a postulated pipe rupture.

12.2.4.1.7.2 Method of Analysis

The biological shield is analyzed in the integrated structural model complex as described above for the Reactor Coolant System Compartments and Refueling Cavity.

12.2.4.1.7.3 Design Codes

Design Codes are the same as outlined above for the Reactor Coolant System Compartments and Refueling Cavity.

12.2.4.1.7.4 Design Criteria

The design stress limits for the various design condition categories are as set forth in Table 12.2-37.

Because of the massive nature of that portion of the shield structure below the vessel supports and its general involvement in structural action with the compartment structures design criteria generally followed are those that are set for the compartment structures.

The shield structure above the vessel supports responds to applied loads of a more local nature. Therefore, the criteria established in Table 12.2-32 for the local effects of pipe rupture were used.

12.2.4.1.8 Floor Systems

Floor systems consist of the following types: (1) Monolithic reinforced slab and beam construction, (2) Composite concrete slabs and steel framing, (3) steel grating on steel framing. The floors form an integral part of the knuckle support system as well as the interior concrete complex as described in preceding sections.

12.2.4.1.8.1 Loads

The floors are designed to carry the following loads:

Dead Loads
Floor Live Loads
Equipment Loads
Construction Loads
Seismic Loads
Loads imposed on the floor system caused by a postulated pipe rupture.

12.2.4.1.8.2 Method of Analysis

To avoid impractical and unwieldy analytical models the floor systems were modeled with any one or combination of the following models as was appropriate.

a. Individual floor support system

b. Floor and knuckle support system

c. Floor and interior structure system

In most instances floors were modeled as individual floor support systems since steel framing structures are secondary structures and therefore do not contribute to the structural support of primary structures under the conditions of DBA, OBE, and DBE. These supporting steel structures as well as the composite floor structures are analyzed and designed to include the seismic effect of the structure dead loads and supported equipment.

12.2.4.1.8.3 Design Codes and Design Criteria

Concrete floor systems are designed in accordance with the Working Stress Design Methods of the ACI-318 Code. As an alternative, floors may be designed in accordance with the Ultimate Strength Design Method discussed in Section 12.2.15.

Structural steel is designed in accordance with the AISC Specifications for the “Design, Fabrication and Erection of Structural Steel for Buildings”.

Design criteria are in accordance with Table 12.2-5.

12.2.4.2 Stresses in Interior Structures

Table 12.2-38 summarizes the allowable and the maximum actual stresses in the four categories of structures which have been treated in detail in preceding sections.

a. During the placement of concrete the liner plate functioned as a form, and

b. During operation of the reactor, the liner plate in the core area functions to minimize the circulation of activated particulate by the ventilation systems, with its attendant effects on the buildup of radiation sources within the accessible areas of containment.

12.2.4.2.1 Steam Generator and Reactor Coolant Pump Vaults

a. The structural steel system within the vault consists of supporting legs and pinned brackets, upper and lower lateral supports, stops, eye bar and pin retainers. All of these members are secured by anchor bolts rather than by embedment into the concrete walls of the vault. The structural steel system basically acts to transmit the load to the reinforced concrete internal structure.

b. The reinforced concrete walls of the vault resist the radial and lateral loads transmitted by the structural steel system through tension in anchor bolts and bearing against load distribution plates. The loads are dissipated through the reactions of the mass of the walls. The massive reinforced concrete base dissipates the vertical loading of the structural steel (supporting legs) system.

12.2.4.2.2 Reactor Refueling Cavity

The refueling cavity is a complementary system consisting of formed reinforced concrete and stainless steel liner plate. The reinforced concrete wall was placed in two lifts except at the fuel transfer tube opening, where three lifts were required. The floor of the refueling cavity for the most part is the top lift of the reactor cavity concrete. Stainless steel liner plate of 3/16″ thick for walls and 1/4″ thick for floor is field “seal” welded to stainless steel angles that are embedded and anchored in the reinforced concrete walls and floor.

12.2.4.2.3 Work Areas

The work areas consist of three types of construction which are as follows:

a. Top surface of mass (reinforced) concrete - i.e. basement floor

b. Reinforced concrete construction, columns, beams and slabs - i.e. mezzanine

c. Structural steel columns and beams and reinforced concrete structural slab - i.e. refueling and operating floor

12.2.4.3 Materials

The specifications and working drawings for materials and their installation are of such scope and detail that the quality of internal structures is assured and the desired degree of integrity is obtained. Figure 12.2-24 shows the typical arrangement of the reinforcing steel patterns for reinforced concrete beam floors. Figure 12.2-25 shows the typical arrangement of the reinforcing patterns for the steam generator and reactor coolant pump enclosure walls.

12.2.4.3.1 Concrete

The basic specifications for concrete materials and construction practices are as delineated in Section 12.2.3.2.2.

12.2.4.3.2 Reinforcements

Standards and specifications for reinforcing steel, testing and construction methods are as follows:

ACI 301	Specifications for Structural Concrete for Buildings
ACI 315	Manual of Standard Practices for Detailing Reinforced Concrete Structures
ACI 318	Building Code Requirements for Reinforced Concrete
CRSI 63	Recommended Practice for Placing Reinforcing Bars
CRSI 65	Recommended Practice for Placing Bar Supports, Specifications and Nomenclature
AWS D1.0	Code for MC and Gas Welding in Building Construction, Metal Inserts and Connections in Reinforced Concrete Construction
AWS D12.0	Recommended Practice for Welding Reinforcing Steel
ASME	Boiler and Pressure Vessel Code, Welding Qualifications, Section IX
ASTM A615	Standard Specifications for Deformed Billet - Steel Bars for Concrete Reinforcing Grade 40 bars Grade 60 bars
ASTM A185	Specifications for Welded Steel Wire Fabric for Concrete Reinforcement

12.2.4.3.3 Structural Steel

Standards and specifications for structural steel, testing and construction methods are as follows:

“Specification for the Design, Fabrication and Erection of Structural Steel for Buildings,” published by the American Institute of Steel Construction

“Code of Standard Practice for Steel Buildings and Bridges,” published by the American Institute of Steel Construction

“Code for Welding in Building Construction,” published by the American Welding Society (AWS D1.0 and D3.0)

"Boiler and Pressure Vessel Code," Section VIII "Unfired Pressure Vessels," and Section IX
"Welding Qualifications," published by the American Society of Mechanical Engineers

ASTM:

A-36	Specifications for Structural Steel
A-502	Specifications for Steel Structural Rivets
A-325	Specifications for High-Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plant Hardened Washers
A354	Specifications for Quenched and Tempered Alloy Steel Bolts and Studs with Suitable Nuts
A490	Specification for Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints
A588	Specifications for High-Strength Low Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. thick
A240	Specifications for Chromium and Chromium-Nickel Stainless Steel Plate Sheet and Strip for Fusion-Welded Untired Pressure Vessels-Type 304
A298	Specifications for Corrosion-Resisting Chromium and Chromium-Nickel Steel Covered Welding Electrodes
A276	Specifications for Hot-Rolled and Cold-Finished Stainless and Heat Resisting Steel Bars
A306	Specifications for Carbon Steel Bars Subject to Mechanical Property Requirements, Grade 70
A370	Methods and Definitions for Mechanical Testing of Steel Products
A593	Specifications for Charpy V-Notch Testing Requirements for Steel Plates for Pressure Vessels
E109	Method for Dry Powder Magnetic Particle Inspections
E164	Method for Ultrasonic Contact Inspection for Weldments

ASME:

Appendix VI	Section VIII	Methods for Magnetic Particle Examination
Appendix U	Section VIII	Ultrasonic Examination of Welds
N-532	Section III	Requirements for Postweld Heat Treatment

12.2.4.4 Construction

Concrete in Ellipsoidal Bottom

Internal reinforced concrete was placed in the bottom and in the knuckle region of the ellipsoidal bottom of the Reactor Containment Vessel after the concrete fill and chemical grouting of the concrete fill support under the Vessel (Section 12.2.2.4). Sufficient time was given for the concrete fill and chemical grouting to develop strength to carry the loads to be imposed by the internal concrete. The internal concrete was placed in a manner to minimize shrinkage effects. Also prior to placement of internal concrete at the knuckle, the temporary steel stiffeners for the steel ellipsoidal bottom of the Vessel were removed.

12.2.5 Protection and Support for Class I Items

Criterion: No single event will cause failure of redundant circuits or Engineered Safety Feature Components in a manner such that a single failure after the event could prevent the protective functions of the associated Engineered Safety Features. (FSAR Appendix B.5)

12.2.5.1 Protection for Class I Items

The Class I items are protected against damage from:

a. Rupture of a pipe or tank resulting in serious flooding or excessive steam release to the extent that Class I function is impaired.

b. Pipe whip and steam/water jets following a pipe rupture of an adjacent pipe.

c. Earthquakes, by having the ability to sustain seismic accelerations adopted for purposes of plant design without loss of function. Protection from interaction with the surrounding buildings is accomplished by providing a separating joint of sufficient size for earthquake displacements. Unless the building is designed to Class I seismic design[1], an analysis is made to demonstrate that it will not collapse, otherwise the systems are protected locally.

d. Tornado wind loads.

e. Other natural hazards. Examples of these hazards are flood and ice.

f. Fire, in such a way that fire and operation of fire fighting equipment does not cause damage to redundant parts of the system.

g. Missiles from different sources. These sources comprise:

   1. Tornado created missiles.

   2. Missiles from components containing moving parts which could be subjected to overspeed. (Potential sources for such missiles are turbines and turbine generators, diesel engines, gas turbines.)

[1] The expression "Class I" used in this context is defined in Appendix B "Design of Nuclear Power Reactors Against Earthquakes" in a document entitled "Behavior of Structures During Earthquakes," Appendix A, by G. W. Housner, Professor of Civil Engineering, California Institute of Technology, Pasadena, California, published by American Society of Civil Engineers – Engineering Mechanics Division (October 1959 EM4).

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3. Missiles from high pressure steam and feedwater piping. (These missiles are limited to non back-seated valve stems and parts bolted to valves with bolts smaller than 3″.)

12.2.5.2 Support Description for Class I Items

The sketches shown on Figures 12.2-22, -23, -26, -27, -28, -29, -31 and -32 were inserted in the FSAR and carried over into the USAR in response to an AEC question regarding description of Class I equipment supports. These sketches are retained in the USAR for historical purposes only. For as-installed support details, the reader is referred to the appropriate controlled drawings. 01409787

a. Reactor Vessel Support (Figure 12.2-22)

   The Reactor Vessel is supported on six vertical steel H-Columns embedded in the biological shield concrete. The tops of these columns are furnished with ventilated support structures to provide for a suitable temperature gradient between the heated parts of the Reactor Vessel coming in contact with the supports and the supporting steel columns and surrounding concrete below.

   Fitted key slot blocks that are furnished with the reactor and bolted to the ventilated support structures provide for the free radial thermal expansion of the Reactor Vessel. Machined keys that are integral with the Reactor Vessel nozzles and support lugs are shimmed for sliding fit in the key slots and restrain the Reactor Vessel from movement in any horizontal direction. Thus the center point of the Reactor Vessel is rigid and has a “zero” movement.

   The tops of the steel H-Columns are connected together by means of a structural tee horizontal bracing system that is welded to a continuous outer steel band. This entire bracing system is embedded in concrete to provide a rigid anchorage. Stud anchors are welded to the flanges and web of the column length to assure the composite action of concrete in carrying the vessel loads thereby providing additional load carrying margins in the supporting structural system.

b. Steam Generators (Figure 12.2-26)

   Vertical support consists of four steel H-Columns hinged at each end to provide for unrestrained movement in the direction of thermal expansion. The column ends are securely bolted into the steam generator support lugs and anchored by embedded bolts at the base to provide for uplift forces that are postulated to occur during lateral loading of the support system.

   Two lateral levels of supports are provided for the lateral seismic and pipe rupture loads.

The lower lateral support provides restraint in three directions by means of close clearance bumper-pedestals that are mounted on the compartment walls. These bumper-pedestals were fitted and shimmed for close clearance during preoperational heatup.

The upper lateral support system consists of a horizontal ring girder that was fitted and shimmed to the contour of the steam generator shell with clearance allowance for the thermal expansion of the girder-shell system. The upper lateral support girder provides restraint in four directions by means of three attached bumper-pedestal plates that come in close clearance contact with wall mounted bumper plates in the hot position of the generator.

The fourth side of the girder is provided with a group of four hydraulic suppressors the longitudinal axis of which is in alignment with the direction of thermal expansion. The suppressors are attached to the girder and the compartment walls by means of pivoted linkage brackets to allow for vertical thermal displacement of the girder. The hydraulic suppressors provide for freedom of movement in the direction of thermal expansion during heat-up and cool-down and provide dampened restraint to the sudden loads of seismic action and pipe rupture.

The following tests have been performed on the steam generator snubber control valve manifold:

High velocity lockup and normal bleed
Low velocity lockup and no bleed

The manifold was shown to perform in accordance with the design intent (Reference 34)

The reactions of jet forces in the main steam line at the top of the steam generator are restrained by means of two cable anchors that are fitted with yokes welded to the pipe bends.

The reactions of jet forces in primary loop piping caused by pipe rupture are restrained by means of a heavy structural steel weldment brackets rigidly anchored at various points throughout the internal concrete structures.

c. Pressurizer (Figure 12.2-27)

The pressurizer is supported by the base concrete slab of the pressurizer vault and anchored thereto by means of 24 high-strength bolts equally spaced on the circumference of the pressurizer support skirt.

d. Reactor Coolant Pumps (Figure 12.2-23)

Vertical Support consists of three steel H-Columns hinged at each end for vertical support and uplift hold-down while providing unrestrained freedom of movement laterally in the direction of thermal expansion during heat-up and cool-down.

The connection between the top of the support columns and the pump support brackets is by means of a single high strength steel threaded rod. Between the support bracket and the top of the column are three high-strength tie bars with slotted holes at each end to accommodate the thermal expansion of the pump loop. The tie bars are anchored to the compartment walls and will prevent slipping of the pump in the event of a pipe rupture.

e. Control Room Equipment: NSS Panels (Figure 12.2-28)

These panels are welded to a steel base plate which is anchored to the control room concrete floor by means of embedded anchor bolts.

f. Control Room Equipment: Process Control, Radiation Monitor and NIS Control Panels (Figure 12.2-29)

These panels are bolted to a welded steel frame which is in turn bolted to the control room concrete floor by means of embedded anchor bolts.

g. Battery Rack Frames (see controlled drawing file)

These frames are cross-braced and bolted to the floor by means of embedded anchor bolts.

h. Switchgear Mounting Panels (Figures 12.2-31 and 12.2-32)

Two different methods are used to attach switchgears to the concrete floors. As depicted in Figure 12.2-32 the switchgear is plug welded to a steel channel which is fastened to the floor by means of embedded anchor bolts. The other method used as depicted in Figure 12.2-31 utilizes expansion bolts to fasten the switchgear to the concrete floor.

i. Primary Equipment Supports and Piping Restraints

The primary equipment supports and the piping restraints are provided to resist undesired motion due to postulated reactor coolant loop pipe ruptures where propagation of damage to required engineered safety features is possible. Also, propagation of damage to the unbroken loop or unbroken leg of the broken loop is prevented. 01409787

The sketches of the primary equipment supports and piping restraints for the reactor coolant loop are provided in Figures 12.2-22, -23, -26 and -27. 01409787

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12.2.6 Missile Protection Features

High-pressure Reactor Coolant System equipment, which could be the source of missiles, is suitably protected either by the concrete wall enclosing the reactor coolant loops or by the concrete operating floor to block any passage of missiles to the containment walls. The steam drum, which forms an integral part of the steam generator, represents a mass of steel which provides protection from missiles originating in the section of the containment within the shield wall and below the operating floor. A steel structure is provided over the RCCA drive mechanisms to block 01047869
any missiles generated from a hypothetical fracture of a RCCA housing.

Note that dynamic effect of missiles generated as a result of double-ended severance of a main coolant pipe, as discussed below, is no longer a design basis for the plant (see section 4.6.2.3 for more details).

Missile protection is designed to the following criteria:

a. The Reactor Containment Vessel is protected from loss of function due to damage by such missiles as might be generated in a loss-of-coolant accident for break sizes up to and including the double-ended severance of a main coolant pipe;

b. The engineered safety features systems and components required to maintain containment integrity are protected against loss of function due to damage by missiles.

During the detailed plant design, the missile protection concept necessary to meet these criteria were developed. These concepts are:

a. Components of the Reactor Coolant System are examined to identify and to classify missiles according to size, shape and kinetic energy for purposes of analyzing their effects.

b. Missile velocities are calculated considering both fluid and mechanical driving forces which could act during missile generation.

c. The structural design of the missile shielding takes into account both static and impact loads.

d. The Reactor Coolant System is surrounded by reinforced concrete and steel structures designed to withstand the forces associated with double-ended rupture of a main coolant pipe and is designed to stop these missiles.

Any removable slabs, blocks, or partitions which could become missiles during a tornado or earthquake were anchored by means of embedded inserts or bolts using load combinations and allowable stresses as established in Tables 12.2- 5, 12.2-6 and 12.2-7.

The type of missiles for which missile protection is provided are:

a. Valve stems

b. Valve bonnets

c. Instrument thimbles

d. Various types and sizes of nuts and bolts

e. Complete RCCA drive mechanisms, or parts thereof

f. Piece of pipe

g. Pressurizer valves, instrumentation thimbles and heaters.

The Containment polar crane is also regarded as a potential earthquake-produced missile. This crane was analyzed for seismic forces under maximum expected crane load conditions during refueling. The procedure of seismic analysis is treated in Section 12.2.2. The design provides for no loss of crane support or structural integrity during a DBE, and that no parts will shake loose.

To assure the stability and prevent toppling of the containment polar crane or its components during an earthquake, the following features have been incorporated in its design. First, the crane bridge is provided with four (4) up-kick lugs, one fastened to each truck, which pass under the building girder top flange to prevent the bridge from dislodging during seismic excitation. Second, the crane trolley is provided with two (2) overturning locks, one fastened to each truck, which pass around the top flange of the trolley rail to prevent the trolley from being dislodged from the bridge girder. Third, the structural steel and Reactor Containment Vessel wall supporting the crane are designed to withstand these earthquake-induced forces.

In addition, the bridge and trolley wheels are equipped with electrically activated, spring set brakes. Upon loss of power or when the crane or trolley are not under operator control, the springs activate the brakes locking the wheels firmly in place to prevent rolling. In the unlikely event of brake failure, positive wheel stops are provided to prevent the trolley from overtravelling and leaving the rails.

12.2.7 Turbine Missiles

12.2.7.1 Probability of Damage to Safeguard Equipment from Turbine Missiles

The probability of unacceptable damage to safeguards equipment resulting from turbine missiles, must be maintained less than $1x10^{-7}$ to satisfy the guidelines values of Section 3.5.3 of the Standard Review Plan. In the case of Prairie Island (unfavorably-oriented turbine), the product of the “strike” and “damage” probabilities is assumed to equal 10^{-2} (Reference 51); therefore, the probability of “ejection” of internal fragments must be less than 10^{-5} to meet the requirements of the Standard Review Plan.

The probability of “ejection,” is the sum of the probability of missile ejection at running speed, design overspeed and destructive overspeed. In the case of running speed and design overspeed, both low cycle and high cycle fatigue mechanisms have failure probabilities well below NRC accepted guidelines and are not controlling (Reference 52). As with previous designs, the potential for stress corrosion cracking has the greatest influence on rotor integrity at running speed and design overspeed. The fully integral rotor construction greatly reduces the chance of formation of stress corrosion cracks with the result that the probabilities of missile ejection at running and design overspeed are demonstrated to be less than $5x10^{-6}$ even for 36 years of 01353880 operation between inspection (Figure 12.2-38). In the case of destructive overspeed missile ejection, the probability is controlled by turbine valve test frequency as discussed in USAR Section 11.2.3.2 “Turbine Overspeed.”

The current methodology of calculating the total probability of “ejection,” and demonstrating compliance with NRC acceptance criteria is detailed in WCAP-11525. To be consistent with the established methodology, and good engineering judgement, a margin of $5x10^{-6}$ per year is considered adequate to cover the missile ejection probabilities at running and design overspeed. Therefore, the selected turbine valve test interval was checked to assure that the probability of destructive overspeed does not exceed $5x10^{-6}$ per year (i.e., total probability maintained less than $1x10^{-5}$). 01353880 WCAP-16054 was commissioned to re-check WCAP-11525. WCAP-16054 confirmed WCAP-11525 with the inclusion of recent valve failure data and increasing the valve exercise surveillance interval to 6 months. The total probability continues to be maintained at less than $1x10^{-5}$.

Following installation of the Unit 1 fully integral rotors in 1997 [Unit 2: 1998], through the remaining Operating License period in 2033 [Unit 2: 2034], the probabilities of missile ejection at running and design overspeed are shown to be less than $5x10^{-6}$, therefore it 01353880 is concluded that periodic in-service inspections are not required for the fully integral nuclear low pressure rotors to meet the requirements of the Standard Review Plan.

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12.2.8 Old Service Building

The Old Service Building is a three-story steel frame structure that is attached to the east side of the Auxiliary Building and abuts the south side of the Turbine Building. As shown in Figures 12.3-1, 12.3-2, and 12.3-3, the ground floor houses water treatment equipment and the D-1/D-2 Emergency Diesel Generators. Floors at Elev. 715′ and 735′ contain locker rooms, a lunch room, and work shops and tool rooms for plant maintenance personnel.

The D-1/D-2 Diesels are located in Design Class I reinforced concrete rooms within the Old Service Building. The Old Service Building's concrete foundation, exterior concrete flood walls, and concrete floor at Elev. 715′ have also been analyzed for Design Class I load combinations and satisfy Design Class I allowable stress limits. 03-011

The concrete floor at Elev. 735′ and supporting structural steel columns must meet Design Class I* requirements because a section of the D-1 Diesel exhaust line extends above Elev. 715′ and is routed below the Elev. 735′ floor slab. In addition to being able to withstand a Design Basis Earthquake, this part of the structure has been analyzed for and is capable of withstanding 360 mph tornado wind loads (Reference 15).

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12.2.9 Protection Against Crane Toppling

The Spent Fuel Pool Bridge Crane, Auxiliary Building crane and the Turbine Building crane are located in areas where they are subjected to possible damage from an earthquake. These crane bridges and trolleys are protected against tipping, derailment and uncontrolled movements that could possibly create damage.

To minimize risk, four levels of protection have been incorporated into crane design and operating procedures. First, the crane bridge and trolley are equipped with fixed, fitted rail yokes that allow free rolling but prevent the wheels from being lifted or derailed. Second, the bridge and trolley wheels are equipped with electrically activated, spring set brakes. Upon loss of power or when the crane or trolley are not under operator control, the springs activate the brakes, locking the wheels firmly in place to prevent rolling out of position. Third, in the unlikely event of brake failure, positive wheel stops and bumpers are provided to prevent the trolley and bridge from overtravelling and leaving the rails. Fourth, when not in use, parking of the Auxiliary and Turbine Building cranes is limited to specific locations where damage to the cranes during a tornado would not threaten spent fuel or safeguards equipment. These restrictions are not applicable to the Spent Fuel Pool Bridge Crane and Containment polar cranes, which are housed in reinforced concrete structures designed to withstand tornado missiles and wind loads.

Since the Containment polar crane can be a potential missile source during a design basis earthquake, protection against its toppling are discussed under Missile Protection Features in Section 12.2.6.

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12.2.10 Masonry Wall Design

NRC I.E. Bulletin No. 80-11 required identifying all masonry walls at the plant in proximity to or supporting safety related piping or equipment whose failure could affect a safety related system. Concrete masonry walls generally function as partition walls or as shield walls. Penetrations were permitted in the masonry walls for pipes and ducts. Loads on walls from safety and non-safety related equipment, ducts, piping, electrical conduits, etc., were permitted only if the loads were relatively insignificant. 01007802

12.2.10.1 Methodology

Masonry walls were designed in accordance with the 1967 Uniform Building Code and modeled as beams or plates. End connections were considered fixed, simply supported, or free based on the following criteria:

a. Fixed – Reinforcement is adequate to resist fixed end moments and shears. 01007802

b. Simply Supported – End connection has enough shear strength to prevent lateral moment and is also flexible enough to permit rotation.

c. Free – End of wall is either not connected to another wall or slab or the connection does not qualify as fixed or simply supported.

12.2.10.2 Loads

12.2.10.2.1 Normal Service Loads

Normal service loads include dead loads, live loads, wind loads, loads due to normal operating thermal gradients and thermal expansion of pipe, and loads due to normal room pressure differentials. Loads are combined by direct summation. 01007802

12.2.10.2.2 Seismic Loads

The floor accelerations and horizontal displacements were in accordance with Reference 6. The damping factor generally used was 4% of critical for load combinations involving operating basis earthquake (OBE) loads and 5% for design basis earthquake (DBE) loads. The design adequacy of the walls was governed by stresses resulting from a load combination including DBE loads. To compute inertia loads the acceleration for a wall was determined by taking average of acceleration of floors above and below the wall. Story Drift Loads were computed using the maximum horizontal relative displacement of the wall. 01007802

12.2.10.2.3 Other Loads

Loads due to tornadoes, accident pressure, pipe rupture, etc., were also included and combined in accordance with Table 12.2-4. 01007802

12.2.10.3 Load Combinations

The following load combinations were used:

a. Normal Service Loads

b. Normal Service Loads + Other loads 01007802

c. Normal Service Loads + Operating Basis Earthquake

d. 2/3 (Normal Service Loads + Design Basis Earthquake)

12.2.10.4 Design Criteria

Maximum allowable stresses on masonry walls are as defined in ACI 531-79, Building Code Requirements for Concrete Masonry Structures. 01007802

12.2.10.5 Evaluation and Modification of Masonry Walls

In response to IE Bulletin 80-11, all safety related masonry walls were evaluated, including those in proximity to or supporting safety related piping or equipment whose failure could affect a safety related system. Walls identified as safety related were reanalyzed using design criteria in Section 12.2.10.4. Of the one hundred and twenty-one (121) safety related walls that were analyzed, twenty-four (24) did not meet the criteria and required field modifications. 01007802

The evaluation of safety related walls did not take into account any future loadings. Design controls implemented during future modifications to pipe and/or equipment will consider these requirements.

Additional information on the subject of masonry wall design is given in References 26, 27, 28, 29 and 30.

12.2.11 Refueling Floor Laydown Area

The refueling floor laydown area in the Auxiliary Building provides additional room for temporary laydown during refueling outages and in-service inspections. The floor structure is designed to carry the expected live load, the dead weight, the trolley loads and the earthquake loads. All the existing structures affected by this addition are evaluated for their structural integrity under the added loads.

The load combinations used are in accordance with NRC Standard Review Plan 3.8.4.II.3. The applicable code allowables are as follows:

TYPE OF STRUCTURE	LOAD COMBINATIONS	ALLOWABLES
Concrete Slab (Seismic Category III)	a - (ii) (1) & (2)	ACI 318-71
Metal Deck (Seismic Category I)	a - (i) (1) & (2)	20 KSI
	b - (i) (4)	30 KSI
Steel Beams (Seismic Category I)	a - (i) (1) & (2)	AISC - 1969
	b - (i) (4)	0.9 Fy

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12.2.12 Control of Heavy Loads

12.2.12.1 Cranes Handling Heavy Loads

12.2.12.1.1 General

Four overhead handling systems handle heavy loads above safety-related components, components required for plant shutdown or decay heat removal. These cranes have been evaluated per Generic Letter 81-07 “Control of Heavy Loads” (References 18, 32, 33, 36, 37, 38, 39, 40 and 41). The evaluations of these cranes are summarized below.

The basis for conducting safe heavy load evolutions other than those specifically discussed in the USAR, is contained in the plant “Heavy Loads Program”.

The following cranes and hoists carry heavy loads at Prairie Island Plant Units 1 and 2:

a. Turbine Building Cranes (two 120 Ton)

b. Auxiliary Building Crane (one 125 Ton)

c. Containment Polar Cranes (two 230 Ton)

d. Spent Fuel Crane

12.2.12.1.2 Turbine Building Crane Evaluation

The turbine building cranes are generally used for the maintenance of turbine-generators and will carry the heavy loads identified in Table 12.2-40.

There is no safeguard equipment on the turbine floor itself. However, Unit 1 safeguards 4.16KV and 480V switchgear are located in a Class I aisle between column rows 8 and 10 below the operating floor. A load drop accident which may perforate the 18 inch thick concrete floor or cause excessive scabbing could potentially damage the buses.

Safeguards 4.16KV bus 15 is redundant to bus 16 and similarly 480V bus 111 is redundant to bus 121. The 4.16KV buses are located to the east of column row 9 and the 480V buses are located to the west of column row 9. Column row 9 is a 12″ thick concrete wall which reaches to the ceiling and provides support for the operating floor. Two 3′ by 3′ columns are also provided along column row 9 to support the operating floor.

Based on the physical separation of the buses and procedural requirements, a load drop accident is very unlikely to damage both redundant systems. Also, the crane has been modified by addition of redundant limit switches to prevent “two block” accidents.

Additionally, the area of the Turbine Building operating floor above the Unit 1 safeguards switchgear has been marked as a heavy load exclusion area. Carrying of heavy loads above this area requires a specific procedure.

The 120 Ton Turbine Building Cranes were purchased from Whiting Corporation of Illinois in the late sixties. The specification against which these cranes were purchased predates CMAA Specifications #70. However, the cranes were qualified against EOSI Specification #61 and USAS B30.2-1967. A review verified the cranes substantially meet the requirements of CMAA Specification #70.

12.2.12.1.3 Auxiliary Building Crane Evaluation

The crane (see Section 10.2.1.2.2) is designed primarily for receiving new fuel, or handling spent fuel casks in the spent fuel pool. The crane is also used to lift heat exchanger hatches, tube bundles, in-service inspection tools, and miscellaneous maintenance related equipment.

The safeguard equipment that is located within the travel path of the crane includes:

a. Spent fuel pool for the storage of spent fuel, new fuel, and casks for spent fuel.

The Boric Acid Storage Tanks are located in a Class I room below the 755′ elev. floor. A load drop accident which may perforate the 12″ thick concrete floor or cause excessive scabbing could potentially damage the tanks. This area at the 755′ elev. floor is indicated as a heavy load exclusion area. Therefore, movement of heavy load over the BAST rooms is controlled by the Heavy Loads Program requirements.

Additionally, the Seal Water Heat Exchangers and Letdown Heat Exchangers are located below removable shield blocks at elev. 735′. The only heavy loads in this area are the removable shield blocks (16,000 lbs) or the heat exchangers themselves. The removable blocks are only removed to service the heat exchangers and can only be done when the equipment is out of service. Furthermore, neither heat exchanger is required for safe shutdown of the plant or for decay heat removal.

The fuel pool enclosure is a Class I reinforced concrete building with 12 to 18 inch thick walls and roof, which is integrally connected to the fuel pool structure. The fuel pool enclosure which covers both new and spent fuel storage facilities is completely contained in the Auxiliary Building and is beneath the Auxiliary Building Crane. The fuel pool enclosure is provided with crane access slots and equipment handling doors which physically limits the area of spent fuel pool over which spent fuel casks or heavy objects can be moved. There are no routine operations which require opening of the access slots. The handling of spent fuel casks is described in Section 10.2.1. When handling any load less than the spent fuel cask, the use of the 125 ton single-failure-proof main hoist, with rigging in accordance with Technical Specification 4.19, eliminates the concern of load drop. If the 15 ton auxiliary hoist is used over the spent fuel pool, a Plant Operating Review Committee reviewed procedure is required for handling of the smaller loads to ensure that the criteria of NUREG-0612 are satisfied with respect to pool structural considerations (Reference 57).

The height of the pool enclosure roof relative to that of the Aux. Bldg. crane physically restricts movement of large objects over it. Even with the physical restriction certain loads can be moved over the enclosure roof and this load path is used for the movement of equipment from the south side of the enclosure to the north side and vice versa. Load drop analysis has provided a limit on the size of the load which can safely be moved over the roof and the movement is controlled by procedure (Reference 50).

The Aux. Bldg. crane is also used in the handling of new fuel shipping containers. The areas where these are handled are not in the vicinity of safety related equipment. Also, new fuel assemblies do not qualify as heavy loads as defined in NUREG-0612 and movement of new fuel assemblies does not require movement over spent fuel in the pool.

An accidental load drop while handling heavy loads over the spent fuel pool could impact irradiated fuel or damage the pool structure with the potential for excessive offsite releases, inadvertent criticality, or loss of water inventory in the spent fuel pool. NUREG-0612, “Control of Heavy Loads at Nuclear Power Plants”, discusses the potential consequences of a load drop over spent fuel and provides guidelines to mitigate these potential consequences.

Section 5.1 of NUREG-0612 describes various alternative approaches which provide acceptable measures for the control of heavy loads. The objectives of the guidelines in Section 5.1 of NUREG-0612 are to assure that (1) the potential for a load drop is extremely small, or (2) to show by analysis that the consequences of a load drop are acceptable.

Section 5.1.1 of NUREG-0612 provides seven general guidelines for handling heavy loads that could be brought in proximity to or over safe shutdown equipment or irradiated fuel in the spent fuel pool area. The seven guidelines are intended to assure reliable operation of the handling systems, and to the extent practical, minimize the movement of heavy loads over irradiated fuel. Controls satisfying the guidelines of Section 5.1.1 of NUREG-0612 were implemented at Prairie Island as described in the responses to NUREG-0612 (References 18, 36, 37, 38, 39, 40 and 41).

In a safety evaluation for Prairie Island License Amendments 48 and 42, dated May 13, 1981, the NRC Staff imposed a temporary limitation on the movement of heavy loads, which included spent fuel casks, over spent fuel pool No. 1 (i.e., “No heavy loads will be transported over or placed in either part of the spent fuel pool when irradiated fuel is stored in that part.”). The primary purpose of that restriction was to protect the fuel assemblies located in pool No. 1 from damage due to the potential dropping of heavy loads during the construction period of the spent fuel pool rerack project. Following completion of the rerack project, the concern for movement of heavy loads over spent fuel pool No. 1 was reduced to movement of spent fuel casks.

Section 5.1.2 of NUREG-0612 provided guidance designed to assure that the evaluation criteria of Section 5.1 of NUREG-0612 were met for load handling operations in the spent fuel pool areas of pressurized water reactors. Section 5.1.2 stated that, in addition to meeting the guidelines of Section 5.1.1, one of four additional conditions should be met. Two of those additional conditions are the following:

1. The overhead crane and associated lifting devices used for handling heavy loads in the spent fuel pool area should satisfy the single-failure-proof guidelines of Section 5.1.6 of NUREG-0612, or

2. The effects of drops of heavy loads should be analyzed and shown to satisfy the evaluation criteria of Section 5.1 of NUREG-0612.

In order to remove the temporary restriction on movement of heavy loads over spent fuel pool No. 1, imposed by the May 13, 1981 safety evaluation, a load drop analysis for shipping casks to be carried over the spent fuel pool was submitted to the NRC by letters dated December 21, 1984 and March 14, 1985. This analysis evaluated a postulated drop of a spent fuel cask in accordance with the guidelines of NUREG-0612. The analysis included, as prescribed in Section 5.1 of NUREG-0612, a confirmation that a postulated cask drop would not result in 1) fuel damage which causes an unacceptable release of radioactivity, 2) a criticality condition within the storage racks, 3) uncovery of stored spent fuel, or 4) damage to safe shutdown equipment.

The cask drop analysis was found acceptable in the safety evaluation for License Amendments 74 and 67, dated June 26, 1985. Those License Amendments amended Technical Specification Sections 3.8.B and 5.6 (now IT.S. 3.7.16, 4.3, and TRM) to impose restrictions that would assure that the assumptions of the cask drop analysis would remain bounding during the handling of a spent fuel cask. The amendments included restrictions on spent fuel boron concentration, the use of an impact limiter or crash pad, the operability of crane travel interlocks and mechanical stops, and the age of the fuel stored in pool No. 1.

In an application dated August 31, 1990, Northern States Power Company applied for a license to construct and operate a Dry Cask Independent Spent Fuel Storage Installation to be located on the Prairie Island Site. The license was requested for the storage of spent fuel from the Prairie Island Nuclear Generating Plant in Transnuclear TN-40 storage casks.

However, because of the height of the TN-40 storage casks, it was not practical to use an impact limiter or crash pad during the handling and loading of those casks. Therefore, Northern States Power contracted with Ederer Incorporated to modify the auxiliary building crane in accordance with the single-failure-proof criteria of Section 5.1.6 and Appendix C of NUREG-0612. This modification consisted of replacement of the existing bridge trolley with an Ederer designed trolley and associated hoists. The upgrade of the auxiliary building crane to a single-failure proof-design was completed in 1992.

This upgrade to the auxiliary building crane was made to provide a handling system for handling heavy loads in the spent fuel pool area that satisfies the single-failure-proof guidelines of Section 5.1.6 of NUREG-0612, and thus eliminate the need to analyze the effects of drops of heavy loads per the evaluation criteria of Section 5.1 of NUREG-0612. The 125 Ton Auxiliary Building crane was purchased from Whiting Corporation of Illinois in the late sixties. The specification against which these cranes were purchased predates CMAA Specifications #70. However, the cranes were qualified against EOSI Specification #61 and USAS B30.2-1967. A review verified the cranes substantially meet the requirements of CMAA Specification #70. The Auxiliary Building Crane was upgraded to single failure proof (per NUREG-0554). Because the installation of a single-failure-proof handling system eliminates the need for a cask drop analysis, the Technical Specification restrictions designed to mitigate the effects of a cask drop are no longer required. A License Amendment Request was submitted (Reference 42) to revise the Technical Specifications to eliminate the cask handling restrictions imposed by Amendments 74 and 67 and replace them with requirements for the use of a single-failure-proof handling system in the handling of spent fuel casks. The upgrade of the auxiliary building crane and use of a single-failure-proof handling system for handling spent fuel casks was found acceptable by the NRC in their safety evaluation for License Amendments 99 and 92, dated July 9, 1992 (Reference 44).

12.2.12.1.4 Containment Polar Crane Evaluation

The containment polar crane is used for handling of missile shields, reactor vessel head, upper and lower internals, in-service inspection tools, reactor coolant pump components and pressurizer safety valves.

The crane is operated in the Reactor Building under the following restrictions:

a. With the reactor head removed, the Reactor Building crane, main or auxiliary, load blocks with attached loads greater than 1799 pounds shall not be moved within 15 horizontal feet of the irradiated fuel without specific written procedures.

b. With the pressurizer missile shield removed, the Reactor Building crane load blocks with attached loads greater than 1799 pounds shall not be moved over the open pressurizer vault except during defueled conditions with the fuel transfer tube gate valve closed.

c. A load drop analysis was completed for lifting the Reactor Vessel Head over irradiated fuel in an open vessel. The bottom of the reactor vessel head shall not be lifted higher than an elevation of 756.5 feet (Reference 56).

The residual heat piping is protected from heavy load drop hazards by structural members of overhead floor levels; therefore, heavy loads may be moved over the reactor coolant pump vaults without administrative control.

Safe shutdown equipment is that equipment required for continued decay heat removal and for maintaining the plant shutdown. The steam generators and/or residual heat removal systems are required per TS 3.1. Since no single load can be carried over both steam generators, the auxiliary feedwater, main feedwater and steam piping in containment are considered part of the safe load path.

With the reactor coolant system temperature above Mode 5, Cold Shutdown, the Containment Building polar crane will not be used for moving loads without specific written procedures.

During heavy load lifts over the open fueled reactor vessel, at least one isolation valve will be closed in each line penetrating the containment which provides a direct path from the containment atmosphere to the outside, except as described below.

Movement of the reactor vessel head and upper internals over the open fueled reactor after the fuel has been subcritical in excess of seven days is permissible with containment atmosphere open to the outside with the following restrictions (Reference 61):

a. The bottom of the upper internals shall not be lifted higher than an elevation of 735′-6″.

b. Administrative controls shall be in place to provide containment closure within 1 hour of a dropped load.

c. Auxiliary Building Special Ventilation Zone (ABSVZ) boundary integrity shall be established. 01514406

Item c above ensures that the plume from this event does not migrate from the containment, through a compromised ABSVZ boundary, and into the Auxiliary Building Normal Ventilation System exhaust. This path would compromise the assumption that the Common Area of the Auxiliary Building is the limiting release path for this particular load drop event.

The 230 Ton Reactor Building Cranes were purchased from Whiting Corporation of Illinois in the late sixties. The specification against which these cranes were purchased predates CMAA Specifications #70. However, the cranes were qualified against EOSI Specification #61 and USAS B30.2-1967. A review verified the cranes substantially meet the requirements of CMAA Specification #70.

12.2.12.1.5 Spent Fuel Crane Evaluation

The spent fuel crane (see Section 10.2.1.2.2) is used for the handling of new and spent fuel assemblies (by means of a long handling tool suspended from a hoist), divider gates, and pool covers. The travel of hoist and tool length are designed to limit maximum lift of a fuel assembly to a safe shielding depth.

The pool divider gates are used to seal the fuel transfer openings located between pools 1 and 2 and between each pool and the fuel transfer canal. Each gate is a stainless steel device 27′-6″ x 2-2″ x 2/3″ and weighs approximately 2620 pounds. The Pool No. 1 protective covers are designed to prevent a heavy load drop from impacting any fuel stored in the small pool. Each cover is 20 ft x 6.5 ft and weighs 4550 pounds.

In addition to the procedure for fuel handling and divider gates, detailed procedures are written for each lift of the pool covers by the crane (not higher than 6″ with two hoists).

The West Hoist is a redundant, 3-ton design rated load (DRL), 3700 pound maximum critical load (MCL) hoist used for general fuel handling, Unit 2 refuelings, and heavy load lifting. The hoist is specifically designed for moving the heavy loads contained in the spent fuel pool enclosure, e.g., the pool divider gates (2600 lbs)

12.2.12.2 Sling Evaluations

The slings used with these cranes comply with design and inspection requirements of ANSI B30.9, 1971. As discussed in plant procedure D58 “Control of Heavy Loads”, the slings will have a minimum factor of safety of five (5) and the rated capacities of the slings shall be taken as those listed in Tables 3 thru 14 of ANSI B30.9.

The operation of the cranes is controlled by administrative procedures which include inspections for safe operating practices with wire slings. Slings shall be visually inspected each day that they are used. The conditions of their replacement and/or repairs comply with requirements of Section 9.2.8 of ANSI B30.9-1971.

The design load rating of the slings is based upon the maximum static and dynamic loads. Analysis has shown that the loading due to dynamic loads is very small. Assuming that the design load for a particular sling is based solely on static loads, the combined dynamic and static load very closely equals the static design load of the slings. As discussed in Reference 42, administrative controls will ensure that lifting devices not specifically designed and that are to be used with the auxiliary building crane for handling of heavy loads over safe shutdown equipment or spent fuel in the spent fuel pool will meet the requirements of guideline (1)(b) of Section 5.1.6 of NUREG-0612.

12.2.12.3 Special Lifting Devices Evaluation

The Special Lifting Devices for lifting heavy loads in the plant consist of the Turbine Spreader Assembly, the Upper Internals Lifting Rig, and the Reactor Head Lifting Rig (these are included in Table 12.2-40). Visual inspection and non-destructive examination (NDE) testing of major load carrying welds and any other critical areas is performed consistent with ANSI N14.6-1978 (Reference 73) annually or prior to each use of the Special Lifting Device, typically at each refueling outage (References 18, 37, 38, 39, and 32). 01513991

As discussed in References 42 and 45, all special lifting devices to be utilized with the auxiliary building crane in the handling of heavy loads over safe shutdown equipment or spent fuel in the spent fuel pool will be designed to meet the requirements of ANSI N14.6 1986, “Standard For Special Lifting Devices for Shipping Containers Weighing 10,000 Pounds or More For Nuclear Materials”, as outlined in guideline (1)(a) of Section 5.1.6 of NUREG-0612.

The special lifting device utilized in handling the TN-40 and TN-40HT spent fuel storage casks at Prairie Island meets the requirements of ANSI N14.6 1986, as outlined in guideline (1)(a) of Section 5.1.6 of NUREG-0612. The testing plan for the TN-40 special lifting device is outlined in Reference 45. Except as noted in Reference 45, testing of the TN-40 special lifting device conforms with the requirements of ANSI N14.6 1986. The testing plan described in Reference 45 was found to be acceptable by the NRC in Reference 46.

12.2.12.4 Inspection, Testing and Maintenance of Cranes

Procedures for inspection, testing and maintenance of the cranes that are in use satisfy the criteria of ANSI B30.2-1976, Chapter 2-2.

12.2.12.5 Crane Operator Qualification

Crane Operators are qualified to operate overhead cranes as required by ASME B30.2.

12.2.13 Administration Building

The old administration building was constructed as part of the original plant construction project. In order to meet present and future space requirements for offices, storage areas, lockers and restroom facilities, lunchrooms and conference rooms, an addition was constructed. This addition consists of a five story structure, lower level at grade, along the north, east and west sides of the old administration building. The addition matched existing floor elevations on the first story (grade) and at the fourth story (El. 735). The building addition is constructed of reinforced concrete foundations and floors, structural steel frame and an insulated steel roof deck.

The column footings of the administration building addition impose new loads on the existing underground circulating water piping and the underground cooling water piping between the screenhouse and the turbine building. The effects of new loads on these piping were investigated. The investigation (Reference 53) has shown for the 00003
circulating water and the cooling water piping, that the new loads when superimposed on the existing soil load produce external pressure on the piping smaller than the allowable external pressure.

The connections between the administration building addition and the old administration/turbine building were also investigated for transfer of seismic and wind loads. Seismic and wind loadings are transferred to the turbine building through connections at elevations above the foundation level. The lateral loads transferred through the connections between the existing and the new building were determined using criteria established in Section 12.2.1. The stresses imposed on the existing structure due to the new addition were all found to be within the allowable values.

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12.2.14 Alternative Seismic Design Adequacy Verification Procedure 01-065

Revision 3 of the Generic Implementation Procedure (GIP) (Reference 58), as modified and supplemented by the U.S. Nuclear Regulatory Commission Supplemental Safety Evaluation Report No. 2 (SSER No. 2) (Reference 59) and SSER No. 3 (Reference 60), may be used, except as noted below, as an alternative seismic adequacy verification method to the existing seismic qualification methods for the seismic design and verification of modified, new and replacement equipment classified as Design Class I or Class 1E. 01-065

This alternative method may not be used for those SSCs where prior commitments to using later seismic qualification methods after initial operation are made. These SSCs include: 01-065

1. Class 1E equipment identified in USAR Section 7.9.2.3.

2. Category 1 Post Accident Monitoring Instrumentation (as defined in USAR section 7.10.2) identified in USAR Tables 7.10-1 & 7.10-2 that has been seismically qualified in accordance with Regulatory Guide 1.100. 01-065

Only those portions of the GIP, as supplemented by SSER No. 2 and SSER No. 3, that pertain to the seismic design and verification of modified, new and replacement mechanical and electrical equipment, electrical relays, tanks and heat exchangers, and cable and conduit raceway systems are applicable for this alternative method. 01-065

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12.2.15 Use of USD Methodology to Evaluate Concrete Structures

Prairie Island's concrete structures were originally designed using Working Stress Design (WSD) methodology as set forth in ACI 318. About 15 years later, in April 1984, during a routine plant modification, it was belatedly discovered that concrete floors were never evaluated for vertical seismic loads. When the additional loads were considered, concrete stresses often exceeded WSD stress limits. However, using Ultimate Strength Design (USD) methodology and structural criteria in ACI 349 (Reference 20), the floors were found to be capable of withstanding the additional loads. Thus, this was the methodology used to verify the acceptability of plant floors and floor load limits on drawings.

05-049

In total, USD methodology was used to evaluate and verify the acceptability of 239 concrete members (floor slabs and beams) in the Auxiliary Building, 24 members in the Screenhouse, and 9 and 24 members in the Unit 1 and Unit 2 Reactor Buildings, respectively. For the Reactor Buildings, load factors were as prescribed in Section 3.8.3 of the NRC Standard Review Plan (Reference 21). For the Screenhouse and Auxiliary Building, they were as prescribed in Section 3.8.4 of the NRC Standard Review Plan (Reference 22). Both SRP Sections refer to ACI 349 for general USD methodology and specific structural acceptance criteria.

05-049

Later, in 1990, the new D5/D6 Building was designed using USD methodology described in the SBO/ESU Project Design Report (Reference 43). Load factors were as prescribed in Section 3.8.4 of the NRC Standard Review Plan. General USD methodology and specific structural acceptance criteria were as specified in ACI 349 and referenced in USAR Table 12.2-41. Regulatory endorsement of the USD methodology described in the Design Report was received in April 1992 (Reference 23).

05-049

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12.3 SHIELDING & RADIATION PROTECTION

12.3.1 Design Basis

12.3.1.1 General

Radiation shielding is designed for reactor operation at maximum calculated thermal power and to limit the normal operating radiation levels at the site boundary to below those levels allowed for continuous non-occupational exposure. The plant is capable of continued safe operation with 1% fuel element defects.

In addition, the shielding and containment provided ensures that in the unlikely event of a maximum design accident, the subsequent offsite radiation exposures will be well below the criteria in 10CFR50.67. 01406407

12.3.1.2 Radiation Exposure of Personnel

Radiation exposure of the personnel is a function of both radiation level and occupancy time in radiation areas. Total permissible radiation dose to be received by personnel in nuclear industries is defined by 10CFR20, “Standards for Protection Against Radiation”. The exposure of personnel to concentrations of radioactive materials in air or water above the contributions from natural background is limited to values in Appendix B of 10CFR20.

Under these regulations, the annual maximum permissible dose is limited to 5 Rem Total Effective Dose Equivalent (TEDE). Under normal operating conditions personnel exposure would be expected to be less than the maximum allowed. 01388733

All radiation and high radiation areas are appropriately marked and isolated in accordance with 10CFR20 and other applicable regulations.

12.3.1.3 Primary Shielding

The primary performance objective of radiation shielding within the Reactor Containment Vessel is to minimize the exposure of plant personnel to radiation emanating from the reactor and auxiliary systems. The radiation levels prevalent during plant operation, as well as those experienced upon shutdown, are recognized in the determination of the shielding requirements, as described below.

The secondary performance objective of the radiation shielding is to minimize radiation effects upon operating equipment as described below.

The primary shielding is designed to:

a. Reduce the neutron fluxes incident on the reactor vessel to limit the radiation induced increase in transition temperature.

b. Attenuate the neutron flux sufficiently to prevent excessive activation of plant components.

c. Limit the gamma flux in the reactor vessel and the primary concrete shielding to avoid excessive temperature gradients or dehydration of the primary shield.

d. Reduce the residual radiation from the core, reactor internals and reactor vessel to levels which will permit access to the region between the primary and secondary shields after plant shutdown.

e. Reduce the radiation leakage contribution to obtain optimum division of the shielding between the primary and secondary shields.

12.3.1.4 Secondary Shielding

The main function of the secondary shielding is to attenuate the radiation originating in the reactor and the reactor coolant. The major source in the reactor coolant is the Nitrogen-16 activity.

The secondary shielding will limit the full power dose rate outside the containment building from radioactivity inside the containment to less than 1 mRem/hr. The shield building also serves as an accident shield.

12.3.1.5 Fuel Handling Shielding

The fuel handling shielding permits the safe removal and transfer of spent fuel assemblies and control rod clusters from the reactor vessel to the spent fuel pit. It is designed to attenuate radiation from spent fuel, control clusters, and reactor vessel internals to less than 2.5 mRem/hr at the refueling cavity water surface and less than 1.0 mRem/hr in the auxiliary building.

12.3.1.6 Auxiliary Shielding

The function of the auxiliary shielding is to protect personnel working near various system components in the Chemical and Volume Control System, the Residual Heat Removal System, the Safety Injection System, the Waste Disposal System and the Sampling System. The shielding provided for the auxiliary building is designed to limit the dose rate to less than 1 mRem/hr in normally occupied areas, and at or below 2.5 mRem/hr in periodically occupied areas.

12.3.1.7 Fuel and Waste Storage Radiation Shielding

Criterion: Adequate shielding for radiation protection shall be provided in the design of spent fuel and waste storage facilities. (GDC 68)

Auxiliary shielding for the Waste Disposal System and its storage components is designed to limit the dose rate to levels not exceeding 1 mRem/hr in normally occupied areas, to levels not exceeding 2.5 mRem/hr to periodically occupied areas and to levels not exceeding 15 mRem/hr in controlled occupancy areas.

12.3.2 Description

12.3.2.1 Radiation Zoning and Access Control

Plant personnel are protected from radiation exposure by adequate shielding, monitoring, and operating procedures. The plant is divided into two major areas, the radiation control area, defined by the controlled access boundary shown on Figures 12.3-1 through 12.3-7 and the non-control area. Radiation Control Area encompasses all plant areas in which significant radiation and radioactive materials are present. Access to the Radiation Control Area (RCA) is administratively controlled. Entry to and normal exit from the Radiation Control Area is through the designated access control point only. Two areas of very high exposure rates are the incore thimble chase area while the irradiated thimbles are withdrawn, and the reactor head area during power operation. Additional procedures are developed for control of entry to the incore thimble chase and the reactor vessel head area.

12.3.2.2 Plant Shielding Areas

12.3.2.2.1 Primary Shielding

The primary shielding consists of the reactor internals, the reactor vessel wall, and a concrete structure surrounding the reactor vessel.

The primary shielding immediately surrounding the reactor vessel consists of a reinforced concrete structure extending from the base of the containment to an elevation of 69.0 feet. The lower portion of the shield is a minimum thickness of 7.0 feet of concrete and is an integral part of the main structural concrete support for the reactor vessel. It extends upward to the operating floor, forming a portion of the refueling cavity. This cavity is approximately rectangular in shape, and has concrete sidewalls which are five feet thick adjacent to areas in which fuel is transported.

The primary concrete shielding is air cooled to prevent overheating and dehydration from the heat generated by radiation absorption in the concrete. Eight “windows” have been provided in the primary shield for insertion of the out-of-core nuclear instrumentation. Cooling for the primary shield concrete, nuclear instrumentation, and vessel supports is provided by circulating 25,000 cfm of containment air between the reactor vessel wall and the surrounding concrete structure. The primary shield neutron fluxes and design parameters are listed in Table 12.3-1.

12.3.2.2.2 Secondary Shielding

The secondary shield surrounds the containment vessel, the reactor coolant loops and the primary shield. It consists of interior walls within the containment building, the operating floor, and the shield building. The shield building also serves as the accident shield.

The total thickness of concrete provided for this function in the area above grade is five feet eight inches. Of this, approximately three feet is concrete wall inside the containment vessel immediately surrounding the reactor coolant loops and two feet six inches is in the shield building wall. This shielding will reduce the radiation intensity at the outside surface of the shield building to a negligible level during normal plant operation.

The shield building consists of the two feet six inches reinforced concrete cylinder capped by a shallow, reinforced concrete dome two feet thick. Supplemental shielding has been provided for the containment penetrations where required. Section 5 provides a detailed discussion of the shield building design. The secondary shield design parameters are listed in Table 12.3-2. The equipment access hatch is shielded by a 2 ft-6 in thick concrete shadow shield. The control room is protected with concrete sidewalls two feet thick, and a concrete roof two feet thick.

The accident shield design parameters are listed in Table 12.3-3.

12.3.2.2.3 Fuel Handling Shielding

In 1980, the NRC requested all Westinghouse PWR licensees (References 54 and 55) to amend their Technical Specifications to state that the water level in the refueling cavity shall not be less then 23 feet above the reactor vessel flange during movement of fuel assemblies or control rods. The Technical Specifications were amended to meet this request.

The refueling cavity is formed by the upper portions of the primary shield concrete, and other sidewalls of varying thicknesses. A portion of the cavity is used for storing the upper and lower internals packages; these are shielded with concrete walls five feet thick. The remaining walls vary from 4 to 6 feet thick, and provide the shielding required for handling spent fuel.

The water level during refueling operations is maintained at the minimum elevation of 752′-6″ which is 23 feet above the reactor vessel flange (El. 729′-6″). This height ensures that approximately 8′-6″ of water will be above the top of a withdrawn irradiated fuel assembly. Under these conditions, the ALARA dose rate objective of not greater than 2.5 mRem/hr above the background dose rate at or near the water surface is maintained.

The spent fuel assemblies and RCC assemblies are remotely removed from the reactor containment through the horizontal spent fuel transfer tube and placed in the spent fuel pool. Concrete, 5 ft. thick, shields the spent fuel transfer tube. The exposed ends of the transfer tube in containment and the Auxiliary Building are submerged in 37 feet of borated water during refueling operations. The transfer tube passage through the Containment/Shield Building annulus is shielded with six-foot thick concrete. This shielding is designed to protect personnel from radiation during the time a spent fuel assembly is passing through the main concrete support of the reactor containment and the transfer tube.

Radial shielding during fuel transfer is provided by the water and concrete walls of the fuel transfer canal. An equivalent of 6 feet of concrete is provided to ensure the ALARA dose rate objective of not greater than 2.5 mRem/hr above background in the auxiliary building areas adjacent to the spent fuel pool.

Spent fuel is stored in the spent fuel pools which are located between the Unit 1 and Unit 2 containment buildings. Radial shielding for the spent fuel is provided by 5 feet thick concrete walls. The pool is flooded with borated water to a level such that the water height above the stored fuel assemblies is approximately 25 ft.

The refueling shield design parameters are listed in Table 12.3-4.

12.3.2.2.4 Auxiliary Shielding

The auxiliary shield consists of concrete walls around certain components and piping which process reactor coolant. In some cases, the concrete block walls are removable to allow personnel access to equipment during maintenance periods. Each equipment compartment is individually shielded so that compartments may be entered without requiring to shutdown and, possibly, to decontaminate the adjacent system.

The shield material provided throughout the auxiliary building is concrete. The principal auxiliary shielding provided is tabulated in Table 12.3-5.

12.3.3 Performance Analysis

12.3.3.1 Personnel Protective Equipment

All personnel entering the Radiation Control Area may be required to wear protective clothing. The nature of the work to be done and the areas involved governs the selection of protective clothing to be worn by individuals.

Caps, lab coats and shoe covers are available for routine operations. Specialized apparel such as coveralls, plastic or rubber suits, rubbers or boots, face shields, hoods, cotton or rubber gloves, and respirators may be required in the contaminated areas. In all cases, radiation protection personnel shall evaluate the radiological conditions and specify the required items of protective clothing to be worn.

Respiratory device use is evaluated prior to personnel exposure in airborne radioactivity areas with the objective of minimizing the Total Effective Dose Equivalent (TEDE). In all cases, radiation protection personnel evaluate the radiological conditions and specify the type of respiratory protective equipment required. Respiratory devices available include Self Contained Breathing Apparatus (SCBA).

12.3.4 Design Review of Plant Shielding During and After a Design Basis Accident (DBA)

The design review of plant shielding was performed to conform with the requirements of NUREG-0737. During and after a DBA, certain systems may contain highly radioactive fluids. The resulting dose rates in the vital areas of the plant may unduly limit personnel occupancy. The design review identified these vital areas and determined the dose rates to these areas from systems that may, as a result of an accident, contain highly radioactive materials. Corrective actions required to limit the dose to an operator to 5 Rem whole body or its equivalent to any part of the body have also been identified.

The result of the design review of plant shielding was contained in a report “Design Review of Plant Shielding and Environmental Qualification of Equipment for Spaces/Systems Outside Containment Which May Be Used in Post Accident conditions” (Reference 19). NRC approval of the Design Review was provided in Reference 31.

Numerous modifications were made to the plant to address the recommendations of the plant shielding design review. Localized shielding walls and panels constructed of concrete, lead or masonry were installed by the following modifications:

80Y099	Shield containment spray pump room openings
80Y102	Shielding on elevation 715′ above containment spray pump rooms
80Y127	Relocate safety injection pump discharge lines
80Y149	Shield walls for safety injection pump areas and stairwells to elevation 715′
80Y157	Shield Auxiliary Building and Shield Building special ventilation system charcoal filter beds

This design review was re-evaluated for impact due to fuel changes and to incorporate power level uncertainty of 2%. (Reference 69) No additional plant modifications were identified due to the re-evaluation.

12.4 RADIOACTIVE MATERIALS SAFETY

12.4.1 Materials Safety Program

The radioactive materials safety program provides reasonable assurance that byproduct, source, and special nuclear material will be stored, used, and accounted for in a manner which meets the applicable radiation protection provisions of 10CFR Parts 20, 30, 40, and 70. Provisions for controlling access to the radiologically controlled area are described in Section 12.3.2.1.

When not in use, access to the fuel storage area may be secured by lock or bulkhead closure. Another security measure employed is routine surveillance by operations personnel who are available 24 hours per day or by a security guard when operating personnel are not available. When in use, a member of the Operations Department is always present in the area. Use of the area will be restricted to fuel handling operations. Maintenance activities and material passage through the storage area using the auxiliary building crane is only allowed under direct procedural control.

This fuel storage area is constructed of concrete and steel and the use and storage of combustible materials in the storage area is strictly controlled. Fire fighting equipment is available in or near the area.

Fuel accountability, receipt, and shipment are conducted in accordance with written procedures contained in the Operations Manual. The new fuel technical evaluation procedure contains instructions on how to handle the new fuel from unloading until it is placed in the fuel storage area. The fuel shipment procedure contains instruction on transferring fuel assemblies from their storage locations to fuel shipment containers, and the transfer of these loaded containers to the common carrier for disposition off-site.

Procedures for shipment of irradiated fuel are prepared as the need for their use arises. For each fuel shipment offsite, a “Fuel Shipment Plan” will be written by the Engineer, Nuclear. This document describes the proposed fuel shipment, explaining which components are to be shipped, in which containers, the present status of the components, the approximate date and order of shipment, the common carrier and the receiver. The Fuel Shipment Plan is reviewed by the Plant Operating Review Committee prior to commencement of the shipping operation. 05-006

The fuel accountability procedures define the methods used for accountability of fuel assemblies and neutron source assemblies. The plant reporting procedures define the methods used for preparing and maintaining reports concerning Special Nuclear Material in accordance with 10CFR70.

The fuel accountability system is designed to meet the following criteria:

a. Locations of all core components shall be known at all times.

b. It shall be possible to assemble a chronological history of each core component’s movement on site.

c. SNM accountability to the NRC, fuel vendors and Northern States Power Company shall be satisfied.

12.4.2 Facilities and Equipment

Two laboratory facilities are available at the plant. One facility is located in the Turbine Building (a non-controlled area) for “cold” chemistry work. The other facility is located in the Auxiliary Building (a radiological controlled area) for “hot” chemistry work. This facility includes the radiochemistry laboratory, sample room and counting room. The radioactive source storage is also located in the Auxiliary Building.

The sample room and radiochemistry laboratory sample hoods are equipped with particulate, absolute and charcoal filters, which exhaust to the Auxiliary Building stack. The counting room, radiochemistry lab and sample room atmosphere exhausts to the general Auxiliary Building environment.

Survey and measuring instruments are described in Section 7.5.4.3. The plant radiation monitoring devices are described in Section 7.5.1.

12.4.3 Personnel and Procedures

Plant personnel are trained in radiation protection procedures prior to starting work. Annually, plant personnel are retrained in radiation protection procedures.

Detailed instructions on radiation safety are contained in the Operations Manual. These instructions set forth rules and procedures of radiation safety that must be followed by all plant personnel.

Radiation areas inside the radiologically controlled area are posted in accordance with the requirements of 10CFR20. All entries into the radiologically controlled area require radiation work permits.

In addition to the Radiation Monitoring System described in Section 7.5.1, routine radiation and contamination surveys are conducted. In addition, specific surveys are made on request to evaluate and determine safe working conditions for personnel of specific jobs. Written procedures for conducting radiation and contamination surveys are provided.

All personnel, other than escorted visitors, who occasionally enter the radiologically controlled area, are indoctrinated in radiological safety. All personnel entering the radiologically controlled area that have not been indoctrinated in radiological safety must obtain permission and be assigned an escort to enter the area.

Personnel exposure is determined using thermoluminescent devices or film badges and dosimeters. Exposure limits are established to conform with the requirements of 10CFR20. Protective clothing and equipment are available to minimize personnel exposure to contamination. Personnel decontamination procedures are provided.

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TABLE 12.2-1 CLASSIFICATION OF STRUCTURES, SYSTEMS AND COMPONENTS

NOTE: This table provides overall (simplified) design classification of structures and components. In general, for systems, the class listed here is the highest design classification assigned to any portion of the system. To determine detailed design classifications and boundaries separating different design classes within the overall classification scheme listed here, refer to the controlled drawings. This may include piping isometric drawings, structural/architectural drawings, and piping and instrumentation drawings. The site equipment database of record includes additional classification details on a case-by-case basis.

Item	Class
Classification of Buildings and Structures	
Containment Vessel (including all Penetrations, Air Locks, Isolation Valves, Vacuum Relief Devices and Containment Vessel Internal Structures Performing Class I Function)	I
Shield Building (Including Vent and all Penetrations)	I
Spent Fuel Pool Structure and Enclosure (Including Fuel Transfer Tube and Valves)	I
Control Room and Relay Room	I
Plant Screenhouse (Concrete Areas Housing Cooling Water Facilities, Equipment and Piping)	I
Plant Screenhouse (Steel Frame Areas Housing Non-Safety Related Equipment)	I*
Auxiliary Building (Areas Housing Auxiliary Building Special Ventilation System, Radwaste Storage and Engineered Safeguards)	I
Auxiliary Building Support System for Aux. Bldg. Crane (See note 1)	I*
Auxiliary Building (Except Class I or I* - See note 2)	III*
D5/D6 Diesel Generator Building (See note 3)	I

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<u>NOTE 1</u> - For definition of Class I* refer to Section 12.2.1.

<u>NOTE 2</u> - For definition of Class III* refer to Section 12.2.1.

<u>NOTE 3</u> - D5/D6 Diesel Generator Building is designed as Seismic Category I, in accordance with Regulatory Guide 1.29, Rev. 3, 9/78, Seismic Design Classification (Reference 43)

TABLE 12.2-1 CLASSIFICATION OF STRUCTURES, SYSTEMS AND COMPONENTS
(Page 2 of 9)

Item	Class
Classification of Buildings and Structures	
Turbine Building (Areas Housing Cooling Water Pipes, Batteries, Safeguards 4160 Volt and 480 Volt Switchgear, Air Compressor and Auxiliary Feedwater Pumps)	I
Turbine Building Support System for Turbine Bldg. Crane	I*
Turbine Building (Except Class I or Class I*)	III*
Administrative Building	III*
Old Service Building (Below Elev. 715′)	I
Old Service Building (Elev. 715′ to 735′)	I*
Old Service Building (Above Elev. 735′)	III*
Cooling Tower Pump Structure	III
Stacks and Miscellaneous Structures	III
Radwaste Building	I*
Resin Disposal Building	III
Lower Level Radwaste Storage Enclosure	III
Classification of Systems and Components	
<u>Reactor Plant Equipment</u>	
<u>Reactor</u>	
Vessel and its Supports	I
Vessel Internals	I
Fuel Assemblies	I
RCC Assemblies and Drive Mechanisms	I
In-core Instrumentation Structures	I

TABLE 12.2-1 CLASSIFICATION OF STRUCTURES, SYSTEMS AND COMPONENTS

(Page 3 of 9)

Item	Class
Classification of Systems and Components (Continued)	
Reactor Coolant System	
Piping and Valves Containing Full System Pressure (Including Safety and Relief Valves)	I
Steam Generators	I
Pressurizer (Excluding Pressurizer Relief Tank and Piping Downstream of Pressurizer Relief Valves)	I
Reactor Coolant Pumps	I
Supporting and Positioning Members	I
Sampling System (Up to Second Isolation Valve)	I
Pressurizer Relief Tank	II
Piping (Downstream of Pressurizer Relief Valves)	II
Emergency Core Cooling System	
Safety Injection System (Including Accumulator Tanks, Pumps, Residual Heat Removal Pumps (ACS), Refueling Water Storage Tank, RHR Heat Exchangers (ACS), and Primary Connecting Piping and Valving)	I
Residual Heat Removal System	I
Containment Spray System (Including Spray Pumps, Spray Headers, and Primary Connecting Piping and Valving)	I
Containment Fan Coil Units (Including Fans, Coils and Housing)	I
Component Cooling System	I
Reactor Protection System	I
Reactor Control System	II
Radiation Monitoring System (to the extent that it must function in support of Class I equipment)	I

TABLE 12.2-1 CLASSIFICATION OF STRUCTURES, SYSTEMS AND COMPONENTS

Item	Class
Classification of Systems and Components (Continued)	
Emergency Power Supply System	
Diesel Generators (See Note 1)	I
Fuel Oil Storage Tank (See Note 1)	I
Diesel Generator Cooling System (See Note 1)	I
Safeguard Buses (See Note 1)	I
Emergency Load Distribution System (See Note 1)	I
DC Power Supply, Batteries, Cables	I
Instrumentation	
Instrumentation and Control (Portions required for safe shutdown)	I
Instrumentation and Control (Other than those Class I or Class II components)	III
Instrument Air System – Portions Required for Safe Shutdown	I
Instrument Air System (Except Portions Required for Safe Shutdown)	II
Computer	III
Turbine Plant System Instrumentation (Except Portions of Reactor Protection System which is Class I or Reactor Control System which is Class II)	III

NOTE 1 - The indicated Design Class I is applicable to D1/D2 Diesel Generators and associated safety related components and systems. D5/D6 Diesel Generators and associated safety related components and systems are designed as Seismic Category I, in accordance with Regulatory Guide 1.29, Rev. 3, 9/78, Seismic Design Classification (Reference 43).

TABLE 12.2-1 CLASSIFICATION OF STRUCTURES, SYSTEMS AND COMPONENTS
(Page 5 of 9)

Item	Class
Classification of Systems and Components (Continued)	
Nuclear Fuel Handling and Storage	
New Fuel Storage	I
Spent Fuel Storage	I
Spent Fuel Pool Liner	I*
Spent Fuel Pool Cooling System (Piping and Valving Whose Failure Could Result in Significant Release of Pool Water)	I
Spent Fuel Pool Cooling System (portions not Class I)	III
Special Ventilation Systems	
Shield Building Vent System	I
Auxiliary Building Special Ventilation System	I
Containment Vessel Purge System	III
Containment Vessel Air Clean-Up System	III
Control Room Air Conditioning and Ventilation System	I
Spent Fuel Pool Special Ventilation System	I
Spent Fuel Pool Normal Ventilation System (Except up to Isolation Valves Which is Class I)	III
Reactor Accessories Cooling and Ventilation System	
Reactor Control Rod Drive Cooling System	II
Reactor Gap Cooling System	II
Reactor Support Cooling System	II
Chemical Lab and Counting Room Ventilation System	III
Relay Room Air Conditioning and Ventilation System	I
D1/D2 Diesel-Generator Room Ventilation System	I
Screenhouse Ventilation System (For Class I Equipment)	I
D5/D6 Diesel Generator Building Safety Related Ventilation System	see Note 1 below

NOTE 1 - D5/D6 Diesel Generator Building Safety Related Ventilation is designed as Seismic Category I, in accordance with Regulatory Guide 1.29, Rev. 3, 9/78, Seismic Design Classification (Reference 43).

TABLE 12.2-1 CLASSIFICATION OF STRUCTURES, SYSTEMS AND COMPONENTS
(Page 6 of 9)

Item	Class
Classification of Systems and Components (Continued)	
Reactor Chemical and Volume Control System	
All items except those listed below	I
Charging Pumps	I
Batching Tanks	III*
Evaporator Condensate Demineralizers	III*
Condensate Filter	III*
Monitor Tanks	III*
Monitor Tank Pumps	III*
Deborating Demineralizers	III*
Concentrates Holding Tank	III*
Concentrates Holding Tank Transfer Pumps	III
Chemical Mixing Tank	III
Boric Acid Evaporator	III
Waste Disposal System	
Waste Hold Up Tank	III*
Sump Tank	III*
Gas Decay Tank	I
Reactor Coolant Drain Tank and Pumps	II
Waste Gas Compressor Package	I
Waste Evaporator Feed Pump	III*
Sump Tank Pumps	III*
Interconnecting Piping and Valves Between Class I Equipment	I
Waste Evaporator	III*
Waste Evaporator Condensate Tanks	III*
Chemical Drain Tank	III
Laundry and Hot Shower Tanks	III
ADT Evaporator	III*
Waste Concentrates Tanks	III*

TABLE 12.2-1 CLASSIFICATION OF STRUCTURES, SYSTEMS AND COMPONENTS
(Page 7 of 9)

Item	Class
Classification of Systems and Components (Continued)	
Waste Disposal System (Continued)	
Miscellaneous Drains Collection Tank	III*
ADT Collection Tanks	III*
ADT Condensate Receiver Tanks	III*
ADT Monitor Tanks	III*
All Other Items Are	III
Automatic Gas Analyzer H_2 and O_2	III*
Nitrogen Supply Manifold	III
Hydrogen Supply Manifold	III
Miscellaneous Reactor Plant Equipment	
Steam Generator Blowdown System Upstream of Isolation Valves	I
Steam Generator Blowdown System Downstream of Isolation Valves	III
Polar Crane	I*
Manipulator Crane	I*
Fuel Pool Bridge Crane (See note 1)	I
Auxiliary Building Crane (See note 2)	I*
Turbine Building Crane	I*
All Other Cranes	III
Conventional Equipment, Tanks, Piping, Other than Class I and II	III
Reactor Make Up Water Storage Tank	III

NOTE 1 - Upgraded to Class I for structural load carrying elements only. Also upgraded to single-failure-proof criteria (NUREG 0554) (Mod 89Y055)

NOTE 2 - Auxiliary Building Crane Trolley upgraded to single-failure-proof criteria (see Section 12.2.12.1.3)

TABLE 12.2-1 CLASSIFICATION OF STRUCTURES, SYSTEMS AND COMPONENTS

Item	Class
Classification of Systems and Components (Continued)	
Turbine Plant	
Turbine, Generator, Foundation, Exciter, Oil Purification, Turbine Gland Seal System, Reheaters and Moisture Separators, Generator Cooling Water System, Hydrogen and CO_2 Systems	III
Cooling Water System	
Up to Class I System Isolation Valves	I
All that is not Class I (Note 1)	III
Circulating Water System	
Emergency Cooling Water Intake	I
Approach Canal	I
Circulating Water Pumping Equipment	III
Intake Canal	I*
Circulating Water Pump Discharge Piping	III
Condenser Discharge Piping	III
Intake and Discharge Equipment	III
Cooling Towers and Pumping Equipment	III
Condensate and Feedwater System	
Main Condenser	III
Condensate System	III
Condensate Polishing System	III
Main Feedwater System (Excluding Class I Piping and Valves)	III
Air Removal System	III
Auxiliary Feedwater System	I

NOTE 1 - Selected Class III piping may be upgraded to withstand a design basis seismic event as discussed in Section 12.2.1.5.2.4.

TABLE 12.2-1 CLASSIFICATION OF STRUCTURES, SYSTEMS AND COMPONENTS
(Page 9 of 9)

Item	Class
Classification of Systems and Components (Continued)	
Main Steam, Extraction, Heater Drain System	
Main Steam, Safety, Relief and Isolation Valves	I
Main Steam, Auxiliary Steam Piping, Up to Isolation Valves	I
Steam Dump System	III*
Miscellaneous Power Plant Equipment	
Station Air Systems	III
Backup Water Supply for Fire Protection (Serving Class I Equipment)	I
Fire Protection System Including Detection and Alarms (Other Than Class I)	III
Fire Protection System piping associated with the safeguards ventilation exhaust filters and containment penetration	I
Transformers	
Main Transformer	III
Auxiliary Transformer	III*
Start-Up Transformer	II
Redundant Offsite Transformer (Cooling Tower Transformer)	III
Non Safeguards 4160 - 480V Auxiliary Transformer	III
4160 - 480V Safeguards Transformers	I
Transformer Serving Pressurizer Heaters from Safeguards Bus	II

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TABLE 12.2-2 NORTHERN STATES POWER TERMINOLOGY 01514406

(Page 1 of 7)

AEC System Terminology	Northern States Power Company Prairie Island Systems Terminology	Extent of Examination for Piping, Valves, and Pumps	Exam, Methods (See Appendix C)	Acceptance Standards
1. Reactor Coolant Pressure Boundary	Reactor Coolant System and Portions of Following Systems Connected to Reactor Coolant System Within Primary Containment:	PIPING 100% Pressure-Containing Welds (Longitudinal and Girth) in Piping Above 1-1/2″ Nom. Pipe Size	RT, PT*	Weld Joints RT - ASA B31 Code Case N7
	(a) Chemical & Volume Control System*	PIPE FITTINGS 100% Pressure-Containing Welds in Fittings (Cast or Forged) Above 1-1/2″ Nom. Pipe Size	RT and PT	PT - ASA B31 Code Case N7 01388733 Cast Materials
	(b) High Head Safety Injection System*	100% Metal in Cast Fittings Above 2″ Nom. Size	RT and PT	RT - Type I - Accept. Std. - See Appendix A or ASA B31 Code Case N2, N10
	(c) Low Head Safety Injection System*	VALVES (INCLUDING SAFETY AND RELIEF VALVES)		
	(d) Accumulators - Safety Injection System **	100% Pressure-Containing Welds in Valve Bodies Above 2I Nom. Pipe Sizes	RT and PT	PT - Type I - Accept. Std. See Appendix E or ASA B31 Code Case N2, N10
	(e) Residual Heat Removal System*	100% Metal in Cast Valves Above 2″ Nom. Size	RT and PT	
	(f) Safety & Relief Valve System	100% Metal in Forged Valves Above 3-1/2″ Nom. Size	(RT or UT) and (PT or MT)	Forged Materials:
	(g) Reactor Coolant Vent & Drain System**	100% Cast Metal Discs in Containment Isolation Valves and Safety Valves Above 2″ Nom. Size	RT and PT	RT - Type I - Accept. Std. - See Appendix A or ASA B31 Code Case N2

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TABLE 12.2-2 NORTHERN STATES POWER TERMINOLOGY

AEC System Terminology	Northern States Power Company Prairie Island Systems Terminology	Extent of Examination for Piping, Valves, and Pumps	Exam, Methods (See Appendix C)	Acceptance Standards
	(h) Reactor Coolant Temperature Control lines	PUMPS 100% Pressure-Containing Welds in Pump Casings	RT and PT	UT - See Appendix D.
* For definition refer to Appendix F	* The system boundary extends to and includes the outermost containment isolation valve.	100% Pressure-Containing Casing of Case Pumps	RT and PT	PT or MT - Type I - Accept. Std. - See Appendix E or
		ASA B31 - Code		
	** The system boundary extends to and includes the second valve normally closed during normal reactor operation.	NOTE: The 100% figure refers to the extent of piping and components accessible for examination without dismantling of systems or removal of structural and concrete obstacles.	* Where size or configuration of weld does not permit effective radiography, PT may be used.	Case N2

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TABLE 12.2-2 NORTHERN STATES POWER TERMINOLOGY

Appendix A (For Table 12.2-2)

Radiographic Acceptance Standards for Ferritic and Austenitic Materials

These RT acceptance standards shall be used to determine the acceptability of radiographically determined indications in pipe fittings, valve bodies and pump casings of ferritic and austenitic materials.

Type I Radiographic Acceptance Standard. Cast materials shall be completely radiographed and shall meet the Class 2 reference standard of ASTM E 71-52 (or optionally meet Severity Level 2 of ASTM E 71-64, ASTM E 186-67, and ASTM E 280-68 except that type D, E, F, and G categories shall be unacceptable). If the geometry or inaccessibility is such that complete radiography is impractical, any lesser coverage shall be noted and indicated on appropriate drawings or radiographic shooting sketches showing the areas covered and including an explanation for the lesser coverage.

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TABLE 12.2-2 NORTHERN STATES POWER TERMINOLOGY

Appendix C (For Table 12.2-2)

Examination Methods

RT - Radiographic examination shall be performed in accordance with the procedures specified in ASTM - Specification E-94 - “Recommended Practice for Radiographic Testing.”

UT - Ultrasonic examination shall be performed in accordance with the procedures specified in ASTM Specifications E-114 - “Ultrasonic Testing,” or ASTM Specification A-388 - “Ultrasonic Testing.”

PT - Liquid penetrant examination shall be performed in accordance with the procedures specified in ASTM Specification E-165 - “Methods for Liquid Penetrant Inspection.”

MT - Magnetic particle examination shall be performed in accordance with the procedures specified in:

ASTM-E-109 - “Dry Powder Magnetic Particle Inspection.”
ASTM-E-138 - “Wet Magnetic Particle Inspection.”

Alternate examination techniques may be substituted for each of the above methods provided the results yield demonstrated equivalence or superiority.

TABLE 12.2-2 NORTHERN STATES POWER TERMINOLOGY

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Appendix D (For Table 12.2-2)

Ultrasonic Testing Acceptance Standards for Forge Materials

Forgings shall be unacceptable if straight beam examinations results show one or more reflectors which produce indications accompanied by a complete loss of back reflection not associated or attributed to the geometric configuration; or if angle beam examination results show one or more reflectors which produce indications exceeding in amplitude the indication from the calibrated notch. Complete loss of back reflection is assumed when the back reflection falls below 5 percent of the full screen height.

TABLE 12.2-2 NORTHERN STATES POWER TERMINOLOGY

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Appendix E (For Table 12.2-2)

Liquid Penetrant (PT) and Magnetic Particle (MT) Inspection Acceptance Standards for Pipe Fittings, Valve Bodies, and Pump Casings

These PT and MT inspection acceptance standards shall be used to determine the acceptability of surface and near surface nondestructive testing indications in cast pipe fittings, valve bodies, and pump casings. These standards shall also be applicable to forged valve bodies.

Type I Acceptance Standard. Casting surfaces shall be free of tears or crack-like linear indications. Other indications such as shrinkage cavities, cold shuts, unfused chaplets, or internal chills are acceptable provided that not more than one indication, 3/16 inch or less in length is detected in any 6 square inch surface. Indications 1/16 inch and less shall not be counted.

Nonlinear or rounded indications are acceptable provided that not more than the number of indications listed below, 3/16 inch or less in size (major dimension of indication) is detected in any 6 square inch surface. Indications 1/16 inch and less shall not be counted.

a. Four nonlinear or rounded indications in line, separated by 1/16 inch or less, edge-to-edge.

b. Ten rounded indications within 6 square inch area with the major dimension of this area not to exceed 6 inches, and the area taken in the most unfavorable location relative to the indications being evaluated.

TABLE 12.2-2 NORTHERN STATES POWER TERMINOLOGY

Appendix F (For Table 12.2-2)

Definition of “Reactor Coolant Pressure Boundary”

“Reactor coolant pressure boundary” means all those pressure-containing components, such as pressure vessels, piping, pumps and valves within the following systems or portions of systems of a water-cooled nuclear power plant:

i. The reactor coolant system.

ii. Portions of associated auxiliary systems connected to the reactor coolant system. For piping of these systems which penetrates primary reactor containment, the boundary extends to and includes the first containment isolation valve outside the containment capable of external actuation. For piping of these systems which contains two valves, both of which are normally closed during normal reactor operation, the boundary extends to and includes the second of these two valves (the second of which must be capable of external actuation), whether or not the system piping penetrates primary reactor containment.

iii. Portions of the emergency core cooling system connected to the reactor coolant system. For piping of this system which penetrates primary reactor containment, the boundary extends to and includes the first containment isolation valve outside containment capable of external actuation. For piping of this system which does not penetrate primary reactor containment, the boundary extends to and includes the second of two valves normally closed during normal reactor operation.

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TABLE 12.2-3 GINNA TERMINOLOGY

AEC SYSTEM TERMINOLOGY	WESTINGHOUSE SYSTEMS TERMINOLOGY	EXTENT OF EXAMINATION FOR PIPING, VALVES, AND PUMPS	EXAM. METHODS (SEE APPENDIX C)	ACCEPTANCE STANDARDS
Reactor Coolant Associated Auxiliary Systems	Portions of Systems Located Outside of Primary Containment:	<u>PIPING</u> Longitudinal and Girth Welds		<u>WELD JOINTS</u>
	(a) Chemical & Volume Control System*	System (a) - In Piping Above 1-1/2″ Nom. Size	RT*	RT - ASME Boiler and Pressure Vessel Code Section III N 624
		System (b) - In Piping Above 2″ Nom. Size	RT*	
		System (c) - In Piping Above 2″ Nom. Size	PT or MT	
	(b) Residual Heat Removal System**	<u>PIPE FITTINGS</u> Cast Fittings		
		System (a) - Above 2″ Nom. Pipe Size	RT* and (PT or MT)	PT MT-Appendix B
		System (b) - Above 2-1/2″ Nom. Pipe Size	RT and (PT or MT)	
		System (c) - Above 2-1/2″ Nom. Pipe Size	PT or MT	

* Includes those components and piping required for boron injection function.

** Includes those portions of system not within the system boundary of the low head safety injection system.

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TABLE 12.2-3 GINNA TERMINOLOGY

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AEC SYSTEM TERMINOLOGY	WESTINGHOUSE SYSTEMS TERMINOLOGY	EXTENT OF EXAMINATION FOR PIPING, VALVES, AND PUMPS	EXAM. METHODS (SEE APPENDIX C)	ACCEPTANCE STANDARDS
	(c) Service Water System***	Welds in Fittings		CAST AND FORGED MATERIALS
		System (a) - Above 1-1/2″ Nom. Pipe Size	RT*	RT - Type II - Accept. Std. - Appendix A
		System (b) - Above 2″ Nom. Pipe Size	RT*	
		System (c) - Above 2″ Nom. Pipe Size	PT or MT	
				PT, MT - Type II Accept. Std. - Appendix E
		VALVES		
	*** Includes only those components and piping required for the heat exchangers to provide cooling function of the component cooling system.	Cast Valve Bodies		UT - Type II - Accept. Std. - Appendix D
		System (a) - Above 2″ Nom. Size	RT and (PT or MT)	
		System (b) - Above 2-1/2″ Nom. Size	RT and (PT or MT)	
		System (c) - Above 2-1/2″ Nom. Size	PT or MT	
			* Where size or configuration of weld did not permit effective radiography, PT or MT was used.	NOTE: PT may be used for system components of ferritic or austenitic materials. MT may be used only for system components of ferritic materials.

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TABLE 12.2-3 GINNA TERMINOLOGY

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AEC SYSTEM TERMINOLOGY	WESTINGHOUSE SYSTEMS TERMINOLOGY	EXTENT OF EXAMINATION FOR PIPING, VALVES, AND PUMPS	EXAM. METHODS (SEE APPENDIX C)	ACCEPTANCE STANDARDS
		Forged Valve Bodies		
		System (a) - Above 3-1/2″ Nom. Size	(RT or UT) & (PT or MT)	
		System (b) - Above 4″ Nom. Size	(RT or UT) & (PT or MT)	
		System (c) - Above 4″ Nom. Size	PT or MT	
		Pressure-Containing Welds in Above Valves		
		System (a) and (b)	RT*	
		System (c)	PT or MT	
		<u>PUMPS</u>		
		Cast Pump Casing		
		System (s) and (b)	RT and (PT or MT)	
		System (c)	PT or MT	
		Pressure-Containing Welds in Casing		
		System (a) and (b)	RT*	
		System (c)	PT or MT	
			* Where size or configuration of weld did not permit effective radiography, PT or MT was used.	<u>NOTE:</u> PT may be used for system components of ferritic or austenitic materials. MT may be used only for system components of ferritic materials.

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TABLE 12.2-3 GINNA TERMINOLOGY

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AEC SYSTEM TERMINOLOGY	WESTINGHOUSE SYSTEMS TERMINOLOGY	EXTENT OF EXAMINATION FOR PIPING, VALVES, AND PUMPS	EXAM. METHODS (SEE APPENDIX C)	ACCEPTANCE STANDARDS
Emergency Core Cooling Systems.	Those Portions of Following Systems Located Outside of Primary Containment	PIPING		WELD JOINTS
		System (a) Longitudinal and Girth Welds in Piping Above 1-1/2″ Nom. Size	RT*	RT - ASME Boiler and Pressure Vessel Code Section III N 624
	(a) High Head Safety Injection System*	Systems (b), (c), (d), (e) Longitudinal and Girth Welds in Piping Above 2″ Nom. Size	RT*	
Containment Heat Removal Systems.	(b) Low Head Safety Injection System*	PIPE FITTINGS		
		System (a) Welds in Fittings Above 1-1/2″ Nom. Pipe Size	RT*	PT, MT - Appendix B
	* Includes those components and piping required for injection of water into the reactor coolant pressure boundary.	Cast Fittings Above 2″ Nom. Pipe Size	RT and (PT or MT)	
			* Where size or Configuration of weld did not permit effective radiography, PT or MT was used	

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TABLE 12.2-3 GINNA TERMINOLOGY

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AEC SYSTEM TERMINOLOGY		WESTINGHOUSE SYSTEMS TERMINOLOGY	EXTENT OF EXAMINATION FOR PIPING, VALVES, AND PUMPS	EXAM. METHODS (SEE APPENDIX C)	ACCEPTANCE STANDARDS
Containment Atmosphere Cleanup Systems.	(c)	Accumulator Tank System*	PIPE FITTINGS (Continued)		CAST MATERIALS
			Systems (b), (c), (d), (e)		RT - Type II - Accept. Std. - Appendix A
			Welds in Fittings Above 2″ Nom. Pipe Size	RT*	
Associated Cooling Water Systems	(d)	Containment Spray Systems**	Cast Fittings Above 2-1/2″ Nom. Pipe Size	RT and (PT or MT)	PT, MT - Type II Accept. Std. - Appendix A and Appendix E
	(e)	Component Cooling System***			FORGED MATERIALS
					RT - Type II - Accept. Std. Appendix A
					UT - Appendix B
	*	Includes those components and piping required for injection of water into the reactor coolant boundary.		*Where size or configuration of weld did not permit effective radiography, PT or MT was used.	PT, MT - Type II - Accept. Std. - Appendix E
	**	Includes those components and piping required for recirculation of water from containment sump through heat removal exchangers and to containment spray.			
	***	Includes those components and piping required to provide cooling function for containment spray system and low head safety injection system.			NOTE: PT may be used for system components of ferritic or austenitic materials. MT may be used only for system components of ferritic materials.

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TABLE 12.2-3 GINNA TERMINOLOGY

AEC SYSTEM TERMINOLOGY	WESTINGHOUSE SYSTEMS TERMINOLOGY	EXTENT OF EXAMINATION FOR PIPING, VALVES, AND PUMPS	EXAM. METHODS (SEE APPENDIX C)	ACCEPTANCE STANDARDS
		VALVES		
		System (a)		
		Cast Valve Bodies Above 2″ Nom. Size	RT and (PT or MT)	
		Forged Valve Bodies Above 3-1/2″ Nom. Size	(RT or UT) and (PT or MT)	
		Pressure-Containing Welds in Above Valves	RT*	
		Systems (b), (c), (d), (e)		
		Cast Valve Bodies Above 2-1/2″ Nom. Size	RT and (PT or MT)	
		Forged Valve Bodies Above 4″ Nom. Size	(RT or UT) and (PT or MT)	
		Pressure-Containing Welds in Above Valves	RT*	
		PUMPS		
		Systems (a), (b), and (d), (e)		
		Cast Pump Casing	RT and (PT or MT)	
		Pressure-Containing Welds in Pump Casing	RT*	

* Where size or configuration of weld did not permit effective radiography, PT or MT was used.

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TABLE 12.2-3 GINNA TERMINOLOGY

AEC SYSTEM TERMINOLOGY	WESTINGHOUSE SYSTEMS TERMINOLOGY	EXTENT OF EXAMINATION FOR PIPING, VALVES, AND PUMPS	EXAM. METHODS (SEE APPENDIX C)	ACCEPTANCE STANDARDS
Main Steam and Feedwater System.	Those Portions of Main Steam and Feedwater System:	PIPING		WELD JOINTS
	(a) Main Steam System	Longitudinal and Girth Welds in Piping Above 2″ Nom. Size	RT*	RT - ASME Boiler and Pressure Vessel Code Section III
	Extending from steam generator to and including the outermost containment isolation valve in Steam Piping Outside Reactor Containment and any Branch Lines Within This Boundary Extending to and including the 1st Externally-actuated Valve or Safety Valve	PIPE FITTINGS		
		Cast Fittings Above 2-1/2″ Nom. Pipe Size	RT and (PT or MT)	N624
		Weld in Fittings Above 2″ Nom. Pipe Size	RT*	PT, MT - Appendix B
		VALVES		CAST MATERIAL
		Cast Valve Bodies Above 2-1/2″	RT and (PT or MT)	RT - Type II - Accept. Std. - Appendix A

TABLE 12.2-3 GINNA TERMINOLOGY

AEC SYSTEM TERMINOLOGY	WESTINGHOUSE SYSTEMS TERMINOLOGY	EXTENT OF EXAMINATION FOR PIPING, VALVES, AND PUMPS	EXAM. METHODS (SEE APPENDIX C)	ACCEPTANCE STANDARDS
	(b) Feedwater System	Nom. Size		
	Extending from steam generator to and including the Outermost Containment Isolation Valves in	Forged Valve Bodies Above 4″ Nom. Size	(RT or UT) and (PT or MT)	PT, MT - Type II - Accept. Std. - Appendix E
	Feedwater Piping Outside Reactor Containment Including Branch Lines within this Boundary Extending to and Including the 1st Externally-actuated Valve.	Pressure-Containing Welds in Above Valves	RT*	
			* Where size or configuration of weld did not permit effective radiography, PT or MT was used NOTE: PT may be used for system components of ferritic or austenitic materials, MT may be used for system components of ferritic materials.	
	(c) Auxiliary Feedwater System			<u>FORGED MATERIALS</u>
	Those portions of the System Extending from the Pumps to the Connection with the Feedwater System.			UT - Type II - Accept. Std. - Appendix D.
				PT, MT - Type II - Accept. Std. - Appendix E.
				RT - Type II - Accept. Std. Appendix A

TABLE 12.2-3 GINNA TERMINOLOGY

Appendix A

Radiographic (RT) Acceptance Standards for Ferritic and Austenitic Materials

These RT acceptance standards shall be used to determine the acceptability of radiographically detected indications in pipe fittings, valve bodies and pump casings of ferritic and austenitic materials.

Type I Radiographic Acceptance Standard. Materials shall be completely radiographed and shall meet the Class 2 reference standard of ASTM E 71-52 (or optionally meet Severity Level 2 of ASTM E 71-64, ASTM E 186-67, and ASTM E 280-68 except that type D, E, F, and G categories shall be unacceptable). If the geometry or inaccessibility is such that complete radiography is impractical, any lesser coverage shall be noted and indicated on appropriate drawings or radiographic shooting sketches showing the areas covered and including an explanation for the lesser coverage.

Type II Radiographic Acceptance Standard. The material examined shall meet the Class 2 reference standard of ASTM E 71-52 except that A3 and B3 type discontinuities may be considered as “borderline” for material thicknesses over 1/2 inch.

Optionally, the material may be examined to meet Severity Level 2 of ASTM E 71-64, ASTM E 186-67, ASTM E 280-67, ASTM E 280-68 as applicable, except that type D, E, F, and G categories shall be unacceptable. However, with these specifications, Severity Levels A3 and B3 are acceptable for material thicknesses over 1/2 inch.

The minimum area of the fitting, valve, or pump to be examined shall be not less than 75 percent of the pressure-containing external surfaces of the component. Areas included in this coverage shall be those designated by the designer to be potentially susceptible to unacceptable radiographic discontinuities. If the geometry and accessibility are such that 75 percent coverage is impractical, any lesser coverage shall be noted and indicated on appropriate drawings or radiographic shooting sketches showing the areas covering and including an explanation of the lower coverage.

TABLE 12.2-3 GINNA TERMINOLOGY

Appendix B

Liquid Penetrant (PT) and Magnetic Particle (MT) Examination Acceptance Standard for Welded Joints

These PT and MT examination Acceptance Standards shall be used to determine the acceptability of surface and near surface indications in welded joints of piping, examined in accordance with “Liquid Penetrant Inspection” ASTM-E-165, or in accordance with “Dry Powder Magnetic Particle Inspection” ASTM-E-109, and “Wet Magnetic Particle Inspection” ASTM-E-138.

Liquid penetrant, or magnetic particle indications at the weld joint and adjacent metal shall be considered unacceptable if any one of the following conditions exist:

a. Any crack-like indication or incomplete fusion.

b. Aligned penetrant indications in which the average of the center-to-center distances between any one indication and the two adjacent indications is less than 3/16 of an inch.

TABLE 12.2-3 GINNA TERMINOLOGY

Appendix C

Examination Methods

RT - Radiographic examinations shall be performed in accordance with the procedures specified in ASTM - Specification E-94 - “Recommended Practice for Radiographic Testing.”

UT - Ultrasonic examination shall be performed in accordance with the procedures specified in ASTM Specifications E-114 - “Ultrasonic Testing,” or ASTM Specification A-388 - “Ultrasonic Testing.”

PT - Liquid penetrant examination shall be performed in accordance with the procedures specified in ASTM Specification E-165 - “Methods for Liquid Penetrant Inspection.”

MT - Magnetic particle examination shall be performed in accordance with the procedures specified in:

ASTM-E-109 - “Dry Powder Magnetic Particle Inspection.”

ASTM-E-138 - “Wet Magnetic Particle Inspection.”

Alternate examination techniques may be substituted for each of the above methods provided the results yield demonstrated equivalence or superiority.

TABLE 12.2-3 GINNA TERMINOLOGY
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Appendix D

Ultrasonic Testing (UT) Acceptance Standards for Forged Materials

These UT acceptance standards shall be used to determine the acceptability of ultrasonically detected indications in forged materials examined in accordance with the "Recommended Practice for Ultrasonic Testing and Inspection of Heavy Steel Forgings" ASTM-A-388.

Type I - Ultrasonic Acceptance Standard

Forgings shall be completely examined and shall be unacceptable if straight beam examinations results show one or more reflectors which produce indications accompanied by a complete loss of back reflection not associated or attributed to the geometric configuration; or if angle beam examination results show one or more reflectors which produce indications exceeding in amplitude the indication from the calibrated notch. Complete loss of back reflection is assumed when the back reflection falls below 5 percent of the full screen height.

Type II - Ultrasonic Acceptance Standard

The minimum area to be examined shall be not less than 75 percent of the pressure-containing surfaces of the component. Areas included in the coverage shall be those designated by the designer to be potentially susceptible to unacceptable ultrasonic discontinuities. If the geometry and accessibility are such that 75 percent coverage is impractical, any lesser coverage shall be noted on appropriate drawings, including an explanation of the lower coverage.

Forgings shall be unacceptable if straight beam examinations results show one or more reflectors which produce indications accompanied by a complete loss of back reflection not associated or attributed to the geometric configuration; or if angle beam examination results show one or more reflectors which produce indications exceeding in amplitude the indication from the calibrated notch. Complete loss of back reflection is assumed when the back reflection falls below 5 percent of the full screen height.

TABLE 12.2-3 GINNA TERMINOLOGY

Appendix E

Liquid Penetrant (PT) and Magnetic Particle (MT) Inspection Acceptance Standards for Pipe Fittings, Valve Bodies, and Pump Casings

These PT and MT inspection acceptance standards shall be used to determine the acceptability of surface and near surface nondestructive testing indications in cast pipe fittings, valve bodies, and pump casings. These standards shall also be applicable to forged valve bodies.

Type I Acceptance Standard. Casting surfaces shall be free of tears or crack-like linear indications. Other indications such as shrinkage cavities, cold shuts, unfused chaplets, or internal chills are acceptable provided that not more than one indication, 3/16 inch or less in length is detected in any 6 square inch surface. Indications 1/16 inch and less shall not be counted.

Nonlinear or rounded indications are acceptable provided that not more than the number of indications listed below, 3/16 inch or less in size (major dimension of indication) is detected in any 6 square inch surface. Indications 1/16 inch and less shall not be counted.

a. Four nonlinear or rounded indications in line, separated by 1/16 inch or less, edge-to-edge.

b. Ten rounded indications within 6 square inch area with the major dimension of this area not to exceed 6 inches, and the area taken in the most unfavorable location relative to the indications being evaluated.

TABLE 12.2-3 GINNA TERMINOLOGY
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Appendix E (Continued)

Liquid Penetrant (PT) and Magnetic Particle (MT) Inspection Acceptance Standards for Pipe Fittings, Valve Bodies, and Pump Casings (Continued)

Type II Acceptance Standard. Casting surfaces shall be free of tears or crack-like linear indications. Other indications such as shrinkage cavities, cold shuts, unfused chaplets, or internal chills are acceptable provided that not more than one indication 1/4 inch or less in length is detected in any 6 square inch surface. Random distributed indications 1/16 inch and less shall not be counted.

Nonlinear or rounded indications are acceptable provided that not more than the number of indications listed below, 1/4 inch or less in size (major dimension of the indication) is detected in any 6 square inch surface. Indications 1/16 inch and less shall not be counted.

a. Six nonlinear or rounded indications in line, separated by 1/16 inch or less, edge-to-edge.

b. Twelve rounded indications, within 6 square inch area, with the major dimension of this area not to exceed 6 inches and the area taken in the most unfavorable location relative to the indications being evaluated.

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TABLE 12.2-4 LOAD COMBINATIONS FOR STRUCTURES

Conditions of Loading	Class of Structures				
	Class I	Class I*	Class II	Class III*	Class III
Normal Operating	Dead + Live + Wind or Snow	Dead + Live + Wind or Snow	Dead + Live + Wind or Snow	Dead + Live + Wind or Snow	Dead + Live + Wind or Snow
Operational Basis Earthquake (OBE)	Dead + Live +DBA + Greater of OBE + Snow or Wind	NA	NA	NA	NA
Design Basis Earthquake (DBE)	Dead + Live + DBA + DBE + Snow	Dead + Live + Snow + DBE	NA	NA	NA
Tornado	Dead + Live + 300 mph design tornado + tornado missile if any	NA	NA	NA	NA
Other	In addition to the above, jet forces, pipe rupture loads, flood, etc., whichever and wherever applicable.	NA	Dead + Live + Uniform Building Code Zone I Loads (see Section 12.2.1.3.5)	Dead + Live + Uniform Building Code Zone I Loads (See Section 12.2.1.3.5)	NA

Note: NA = Not Applicable

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TABLE 12.2-5 APPLICABLE CODE STRESSES

	Loading Condition	Reinforced Concrete	Structural Steel
1.	Normal operating condition	ACI 318-63 Allowable values	AISC Allowable values
2.	Operational Basis Earthquake Condition	ACI 318-63 Allowable values	AISC Allowable values
3.	Design Basis Earthquake Condition	1-1/2 times ACI 318-63 Allowable values	1-1/2 times AISC Allowable values
4.	Tornado Condition	$f_c = 0.85\ f'_c$ $f_s = 0.90$ Y.S.	$f_s = 0.90$ Y.S.
5.	Pipe Rupture (On Walls and Floors) Dead + Live + Restraint Reactions + Jet + DBE + Pressure (On Ceilings) Dead + Restraint Reactions + Jet + DBE + Pressure (All Accessible Elements) Dead + Live + Restraint Reactions + Whip + DBE + Pressure	$f_c = 0.85\ f'_c$ $f_s = 0.90$ Y.S.	$f_s = 0.90$ Y.S.

Where:
f'_c = Minimum 28 day compressive strength of concrete
f_c = Compressive stress in concrete
f_s = Tensile stress in steel
Y.S. = Specified minimum yield strength or yield point of steel

Notes:

1. Stresses are based on Working Stress Design methods. For Ultimate Strength Design requirements, refer to USAR Section 12.2.15.

2. For additional detail on Loading Conditions 1 through 4, see Table 12.2-4.

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TABLE 12.2-6 ALLOWABLE STRESSES – REINFORCED CONCRETE

Class I Structures

			Reinforced Steel Stresses			
			A 615 Grade 40		A 615 Grade 60	
Loading Condition	**Criteria (1)**	**Concrete Stresses f_c**	**Allowable Working Stress - psi**	**Percent of Min Spec Yield (2)**	**Allowable Working Stress - psi**	**Percent Min Spec Yield (2)**
1. Normal operating condition	ACI 318-63 allowable values	$0.45 f'_c$	20,000	50	24,000	40
2. Operation Basis Earthquake	ACI 318-63 allowable values	$0.45 f'_c$	20,000	50	24,000	40
3. Design Basis Earthquake	1-1/2 times ACI 318-63 allowable values	$0.675 f'_c$	30,000	75	36,000	60

NOTES:

(1) Stress criteria are based on Working Stress Design Methods. Ultimate Strength Design requirements are described in USAR Section 12.2.15.

(2) Minimum specified yield points of steel reinforcement are as follows:

A 615 Grade 40 40,000 psi

A 615 Grade 60 60,000 psi

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TABLE 12.2-7 ALLOWABLE STRESSES: CLASS I*, II, III* & III STRUCTURES

Class	Loading Condition	Criteria
I*	Condition #3 in Tables 12.2-5 & 12.2-6	1½ times ACI 318-63 & AISC Allowable Stresses
II	Dead load plus Live loads, plus greater of Wind, Zone I Earthquake, or Snow	ACI 318-63 & AISC Allowable Stresses with no increase in stresses for earthquake condition
III*	Same as for Class II above	ACI 318-63 & AISC Allowable Stresses with no increase in stresses for earthquake condition
III	Condition #1 in Table 12.2-5	ACI 318-63 & AISC Allowable Stresses

Note: Stress criteria are based on Working Stress Design methods. Ultimate Strength Design requirements are described in USAR Section 12.2.15.

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TABLE 12.2-8 DAMPING FACTORS

Item	% of Critical Damping*
Reactor Building Containment Vessel	1.0
Reactor Building Shield Structure	2.0
Reactor Building Internal Concrete Construction	5.0
Steel Frame Structures	2.0
Reinforced Concrete Construction	2.0
Piping Systems	0.5
Mechanical Equipment	2.0**
Foundation Soils	5.0

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*The maximum percent of critical damping factors given is applied to both the operational basis earthquake (OBE) and the design basis earthquake (DBE).

**The value of 2% critical damping for mechanical equipment is based on laboratory and field results of a variety of equipment ranging from small piping to instrumentation, control and electrical equipment and to primary loop components. Damping values in excess of 2% of critical were obtained from the above tests.

TABLE 12.2-9 TORNADO GENERATED MISSILES

			Vertically Ejected Missiles				**Horizontally Ejected Missiles**		
Missile	**Weight (lbs)**	**Weight to X-Sectional Area Ratio (lbs/in^2)**	**Elevation of Origin Above Target (ft)**	**Total Height of Drop (ft)**	**Vert. Impact Velocity (ft/sec)**	**Vertical Energy (ft-lbs)**	**Missile Impact Height (ft)**	**Horiz. Impact Velocity (ft/sec)**	**Horizontal Energy (ft-lbs)**
Wood Plank Rough Douglas Fir 4″ x 12″ x 12′-0	150	3.1	67	179	108	27,168	Any	440	450,930
Steel Girt A36 Steel W10x11.5 x 20′ long	230	68	67	159	102	37,157	Any	35	4,375
Steel Girt A36 Steel W8x15 x 20′ long	300	68	67	102	81	30,565	Any	35	5,707
Steel Pipe 10.79 lbs/ft 4.5″ O.D. x 20-0′ long	216	68	67	77	70	16,435	Any	66	14,610
Automobile	4,000	1.5	N/A	N/A	N/A	N/A	0 to 25	73	330,990

Notes

1. All missiles are assumed to impact end-on, at 90 degrees to the surface being impacted.
2. Weight to area ratios are based on the following unadjusted cross-sectional areas: wood plank = 48 in^2, 10B11.5 girt = 3.39 in^2, 8B15 girt = 4.43 in^2, steel pipe = 3.17 in^2, and automobile = 2,700 in^2. For area correction factors used in the Modified Petry Formula, refer to USAR Section 12.2.1.4.3.1.4.
3. For vertically ejected missiles, it is assumed the missile is an object on a roof 67′-0 above the eventual target, the tornado passes over and picks up the object, and it rises up inside the tornado's vortex before being dropped onto the target. The total drop height is as shown in the fifth column.

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TABLE 12.2-10 INTERNALLY GENERATED MISSILES

Inside of Containment

Missile	Weight to Cross Sectional Area Ratio (lbs/sq. in.)	Velocity ft/sec	Impact Point
3″ Motor Oper. Isolation Valve	14.1	100	Missile Shield Slab
6″ x 6″ Valve (Safety Relief Valve)	9.3	75	Missile Shield Slab
3″ Air Operator Relief Valve	2.4	50	Missile Shield Slab
Housing Plug	1.8	240	Reactor Vessel Missile Shield
Drive Shaft	52.1	141	Reactor Vessel Missile Shield

NOTE: All internally generated missiles are assumed to impact at 90 degrees.

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TABLE 12.2-11 LOAD COMBINATIONS FOR COMPONENTS

Condition of Loading	Class of Components Class I	Class I*	Class II & III*	Class III
1. Normal	Dead + Live + Environmental Loads (snow + wind) if applicable	Dead + Live + Environmental Loads (snow + wind) if applicable	Dead + Live + Environmental Loads (snow + wind) if applicable	Dead + Live + Environmental Loads (snow + wind) if applicable
2. Normal and Operational Basis Earthquake (OBE)	Dead + Live + Greater of OBE or Wind***	NA	NA	NA
3. Normal and Design Basis Earthquake (DBE)	Dead + Live + DBE	Dead + Live + DBE	NA	NA
4. Normal and Pipe Rupture	Dead + Live + DBA (or other pipe rupture loads if greater)	NA	NA	NA
5. Normal and Design Basis Earthquake and Pipe Rupture	Dead + Live + DBE + DBA (or other pipe rupture loads if greater)**/***	NA	NA	NA
6. Other	NA	NA	Dead + Live + Uniform Building Code Zone 1 Earthquake Loads	NA

NOTES:
1. NA = Not Applicable
2. Load Combinations for the Pressurizer Safety and Relief Discharge piping also include hydraulic forces associated with discharge from the Safety or Relief valve(s). References 67 and 68 describe the load combinations, including the valve discharge load cases, considered in the stress analyses for the Pressurizer Safety and Relief Discharge piping.

** For components designed to ASME Section III, Code Class 1, 2, or 3 requirements, the DBE and LOCA loads may be combined using the Square-Root-Sum-of-the-Squares (SRSS) methodology per Reference 66 provided a linear elastic stress analysis is performed.

*** GL 08-01 Waterhammer loads are combined with DBE and OBE loads using SRSS methodology due to the dynamic nature of each load condition.

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TABLE 12.2-12 LOADING CONDITIONS AND STRESS LIMITS: PRESSURE VESSELS

Loading Condition		Stress Intensity Limits	Note*
1. Normal Condition	(a)	$P_m \leq S_m$	
	(b)	P_m (or P_L) + $P_B \leq 1.5\ S_m$	1
	(c)	P_m (or P_L) + P_B + Q $\leq 3.0\ S_m$	2
2. Upset Condition	(a)	$P_m \leq S_m$	
	(b)	P_m (or P_L) + $P_B \leq 1.5\ S_m$	1
	(c)	P_m (or P_L) + P_B + Q $\leq 3.0\ S_m$	2
3. Emergency Condition	(a)	$P_m \leq 1.2\ S_m$ or S_y whichever is larger	
	(b)	P_m (or P_L) + $P_B \leq 1.5\ (1.2\ S_m)$ or $1.5\ S_y$ whichever is larger	3
4. Faulted Condition	(a)	$P_m \leq 1.5\ S_m$ or $1.2\ S_y$ whichever is larger	
	(b)	Carbon Steel P_m (or P_L) + $P_B \leq 2.25\ S_m$ or $1.875\ S_y$ whichever is larger Stainless Steel Design limit curves as given in Figures 12.2-2A & B.	

TABLE 12.2-12 LOADING CONDITIONS AND STRESS LIMITS: PRESSURE VESSELS

(Page 2 of 2)

Loading Condition		Stress Intensity Limits	Note*
	(c)	Steam Generator	5
		$P_m < 0.7\ S_u$ (**)	
		$P_L \leq 1.05\ \leq \min\{2.4\ S_m, 0.7\ S_u\}$ (***)	
		$P_L \leq 1.05\ S_u$ (**)	
		$P_L \leq \min\{2.4\ S_m, 0.7\ S_u\}$ (***)	
		$P_m + P_B \leq 1.05\ S_u$ (**)	
		$P_L + P_B \leq 1.05\ S_u$ (**)	
		$P_m + P_B \leq \min\{3.6\ S_m, 1.05\ S_u\}$ (***)	
		$P_L + P_B \leq \min\{3.6\ S_m, 1.05\ S_u\}$ (***)	
	(d)	Replacement Reactor Vessel Head	6
		$P_m \leq \min\{2.4\ S_m, 0.7\ S_u\}$	
		$P_L \leq$ 150% of the limit for P_m	
		$P_L + P_B \leq$ 150% of the limit for P_m	

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P_m = primary general membrane stress intensity

P_L = primary local membrane stress intensity

P_B = primary bending stress intensity

Q = secondary stress intensity

S_m = stress intensity value from ASME B&PV Code, Section III, Nuclear Vessels

S_y = minimum specified material yield strength (ASME B&PV Code, Section III, Table N-421 or equivalent)

S_u = Material ultimate tensile strength (ASME Code Section II)

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* For description of notes see Notes for Tables 12.2-12 and 12.2-13.

** Ferritic materials included in Table 2A of the ASME Code.

*** Materials included in Section II Part D, Subpart 1, Tables 2A and 2B for austenitic and high alloy steels of the ASME Code. 01388733

TABLE 12.2-13 LOADING COMBINATIONS AND STRESS LIMITS: PRESSURE PIPING IN ACCORDANCE WITH USAS B31.1

(Page 1 of 2)

	Loading Condition	Stress Limits
1.	Normal Condition	$P \leq S$
2.	Upset Condition	$P \leq 1.2S$
3.	Emergency Condition	$P \leq 1.5\ (1.2S)$
4.	Faulted Condition	<u>For Stainless Steel</u> Design Limit Curves as defined in Figure 12.2-2 See Note 4* <u>For Carbon Steel</u> $P \leq S_y$ or 1.8S whichever is higher**
5.	Other	See Note 7

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Where:

P = stress

S = Allowable stress from ANSI B31.1, code for Power Piping, 1967 01514406

S_y = Minimum specified yield strength (ASME B&PV Code, Section III, Table N-421 or equivalent)

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* For description of Notes 4 and 7 see notes for Table 12.2-12 and 12.2-13 01514406

** At some points of high local stress intensification, P may exceed this limit. For such points local piping deflection will be limited to twice the calculated OBE deflection to insure no loss of function in the "Faulted Condition".

TABLE 12.2-13 LOADING COMBINATIONS AND STRESS LIMITS: PRESSURE PIPING IN ACCORDANCE WITH USAS B31.1

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Notes for Tables 12.2-12 and 12.2-13

Note 1: The limits on local membrane stress intensity ($P_L \leq 1.5\ S_m$) and primary membrane plus primary bending stress intensity (P_m (or P_L) + $P_B \leq 1.5\ S_m$) need not be satisfied at a specific location if it can be shown by means of limit analysis or by tests that the specified loadings do not exceed 2/3 of the lower bound collapse load as per paragraph N417.6 (b) of the ASME B&PV Code, Section III, Nuclear Vessels.

Note 2: In lieu of satisfying the specific requirements for the local membrane ($P_L \leq 1.5\ S_m$) or the primary plus secondary stress intensity ($P_L + P_B + Q \leq 3S_m$) at the specific location, the structural action may be calculated on plastic basis and the design will be considered to be acceptable if shakedown occurs, as opposed to continuing deformation, and if the deformations which occur prior to shakedown, do not exceed specified limits, as per paragraph N-417.6 (a) (2) of the ASME B&PV Code, Section III, Nuclear Vessels.

Note 3: The limits on local membrane stress intensity ($P_L \leq 1.5\ S_m$) and primary membrane plus primary bending stress intensity (P_m (or P_L) + $P_B \leq 1.5\ S_m$) need not be satisfied at a specific location if it can be shown by means of limit analysis or by test that the specified loadings do not exceed 120 percent of 2/3 of the lower bound collapse load as per paragraph N-417.10 (c) of the ASME B&PV Code, Section III, Nuclear Vessels.

Note 4: As an alternate to the design limit curves which represent a pseudo plastic instability analysis, a plastic instability analysis may be performed in some specific cases considering the actual strain-hardening characteristics of the material, but with yield strength adjusted to correspond to the tabulated value at the appropriate temperature in Table N-424 or N-425, as per paragraph N417.11 (c) of the ASME B&PV Code, Section II, Nuclear Vessels. These specific cases will be justified on an individual basis.

This alternate design procedure was not utilized on this application.

Note 5: Faulted condition stress intensity limits for the RSG design is defined in ASME Section III 1995 Edition 1996 Addenda Subsection NB-3225 and Appendix F.

Note 6: Faulted condition stress intensity limits for the Replacement Reactor Vessel Head is defined in ASME Section III 1998 Edition 2000 Addenda Subsection NB-3225 and Appendix F.

Note 7: As described in References 67 and 68, the Pressurizer Safety and Relief Discharge piping is also qualified to the stress limits of Thermal loads ≤ $1.0S_a$ and (Pressure + Deadweight + Thermal) ≤ ($1.0S_h + 1.0S_a$).

TABLE 12.2-14 LOADING COMBINATIONS AND STRESS LIMITS: EQUIPMENT SUPPORTS

	Loading Conditions	Stress Limits
1.	Normal Condition	Working Stresses or Applicable Factored Load Design Values
2.	Upset Condition	Working Stresses or Applicable Factored Load Design Values
3.	Emergency Condition	Within Yield after Load Redistribution to Maintain Supported Equipment Within Emergency Condition Stress Limits
4.	Faulted Condition	Permanent Deflection of Supports Limited to Maintain Supported Equipment Within Faulted Condition Stress Limits

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TABLE 12.2-15 LOAD COMBINATIONS AND STRESS LIMITS FOR CLASS I COMPONENTS

	Loading Conditions	Stress Limits
1.	Normal (Deadweight, thermal and pressure)	Normal Conditions
2.	Normal and Operational Basis Earthquake	Upset Condition
3.	Normal and Design Basis Earthquake	Faulted Condition
4.	Normal and Pipe Rupture	Faulted Condition
5.	Normal and Design Basis Earthquake and Pipe Rupture	Faulted Condition

TABLE 12.2-16 STEEL AND CONCRETE STRESSES IN CONTAINMENT VESSEL BOTTOM AT MEZZANINE FLOOR LINE

	Calculated Stress (PSI)					[1] Allowable Stress (PSI)	
	Horizontal Seismic Loads Only		Vessel Dead Loads Plus Internal Accident Pressure (41.4 PSI)	Total			
Type of Stress	**OBE**	**DBE**		**OBE**	**DBE**	**OBE**	**DBE**
Average Vessel Plate Membrane Hoop Stress in 20 in. Wide Band	6330	12660	14330	20660	26990	26250	30450
Approximate Bearing Stress of 20 in. Wide Hoop Band Bearing Against Internal Concrete	15	30	N.A.	15	30	1500 [2]	2250 [2]

[1] See Table 12.2-17

[2] Maximum Allowable Bearing Stress on Concrete (See ACI 318-63)

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TABLE 12.2-17 ALLOWABLE STRESS CRITERIA – REACTOR CONTAINMENT VESSEL

Material: ASTM A516 Grade-70

Loading Conditions	Limits of Stress Intensity[1]
1. Dead loads plus operating loads plus DBA loads plus Operational Basis Earthquake Loads (0.06g).	ASME Boiler and Pressure Vessel Code, Section III, Figure N-414
2. Dead loads plus Operating loads plus DBA loads plus Design Basis Earthquake Loads (0.12g)	Safe shutdown of plant can be achieved a) $P_m \leq 1.16\ S_m$ b) $P_L + P_b \leq 1.16\ (1.5\ S_m)$ c) $P_L + P_b + Q \leq 3.0\ S_m$
3. Dead loads plus operating loads plus pipe rupture forces plus Operating basis or Design Basis Earthquake Loads.	Analysis done on an elastic basis a) $P_m \leq 1.5\ S_m$ or $1.2\ S_y$ whichever is larger b) P_m (or P_L) + $P_b \leq 2.25\ S_m$ or $1.875\ S_y$ whichever is larger

P_m = Primary general membrane stress intensity

P_L = Primary local membrane stress intensity

P_b = Primary bending stress intensity

Q = Secondary stress intensity

S_m = Allowable stress intensity value

S_y = Maximum specified material yield strength (ASME B&PV Code, Section III, Table N-421 or equivalent)

[1] Refer to Table N-414 ASME Boiler and Pressure Vessel Code, Section III

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TABLE 12.2-18 MEMBRANE AND SURFACE STRESSES

	Membrane Stress (psi) Actual			Surface Stress (psi) Actual		
Case	**Longitudinal**	**Circumferential**	**Allowable**	**Longitudinal**	**Circumferential**	**Allowable**
		PERSONNEL AIRLOCK				
OBE	17813	17488	26250	26328	26430	52500
DBE	17850	17566	30450	27003	27732	52500
		MAINTENANCE AIRLOCK				
OBE	17941	17485	26250	26713	26386	52500
DBE	18000	17563	30450	26898	27680	52500

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TABLE 12.2-19 STRESSES DUE TO TEMPERATURE DIFFERENCE

	Top of Cold Spot	Bottom of Cold Spot	Bottom of Cylinder 5.5 ft Below Cold Spot
Membrane Stresses			
Meridional Stress	.58ksi	.82ksi	.84ksi
Circumferential Stress	2.20	2.24	.25
Bending Stresses			
Meridional Stress	4.65	.40	.57
Circumferential Stress	4.46	4.11	.17

TABLE 12.2-20 STRESSES DUE TO PRESSURE DIFFERENCE

	Top of Cold Spot	Bottom of Cold Spot	5.5 ft Below Cold Spot
Membrane Stresses			
Meridional Stress	1.64ksi	4.35ksi	6.72ksi
Circumferential Stress	.06	.04	2.02
Bending Stresses			
Meridional Stress	1.27	.39	7.20
Circumferential Stress	3.74	.94	2.16

TABLE 12.2-21 SUMMARY OF STRESSES WITHOUT COLD SPOT

	Top of Cold Spot	Bottom of Cold Spot	5.5 ft Below Cold Spot
With Operating Basis Earthquake			
Membrane Stresses			
Meridional Stress	8.45ksi	8.39ksi	8.38ksi
Circumferential Stress	17.29ksi	17.39ksi	7.22ksi
With Design Earthquake			
Membrane Stresses			
Meridional Stress	8.66ksi	8.68ksi	8.68ksi
Circumferential Stress	17.39ksi	17.39ksi	7.22ksi

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TABLE 12.2-22 COMBINED STRESSES, KSI
(Page 1 of 2)

	Top of Cold Spot	Bottom of Cold Spot	5.5 Below Cold Spot	Allowable Stress Intensity
WITH OPERATING BASIS EARTHQUAKE				
Membrane Stresses Only - including local membrane (No Temperature Stresses)				
Meridional	10.09	12.74	15.10	
Circumferential	17.45	17.43	9.24	
Max. Stress Intensity	17.50	19.01*	16.10	17.50
Membrane and Bending Stresses (No Temperature Stresses)				
Meridional	11.36	13.13	22.30	
Circumferential	21.19	18.37	11.40	
Max. Stress Intensity	21.23	19.84	22.91	26.25
All Stresses, Including Temperature				
Meridional	16.59	14.35	23.71	
Circumferential	27.85	24.72	11.82	
Max. Stress Intensity	27.86	25.34	24.19	52.50
WITH DESIGN BASIS EARTHQUAKE				
Membrane Stresses Only - including local membrane (No Temperature Stresses)				
Meridional	10.30	13.03	15.40	
Circumferential	17.45	17.43	9.24	
Max. Stress Intensity	17.50	19.07	16.36	20.30
Membrane and Bending Stresses (No Temperature Stresses)				
Meridional	11.57	13.42	22.60	
Circumferential	21.19	18.37	11.40	
Max. Stress Intensity	21.23	19.90	23.18	30.45

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TABLE 12.2-22 COMBINED STRESSES, KSI
(Page 2 of 2)

All Stresses, Including Temperature				
Meridional	16.80	14.64	24.01	
Circumferential	27.85	24.72	11.82	
Max. Stress Intensity	27.86	25.36	24.48	52.50

*The maximum membrane stress intensity is 20530 psi and occurs at 6″ below the bottom of this “cold spot”. Because this point is not within 2.5 $(Rt)^{1/2}$, of another point where the membrane stress intensity exceeds 1.1 Sm or 19250 psi, and since the height of the area stressed above 1.1 Sm does not exceed 0.5 $(Rt)^{1/2}$, this area is classified as a local stress region with an allowable membrane stress intensity of 1.5 Sm or 26250 psi in accordance with Paragraph N412 (j) of Section III of the ASME Boiler and Pressure Vessel Code.

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TABLE 12.2-23 MAXIMUM STRESSES IN THE CONTAINMENT SHELL AT THE EXTERNAL EMBEDMENT LINE

	Calculated Stress (PSI)					[2] Allowable Stress (PSI)	
	Vertical & Horizontal Seismic Loads		Vessel Dead Loads Plus Internal Accident Pressure	Total			
Type of Stress	**OBE**	**DBE**	**(41.4 PSI)**	**OBE**	**DBE**	**OBE**	**DBE**
Membrane:							
Meridian	423	846	7878	8301	8724	17500	20300
Circumferential	238	476	3594	3832	4070	17500	20300
Surface:							
Meridian	259	518	22438[1]	22697	22956	26250	30450
Circumferential	78	156	6052[1]	6130	6208	26250	30450
Shear:							
Tangential	293	586	N.A.	293	586	N.A.	N.A.
Transverse	5.3	10.6	N.A.	5.3	10.6	N.A.	N.A.

(1) Accident Pressure (41.1 Psi) ONLY

(2) See Table 12.2-17

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TABLE 12.2-24 FACTORS OF SAFETY FOR CLASS I STRUCTURES AGAINST UPLIFT

BUILDING	DEAD LOAD (Kips)	UPLIFT (Kips)	FACTOR OF SAFETY
Turbine Bldg.	39,300	32,180	1.22
Class I Area in Turbine Building	18,400	9,100	2.02
Aux. Bldg.	110,478	39,600	2.79
Screenhouse	36,796	24,369	1.51
Radwaste Bldg.	4,600	2,420	1.9

TABLE 12.2-25 ALLOWABLE STRESSES – SHIELD BUILDING

Reinforced Concrete - Structural Steel

	Loading Condition	Reinforced Concrete	Structural Steel
1.	Dead loads plus live loads plus DBA load	ACI 318-63 Allowable values	AISC Allowable values
2.	Dead loads plus live loads plus DBA load plus wind load or seismic load (0.06g)	ACI 318-63 Allowable values	AISC Allowable values
3.	Dead loads plus live loads plus DBA load plus seismic load (0.12g)	1-1/2 times ACI 318-63 Allowable values	1-1/2 times AISC Allowable values
4.	Dead loads plus live loads plus 300 mph design tornado	$f_c = 0.75 f'_c$ $f_s = 0.90$ Y.S.	$f_s = 0.90$ Y.S.

f'_c = Minimum 28 day compressive strength of concrete

f_c = Compressive stress in concrete

f_s = Tensile stress in steel

Y.S. = Specified minimum yield strength or yield point of steel.

TABLE 12.2-26 ALLOWABLE STRESS CRITERIA – SHIELD BUILDING

			A-15 & A-408 Bars		A-432 Bars	
Loading Condition	**Criteria**	**f_c**	**Allowable Working Stress-psi**	**Percent of Min Spec Yield (1)**	**Allowable Working Stress psi**	**Percent of Min Spec Yield (1)**
1. Normal loads + DBA	ACI 318-63 allowable values	$0.45\ f'_c$	20,000	50.	24,000	40.
2. Normal loads + DBA + wind (or) ± operating basis earthquake	ACI 318-63 allowable values	$0.45\ f'_c$	20,000	50.	24,000	40.
3. Normal loads + DBA ± design basis earthquake	1-1/2 times ACI 318-63 allowable values	$0.675\ f'_c$	30,000	75.	36,000	60.

(1) Minimum specified yield points of steel reinforcement are as follows:

A-15 40,000 psi

A-408 40,000 psi

A-432 60,000 psi

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TABLE 12.2-27 STRESSES IN DOME

(Page 1 of 5)

		Stresses (PSI)					
		Membrane		Flexural		**Combined**	
Element No.	**Load Case**	**Circumferential**	**Meridional**	**Circumferential**	**Meridional**	**Circumferential**	**Meridional**
	D.L.	71.38	-41.97	-13.27	+91.67	58.11	49.70 T
				+13.27	-91.67	84.65	-133.64 B
217	R.M.S.	±34.23	±6.43	±5.58	±19.30	±39.81	±12.01
	D.L. & R.M.S.					97.92	61.71 T
						124.46	-145.65 B
	D.L.	25.91	-47.16	+16.29	-31.43	42.20	-78.59
				-16.29	+31,43	9.62	-15.73
235	R.M.S.	±21.90	±1.04	±8.13	±13.20	±30.03	±14.24
	D.L. & R.M.S.					72.23	-92.83
						39.65	-29.97
	D.L.	-18.21	-49.02	+14.83	-40.10	-3.38	-89.12
				-14.83	+40.10	-33.04	-8.92
253*	R.M.S.	±16.02	±4.08	±8.69	±10.13	±24.71	±14.21
	D.L. & R.M.S.					21.33	-103.33
						-57.75	5.29
	D.L.	-38.02	-48.00	+66.01	-18.96	27.99	-66.96
				-66.01	+18.96	-104.03	-29.04
271	R.M.S.	±15.02	±5.03	±1.51	±1.88	±16.53	±6.91
	D.L. & R.M.S.					44.52	-73.87
						-120.56	-35.95
	D.L.	-42.93	-45.86	+1.60	-4.61	-41.33	-50.47
				-1.60	+4.61	-44.53	-41.25
289	R.M.S.	±14.31	±7.13	±2.79	±6.26	±17.10	±13.39
	D.L. & R.M.S.					-58.43	-63.86
						-61.63	-54.64

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NOTE:
T = TOP FIBER
B = BOTTOM FIBER
\- = COMPRESSION
\+ = TENSION

* See end of Table

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TABLE 12.2-27 STRESSES IN DOME

(Page 2 of 5)

		Stresses (PSI)					
		Membrane		Flexural		Combined	
Element No.	Load Case	Circumferential	Meridional	Circumferential	Meridional	Circumferential	Meridional
	D.L.	-43.12	-43.79	+0.10	-1.21	-43.02	-45.00
				-0.10	+1.21	-43.22	-42.58
307	R.M.S.	±11.95	±12.11	±0.50	±2.93	±12.45	±15.04
	D.L. & R.M.S.					-55.47	-60.04
						-55.67	-57.62
	D.L.	-42.38	-42.08	-2.25	-2.56	-44.63	-44.64
				+2.25	+2.56	-40.13	-39.52
325	R.M.S.	±12.51	±24.64	±2.61	±2.58	±15.12	±27.22
	D.L. & R.M.S.					-59.75	-71.68
						-55.25	-66.74
	D.L.						
343	R.M.S.						
	D.L. & R.M.S.						
	D.L.						
351	R.M.S.						
	D.L. & R.M.S.						
	D.L.	-42.40	-42.08	-2.25	-2.56	-44.65	-44.64
				+2.25	+2.56	-40.15	-39.52
342	R.M.S.	±12.39	±24.44	±3.14	±3.35	±15.53	±27.79
	D.L. & R.M.S.					-60.18	-72.43
						-55.68	-67.31
	D.L.	-43.12	-43.79	+0.10	-1.21	-43.02	-45.00
				-0.10	+1.21	-43.22	-42.58
324	R.M.S.	±11.33	±12.10	±0.88	±2.06	±12.21	±14.16
	D.L. & R.M.S.					-55.23	-59.16
						-55.43	-56.74

TABLE 12.2-27 STRESSES IN DOME

(Page 3 of 5)

Element No.	Load Case	Membrane Circumferential	Membrane Meridional	Flexural Circumferential	Flexural Meridional	Combined Circumferential	Combined Meridional	
		Stresses (PSI)						
	D.L.	-42.93	-45.86	+1.60	-4.61	-41.33	-50.47	
				-1.60	+4.61	-44.53	-41.25	
306	R.M.S.	±12.44	±7.15	±3.14	±1.80	±15.58	±8.95	
	D.L. & R.M.S.					-56.91	-59.42	
						-60.11	-50.20	
	D.L.	-38.02	-48.0	+6.60	-18.96	-31.42	-66.96	98124
				-6.60	+18.96	-44.62	-29.04	
288	R.M.S.	±14.06	±3.77	±3.54	±6.73	±17.60	±10.50	
	D.L. & R.M.S.					-49.02	-77.46	
						-62.22	-39.54	
	D.L.	-18.21	-49.02	+14.83	-40.10	-3.38	-89.12	
				-14.83	+40.10	-33.04	-8.92	
270	R.M.S.	±19.11	±2.12	±1.38	±3.81	±20.49	±5.93	98124
	D.L. & R.M.S.					17.11	-95.05	
						-53.53	-14.85	
	D.L.	25.91	-47.16	+16.29	-31.43	42.20	-78.59	
				-16.29	+31.43	9.62	-15.73	
252	R.M.S.	±26.92	±4.17	±3.45	±4.17	±30.37	±8.34	
	D.L. & R.M.S.					72.57	-86.93	
						39.99	-24.07	
	D.L.	71.38	-41.97	-13.27	+91.67	58.11	49.70	
				+13.27	-91.67	84.65	-133.64	
234	R.M.S.	±38.30	±9.02	±10.64	±25.52	±48.94	±34.54	
	D.L. & R.M.S.					107.05	84.24	
						133.59	-168.18	
	D.L.	71.38	-41.97	-13.27	91.67	58.11	49.70	98124
				13.27	-91.67	84.65	-133.64	
225	R.M.S.	±2.95	±1.11	±0.70	±1.69	±3.65	±.2.8	
	D.L. & R.M.S.					61.76	52.50	
						88.30	-136.44	

TABLE 12.2-27 STRESSES IN DOME

(Page 4 of 5)

Element No.	Load Case	Membrane Circumferential	Membrane Meridional	Flexural Circumferential	Flexural Meridional	Combined Circumferential	Combined Meridional
	D.L.	25.91	-47.16	16.29	-31.43	42.20	-78.59
				-16.29	31.43	9.62	-15.73
243	R.M.S.	±2.97	±0.79	±1.63	±0.64	±4.60	±1.63
	D.L. & R.M.S.					46.80	-80.02
						14.22	-17.16
	D.L.	-18.21	-49.02	14.83	-40.10	-3.38	-89.12
				-14.83	40.10	-33.04	-8.92
261	R.M.S.	±2.10	±0.65	±1.75	±0.48	±3.85	±1.13
	D.L. & R.M.S.					0.47	-90.25
						-36.89	-10.05
	D.L.	-38.02	-48.00	6.60	-18.96	-31.42	-66.96
				-6.60	18.96	-44.62	-29.04
279	R.M.S.	±1.33	±0.64	±0.41	±0.61	±1.74	±1.25
	D.L. & R.M.S.					-33.16	-68.21
						-46.36	-30.29
	D.L.	-42.93	-45.86	-1.60	-4.61	-44.53	-50.47
				1.60	4.61	-41.33	-41.25
297	R.M.S.	±1.14	±0.80	±0.08	±0.37	±1.15	±1.17
	D.L. & R.M.S.					-45.68	-51.64
						-42.48	-42.42
	D.L.	-43.12	-43.79	0.10	-0.10	-43.02	-43.89
				-0.10	0.10	-43.22	-43.69
315	R.M.S.	±1.07	±1.11	±0.09	±0.20	±1.16	±1.31
	D.L. & R.M.S.					-44.18	-45.20
						-44.38	-45.00
	D.L.	-42.40	-42.08	-2.25	-2.56	-44.65	-44.64
				2.25	2.56	40.15	-39.52
333	R.M.S.	±0.45	±2.25	±0.02	±0.18	±0.47	±2.43
	D.L. & R.M.S.					-45.12	-47.07
						-40.62	-41.95

Stresses (PSI)

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TABLE 12.2-27 STRESSES IN DOME

(Page 5 of 5)

		Stresses (PSI)					
		Membrane		**Flexural**		**Combined**	
Element No.	**Load Case**	**Circumferential**	**Meridional**	**Circumferential**	**Meridional**	**Circumferential**	**Meridional**
	D.L.	-18.21	-49.02	14.83	-40.10	-3.38	-89.12
				-14.83	40.10	-33.04	-8.92
253	MODE #11	-0.25	1.25	70.07	±1.34	-0.18/-0.32	-0.11/+2.59
	D.L. + #11					-3.70	-86.53
						-33.22	-9.03
	D.L.	-18.21	-49.02	14.83	-40.10	-3.38	-89.12
				-14.83	40.10	-33.04	-8.92
253	MODE #1	16.09	-3.54	±8.68	710.00	-7.41/24.77	6.46/-13.54
	D.L. + #1					21.39	-102.66
						-40.45	-2.46
	D.L.	-18.21	-49.02	14.83	-40.10	-3.38	-89.12
				-14.83	40.10	-33.04	-8.92
253	MODE #17	-0.48	-0.41	70.10	±0.25	-0.38/-0.58	-0.65/-0.16
	D.L. + #17					-3.96	-89.28
						-33.42	-9.57

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TABLE 12.2-28 STRESSES IN DOME
(Page 1 of 4)

		Stresses (p.s.i.)					
		Membrane		Flexural		Combined	
Element Number	Load Case	Circumferential	Meridional	Circumferential	Meridional	Circumferential	Meridional
	D.L.	71.38	-41.97	713.27	691.67	84.65	49.70
	3	14.41	2.50	-0.39	4.52	14.02	7.02
217	4	37.26	-10.92	-4.63	-24.81	32.63	-35.73
	D.L. + 3					98.67	56.72
	D.L. + 4					117.28	13.97
	D.L.	25.91	-47.16	616.29	731.43	42.20	-78.59
	3	16.66	3.73	-2.22	-3.88	14.44	-0.15
235	4	24.95	-11.36	2.99	-6.18	27.94	-5.18
	D.L. + 3					56.64	-78.74
	D.L. + 4					70.14	-83.77
	D.L.	-18.21	-49.02	614.83	740.10	-3.38	-89.12
	3	18.06	4.50	1.75	-3.83	19.81	0.67
253	4	12.24	-11.19	3.00	9.00	15.24	-2.19
	D.L. + 3					16.43	-88.45
	D.L. + 4					11.86	-91.31
	D.L.	-38.02	-48.00	666.01	718.96	27.99	-66.96
	3	17.32	4.94	-0.93	-1.95	16.39	2.99
271	4	5.15	-10.42	1.18	4.12	6.33	-6.30
	D.L. + 3					44.38	-63.97
	D.L. + 4					22.72	-73.26

For D.L.: USE RESULTS OF CASE 2 AND DIVIDED BY -0.16

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TABLE 12.2-28 STRESSES IN DOME
(Page 2 of 4)

		Stresses (p.s.i.)						
		Membrane		Flexural		Combined		
Element Number	Load Case	Circumferential	Meridional	Circumferential	Meridional	Circumferential	Meridional	
	D.L.	-42.93	-45.86	61.60	74.61	-44.53	-50.47	98124
	3	15.31	5.25	-0.44	-0.80	14.87	4.45	
289	4	1.58	-9.42	0.07	0.67	1.65	-8.75	
	D.L. + 3					-29.66	-46.02	
	D.L. + 4					-42.88	-59.22	
	D.L.	-43.12	-43.79	60.10	71.21	-43.22	-45.00	98124
	3	12.86	5.52	-0.19	-0.50	12.67	5.02	
307	4	0.94	-8.49	-0.16	-0.11	0.78	-8.60	
	D.L. + 3					-30.55	-39.98	
	D.L. + 4					-42.44	-53.60	
	D.L.	-42.38	-42.08	72.25	72.56	-44.63	-44.64	98124
	3	10.34	5.74	0.35	-0.44	10.69	5.30	
325	4	-3.22	-7.73	-0.37	0.38	-3.59	-7.35	
	D.L. + 3					-33.94	-39.34	
	D.L. + 4					-48.22	-51.99	
	D.L.	-42.40	-42.08	72.25	72.56	-44.65	-44.64	
	3	3.22	7.73	0.37	-0.38	3.59	7.35	
342	4	-10.34	-5.74	-0.35	0.44	10.69	-5.30	
	D.L. + 3					-41.06	-37.29	
	D.L. + 4					-33.96	-49.94	

TABLE 12.2-28 STRESSES IN DOME
(Page 3 of 4)

		Stresses (p.s.i.)						
		Membrane		Flexural		Combined		
Element Number	Load Case	Circumferential	Meridional	Circumferential	Meridional	Circumferential	Meridional	
	D.L.	-43.12	-43.79	60.10	71.21	-43.22	-45.00	98124
	3	0.94	8.49	0.16	0.11	1.10	8.60	
324	4	-12.86	-5.52	0.19	0.50	-12.67	-5.02	
	D.L. + 3					-42.12	-36.40	
	D.L. + 4					-55.89	-50.02	
	D.L.	-42.93	-45.86	61.60	74.61	-44.53	-50.47	
	3	-1.58	9.42	-0.07	-0.67	-1.65	8.75	
306	4	-15.31	-5.25	0.44	-0.80	-14.87	-4.45	
	D.L. + 3					-46.18	-41.72	98124
	D.L. + 4					-59.40	-54.92	
	D.L.	-38.02	-48.00	66.60	718.96	-44.62	-66.96	
	3	-5.15	10.42	-1.18	-4.12	-6.33	6.30	
288	4	-17.32	-4.94	0.93	1.95	-16.39	-2.99	
	D.L. + 3					-50.95	-60.66	
	D.L. + 4					-61.01	-69.95	98124

TABLE 12.2-28 STRESSES IN DOME
(Page 4 of 4)

		Stresses (p.s.i.)						
		Membrane		Flexural		Combined		
Element Number	Load Case	Circumferential	Meridional	Circumferential	Meridional	Circumferential	Meridional	
	D.L.	-18.21	-49.02	614.83	740.10	-33.04	-89.12	98124
	3	-12.24	11.19	-3.00	-9.00	-15.24	2.19	
270	4	-18.06	-4.50	1.75	3.83	-16.31	-0.67	
	D.L. + 3					-48.28	-86.93	
	D.L. + 4					-49.35	-89.79	
	D.L.	25.91	-47.16	616.29	731.43	42.20	-78.59	
	3	-24.95	11.36	-2.99	-6.18	-27.94	5.18	
252	4	-16.66	-3.72	2.22	3.88	-14.44	0.16	98124
	D.L. + 3					14.26	-73.41	
	D.L. + 4					27.76	-78.43	
	D.L.	71.38	-41.97	713.27	691.67	84.65	49.70	
	3	-37.26	10.92	4.63	24.81	-32.63	35.73	
234	4	-14.41	-2.51	0.39	-4.52	-14.02	-7.03	98124
	D.L. + 3					52.02	85.43	
	D.L. + 4					70.63	42.67	

Loading 3 - Forces from 25% g horizontal acceleration combined with 16% g vertical acceleration upward.
Loading 4 - Same as loading 3 with vertical acceleration acting downward.

TABLE 12.2-29 ALLOWABLE STRESSES – INTERNAL STRUCTURES
REINFORCED CONCRETE – STRUCTURAL STEEL

	Loading Conditions	Reinforced Concrete (2)	Structural Steel
1.	Dead loads plus live loads plus DBA load	ACI 318-63 Allowable Values	AISC Allowable Values
2.	Dead loads plus live loads plus DBA load plus seismic load (0.06g)	ACI 318-63 Allowable Values	AISC Allowable Values
3.	Dead loads plus live loads plus DBA load plus seismic load (0.12g)	1-1/2 Times ACI 318-63 Allowable Values	1-1/2 Times AISC Allowable Values
4.	Dead loads plus live loads plus pipe rupture (1)	$f_c = 0.75\ f'_c$ $f_s = 0.90$ Y.S.	$f_s = 0.90$ Y.S.
5.	Missile loads	$f_c = 1.25\ f'_c$ f_s = Stress @ 0.50 Ultimate Strain	$f_s = 0.90$ Y.S.

05-049

f'_c = Minimum 28 day compressive strength of concrete
f_c = Compressive stress in concrete
f_s = Tensile stress in steel
Y.S. = Specified minimum yield strength or yield point of steel

NOTES:

(1) Criterion for pipe rupture is based on maximum usable in pipe as per Westinghouse Electric Corporation "Support Design Criteria – Reactor Coolant System Components."

(2) ACI Allowable Values are working stress design allowable stresses. For Ultimate Strength Design requirements, refer to USAR Section 12.2.15. 05-049

TABLE 12.2-30 ASSUMPTIONS AND RESULTS OF CALCULATION OF WALL TEMPERATURES AND TOTAL STRESSES

Wall density	144 lbm/ft^3
conductivity	0.73 BTU/hr ft°F
heat capacity	0.2 BTU/lbm°F
Containment temperatures	output of the revised CONTEMPT code with condensate heat transfer rate 25% higher than that of Tag- 98124
Annulus air temperatures	ami's output of the Shield Building Computer Code with best estimated heat transfer rates.
Compression Stress for concrete	
Allowable	1.8Ksi
Evaluated*	1.51Ksi
Tension Stress for steel	
Allowable	24 Ksi
Evaluated*	22.42 Ksi

*Including all stresses due to thermal, dead load, live load, design earthquake, membrane shear, and out of plumb effects. 98124

TABLE 12.2-31 LOADS AND LOAD COMBINATIONS FOR FINITE ELEMENT

Loading Case Number	Loads and Load Combinations
1	Pipe Rupture; Hot Leg, Horizontal Run-A
2	Pipe Rupture; Hot Leg, Horizontal Run-B
3	Pipe Rupture; Cross-over Leg, Vertical Run, S.G. Side-A
4	Pipe Rupture; Cross-over Leg, Vertical Run, S.G. Side-B
5	Pipe Rupture; Cross-over Leg, Horizontal Run-A
6	Pipe Rupture; Cross-over Leg, Horizontal Run-B
7	Pipe Rupture; Cold Leg - A
8	Pipe Rupture; Cold Leg - B
9	Pipe Rupture; Cross-over Leg, Run-A (Split)
10	Pipe Rupture; Cross-over Leg, Run-B (Split)
11	Load Combination: 0.766DDP(1)(4) + (Case 1) + DBE(2)
12	Load Combination: 0.766DDP(4) + (Case 2) + DBE
13	Load Combination: 0.766DDP(4) + (Case 3) + DBE
14	Load Combination: 0.766DDP(4) + (Case 4) + DBE
15	Load Combination: 0.766DDP(4) + (Case 5) + DBE

TABLE 12.2-31 LOADS AND LOAD COMBINATIONS FOR FINITE ELEMENT
(Page 2 of 2)

Loading Case Number	Loads and Load Combinations
16	Load Combination: 0.766DDP[(4)] + (Case 6) + DBE
17	Load Combination: 0.766DDP[(4)] + (Case 7) + DBE
18	Load Combination: 0.766DDP[(4)] + (Case 8) + DBE
19	Load Combination: 0.766DDP[(4)] + (Case 9) + DBE
20	Load Combination: 0.766DDP[(4)] + (Case 10) + DBE
21	Design Differential Pressure = 27.4 PSI[(3)]
22	DDP + DBE
23	DDP + (0.5) x DBE
24	DDP + (1.0) x DBE

---

[1] Design Differential Pressure (DDP)

[2] Design Basis Earthquake (DBE)

[3] Maximum differential pressure for Unit 1 based on the configuration for Steam Generator/RCP compartment 1A. The confirmatory finite element analysis for compartment 1A uses a differential pressure of 28.6 psi. Unit 2 has a maximum differential pressure of 27 psi, based on the configuration for Steam Generator/RCP compartment 2A, and the finite element analysis for compartment 2A uses a differential pressure of 27 psi.

[4] Unit 2 finite element analysis uses 0.778DDP in the equivalent load combinations 11 through 20.

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TABLE 12.2-32 DESIGN STRESS LIMITS FOR REACTOR COOLANT SYSTEM COMPARTMENTS AND REFUELING CAVITY

(Page 1 of 2)

DESIGN CONDITION CATEGORIES	DESIGN STRESS LIMITS FOR REINFORCED CONCRETE		DESIGN STRESS LIMITS FOR STRUCTURAL STEEL	
	CRITERIA	STRESS LIMIT	CRITERIA	STRESS LIMIT
1. Normal Operation Operating Loads (1)	ACI-318 (WSD)	Code Allowable	AISC (Part 1)	Code Allowable
2. Design Basis Accident Operating Load + DBA (Maximum Internal Differential Pressure Load) (2) + OBE	”	”	”	”
3. Safe Shut-Down (4) Operating Load + DBA (Maximum Internal Differential Pressure Load) (2) + DBE	”	1.5x (Code Allowable)	”	1.5x (Code Allowable)
4. Pipe Rupture Reaction Loads and Jet Loads (Local effect) (5)				
a) Operating Load + DBA (Maximum Pipe Rupture Reaction Load) (2) + DBE	”	0.75 f'_c (concrete) 0.9 Y.S. (Reinforcement)	”	0.9 Y.S.

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TABLE 12.2-32 DESIGN STRESS LIMITS FOR REACTOR COOLANT SYSTEM COMPARTMENTS AND REFUELING CAVITY

DESIGN CONDITION CATEGORIES	DESIGN STRESS LIMITS FOR REINFORCED CONCRETE		DESIGN STRESS LIMITS FOR STRUCTURAL STEEL	
	CRITERIA	STRESS LIMIT	CRITERIA	STRESS LIMIT
b) Operating Load + DBA (Pipe Rupture Reaction Load & Internal Differential Pressure Load) (2) (3) + DBE	”	”	”	”

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NOTES:

1. Operating load includes:
   a) Dead load
   b) Live Load
   c) Residual construction loads
2. DBA Load includes thermal loads
3. The combination of pipe rupture reaction load and jet load with the associated internal differential pressure load that resulted in the most severe effect on the structure was used in the analysis and design.
4. Safe shutdown analysis included the general effects of pipe rupture loads on the complex of integrated structures.
5. Safe shutdown analyzed for the local effects of pipe rupture loads and jet loads.

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TABLE 12.2-33 STEAM GENERATOR AND REACTOR COOLANT PUMP COMPARTMENT MAXIMUM STRESS AND STRESS LIMITS

(Page 1 of 2)

	Design Stress Limits For Reinforced Concrete		Design Stress Limits For Reinforcing Steel		
Load Combination	**Design Stress (Psi)**	**Stress Limit (Psi)**	**Design Stress (Psi)**	**Stress Limit (Psi)**	
1. Normal Operation Operating Loads	NOT CRITICAL	NOT CRITICAL	NOT CRITICAL	NOT CRITICAL	
2. Design Basis Accident					
Operating Load + DBA (Maximum Internal Differential Pressure Load) + OBE	780	1,800	22,400	24,000	01210617
3. Safe Shutdown					
Operating Load + DBA (Maximum Internal Differential Pressure Load) + DBE	860	2,700	24,600	36,000	01210617

TABLE 12.2-33 STEAM GENERATOR AND REACTOR COOLANT PUMP COMPARTMENT MAXIMUM STRESS AND STRESS LIMITS
(Page 2 of 2)

Load Combination	Design Stress Limits For Reinforced Concrete: Design Stress (Psi)	Design Stress Limits For Reinforced Concrete: Stress Limit (Psi)	Design Stress Limits For Reinforcing Steel: Design Stress (Psi)	Design Stress Limits For Reinforcing Steel: Stress Limit (Psi)
4. Pipe Rupture Reaction Loads and Jet Loads (Local effect)				
a) Operating Load + DBA (Pipe Rupture Reaction Load & Internal Differential Pressure Load) (28.6 Psi) + DBE	2,040	3,400	52,100	54,000
b) Operating Load + DBA (Pipe Rupture Reaction Load & Internal Differential Pressure Load) (50 Psi) + DBE	1,500	3,400	52,050	54,000
c) Operating Load + DBA (Pipe Rupture Reaction Load & Internal Differential Pressure Load) (75 Psi) + DBE	1,530	4,000	57,750	60,000

01210617

TABLE 12.2-34 REACTOR VESSEL CAVITY MAXIMUM STRESS AND STRESS LIMITS

	Design Stress Limits For Reinforced Concrete		Design Stress Limits For Reinforcing Steel	
	Design Stress (psi)	**Stress Limit (psi)**	**Design Stress (psi)**	**Stress Limit (psi)**
1. Normal Operating Loads (Dead Loads + Thermal)	389	1,800	6,150	24,000
2. Dead Loads + Thermal + OBE + Differential Pressure (100 psi)	479	1,800	12,351	24,000
3. Dead Loads + Thermal + DBE + Maximum Differential Pressure (185 psi)	569	2,700	22,800	36,000
4. Dead Loads + Thermal + DBE + Maximum Hypothetical Differential Pressure (480 psi)	569	4,000	59,284	60,000

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TABLE 12.2-35 NOZZLE CAVITY MAXIMUM STRESS AND STRESS LIMITS

	Design Stress (psi)	Stress Limit (psi)
1. Differential Pressure = 475 psi	10,133	30,000
2. Differential Pressure = 1400 psi	29,870	45,000

TABLE 12.2-36 DESIGN STRESS LIMITS FOR STRUCTURAL EQUIPMENT SUPPORTS (1) 01209592

DESIGN CONDITION CATEGORIES	DESIGN STRESS LIMITS (2) STEEL	CONCRETE
1. Normal Conditions: Operating Loads	0.6 Fy	0.45 f′c
2. Upset Condition: Operating Loads + OBE	0.6 Fy	0.45 f′c
3. Emergency Condition: Operating Loads + DBE	0.9 Fy	0.675 f′c
4. Faulted Condition:		
a) Operating Loads + DBA (3)		
1. Duration: > 0.01 Sec.	0.9 Fy	0.675 f′c
2. Duration: ≤ 0.01 Sec.	.95 Fy or 0.8 Fu	0.75 f′c
b) Operating Loads + DBA + DBE		
1. Duration: > 0.01 Sec.	Fy or 0.85 Fu	0.75 f′c
2. Duration: ≤ 0.01 Sec.	1.1 Fy or 0.9 Fu	0.85 f′c

01209592

NOTES:

1. Design stress limit values in this table are applicable for analysis performed during or after December 1981. For analysis performed before December 1981, FSAR Appendix B, Table B.7-4, “LOADING CONDITIONS AND STRESS LIMITS FOR EQUIPMENT SUPPORTS”, is applicable.

2. Fy = Minimum Yield Stress
   Fu = Ultimate Tensile Stress
   f′c = Minimum 28-Day Compressive Strength of Concrete

3. DBA Load Includes: 01209592
   a) Pipe Rupture Reaction Loads
   b) LOCA Temperature Transients Through Steel

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TABLE 12.2-37 DESIGN STRESS LIMITS FOR BIOLOGICAL SHIELD STRUCTURES

	Design Stress Limits For Reinforced Concrete		Design Stress Limits For Structural Steel	
STRUCTURAL SYSTEM AND DESIGN CONDITION CATEGORIES	**CRITERIA**	**STRESS LIMITS**	**CRITERIA**	**STRESS LIMITS**
1. Shield Structure Below Reactor Vessel Supports				
a) Normal Operation: Operating Loads (1)	ACI-318 (WSD)	Code Allowable	AISC (Part-1)	Code Allowable
b) Design Basis Accident: Operating Load + DBA + OBE	”	”	”	”
c) Safe Shut-Down: Operating Load + DBA + DBE	”	1.5x (Code Allowable)	”	1.5x (Code Allowable)
2. Shield Structure Above Reactor Vessel Supports				
a) Normal Operation: Operating Loads (1)	”	Code Allowable	”	Code Allowable
b) Design Basis Accident: Operating Load + DBA + OBE	”	0.75 f'_c (concrete) 0.9 Y.S. (reinforcement)	”	0.9 Y.S.
c) Safe Shut-Down: Operating Load + DBA + DBE	”	”	”	”

TABLE 12.2-37 DESIGN STRESS LIMITS FOR BIOLOGICAL SHIELD STRUCTURES

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NOTES:

1. Operating Load includes:
    a. Dead loads
    b. Live loads
    c. Neutron absorption
    d. Reactor vessel operating weight
    e. Residual construction loads
2. DBA Load includes:
    a. Internal pressure
    b. Pipe rupture reaction loads
    c. Safety injection water load

TABLE 12.2-38 ALLOWABLE AND CRITICAL STRESSES IN REACTOR BUILDING INTERIOR STRUCTURES

STRUCTURAL SYSTEM & DESIGN CONDITION CATEGORIES	LOADING CRITERIA	DESIGN STRESS LIMITS (psi) f′c	DESIGN STRESS LIMITS (psi) fs	ACTUAL MAX. STRESSES (psi) f′c	ACTUAL MAX. STRESSES (psi) fs
1. EQUIPMENT COMPARTMENTS					
a) Normal Operation	Operating Loads	1800	24000	200	5900
b) Design Basis Accident	Operating Loads + DBA + OBE	1800	24000	610	24000
c) Safe Shut-Down	Operating Loads + DBA + DBE	2700	36000	2200	32000
d) Pipe Rupture Reaction Loads & Jet Loads (Local Effect)	1. Operating Loads + DBA (Maximum Pipe Rupture Reaction Load) + DBE	3000	54000	2260	48430
	2. Operating Loads + DBA (Pipe Rupture Reaction Load & Internal Differential Pressure Load) + DBE	3000	54000	2890	53200

TABLE 12.2-38 ALLOWABLE AND CRITICAL STRESSES IN REACTOR BUILDING INTERIOR STRUCTURES

		DESIGN STRESS LIMITS (psi)		ACTUAL MAX. STRESSES (psi)	
2. STRUCTURAL EQUIPMENT SUPPORTS					
a) Normal Condition	Operating Loads	1500	30000 (*)	200	5850 (*)
b) Upset Condition	Operating Loads + OBE	1800	30000 (*)	500	19700 (*)
c) Emergency Condition	Operating Loads + DBE	2700	45000 (*)	2190	24700 (*)
d) Faulted Condition	Operating Loads + DBA:				
	1. Duration: > 0.01 Sec.	2700	45000 (*)	(1)	(1)
	2. Duration: ≤ 0.01 Sec.	3000	47500 (*)	(1)	(1)
	Operating Loads + DBA + DBE:				
	1. Duration: > 0.01 Sec.	3000	56000 (*)		
	2. Duration: ≤ 0.01 Sec.	3400	56000 (*)	2630	56000 (*)(3)
3. BIOLOGICAL SHIELD					
<u>Structure Below Reactor Vessel Supports</u>					
a) Normal Operation	Operating Loads	1800	24000	420	20900
b) Design Basis Accident	Operating Loads + DBA + OBE	1800	24000	500	21500
c) Safe Shutdown	Operating Loads + DBA + DBE	2700	36000	500	21500

TABLE 12.2-38 ALLOWABLE AND CRITICAL STRESSES IN REACTOR BUILDING INTERIOR STRUCTURES

			DESIGN STRESS LIMITS (psi)		ACTUAL MAX. STRESSES (psi)	
	Structure Above Reactor Vessel Supports					
	a) Normal Operation	Operating Loads	1800	24000	60	20900
	b) Design Basis Accident	Operating Loads + DBA + OBE	3400	54000	70	25400
	c) Safe Shut-Down	Operating Loads + DBA + DBE	3400	54000	70	25400
4.	FLOOR SYSTEMS (2)					
	Monolithic Concrete Beam and Slabs					
	a) Normal Operation	Operating Loads	1800	24000	1800	22600
	b) Design Basis Accident	Operating Loads + DBA + OBE	1800	24000	1800	23900
	c) Safe Shut-Down	Operating Loads + DBA + DBE	2700	36000	1900	25300
	Composite Floors (Concrete Slab on Steel Framing) (All Design Condition Categories)	1. Operating Loads + OBE	1800	24000	1570	22400
		2. Operating Loads + DBE	2700	36000	1660	23800
	Steel Grating on Steel Framing (All Design Condition Categories)	1. Operating Loads + OBE	N.A.	22000 (*)	N.A.	20500 (*)
		2. Operating Loads + DBE	N.A.	33000 (*)	N.A.	25000 (*)

NOTES:

fs (without asterisk) = Tensile Stress in reinforcing Steel
fs (*) = Tensile Stress in Structural Steel

(1) Stresses not calculated

(2) Concrete stresses are based on Working Stress Design methods. For Ultimate Strength Design requirements, refer to USAR Section 12.2.15.

(3) Historical calculated stress from original support design, based upon postulated ruptures in primary Reactor Coolant Loop piping. Current support stress levels are less based upon postulated ruptures in limiting RCS Branch Lines. Note that reported stresses are the result of modifying the RPV supports as free-standing structures. Actual RPV supports are embedded in concrete and concrete will share bending stresses.

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TABLE 12.2-39 PROBABILITY OF DAMAGE TO CATEGORY I BUILDINGS AND SPENT FUEL POOL ROOF

INSPECTION INTERVAL IN YEARS	PROBABILITY (10^{-7}) OF DAMAGE TO TARGETS 1 THRU 4**		PROBABILITY (10^{-7}) OF DAMAGE TO TARGET 5**		
	UNIT 1	UNIT 2	UNIT 1	UNIT 2	TOTAL
1	*	*	*	*	*
2	*	*	*	*	*
3	0.003	0.001	*	*	*
4	0.024	0.011	*	*	*
5	0.116	0.050	0.001	0.001	0.002
10	8.967	3.096	0.050	0.049	0.099

*Probabilities less than 10^{-10}

** Target 1: 1′-6″ thick concrete wall at G row.

Target 2: 1′-0″ thick concrete roof of the Auxiliary Building.

Target 3: 2′-6″ thick concrete wall of the Shield Building.

Target 4: 2′-0″ thick concrete roof of the Shield Building.

Target 5: 1′-0″ thick concrete roof of the Spent Fuel Pool.

01514406

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TABLE 12.2-40 LOADS HANDLED OVER SAFETY RELATED COMPONENTS, COMPONENTS REQUIRED FOR PLANT SHUTDOWN OR DECAY HEAT REMOVAL

SYSTEM	LOAD(S) HANDLED	WEIGHT	DESIGNATED LIFTING DEVICE
Turbine Building Crane	1) HP Cover	81,700 lbs	Load Spreader & Slings
	2) LP#1 Outer Casing	122,300 lbs	Load Spreader & Slings
	3) LP#2 Outer Casing	122,300 lbs	Load Spreader & Slings
	4) LP#1 Inner Cyl. #1	50,000 lbs	Load Spreader & Slings
	5) LP#2 Inner Cyl. #1	50,000 lbs	Load Spreader & Slings
	6) LP#1 Inner Cyl. #2	90,000 lbs	Load Spreader & Slings
	7) LP#2 Inner Cyl. #2	90,000 lbs	Load Spreader & Slings
	8) HP Rotor	70,000 lbs	Load Spreader & Slings
	9) LP Rotor	160,000 lbs	Load Spreader & Slings
	10) Condensate Pump &	18,600 lbs (pump)	Slings
	11) Motor	12,000 lbs (motor)	Slings
	12) Vertical Cooling Water	6,400 lbs (motor)	Slings
	13) Pumps Smaller Turbine Parts Valves, etc.	14,670 lbs (pump) Various	Slings
	14) Spare Rotor Stands	12,500 lbs	Slings
	15) Generator Rotor (Unit 1)	246,000 lbs	Load Spreader & Slings
	16) Generator Rotor (Unit 2)	255,740 lbs	Load Spreader & Slings
	17) Load Block	7,000 lbs	NA
Auxiliary Building Crane	1) New Fuel Shipping Containers	6,600 lbs	Slings
	2) Heat Exchanger Removal Hatches	16,600 lbs	Slings
	3) Heat Exchanger Bundles	1,100 lbs (seal water) 1,900 lbs (letdown)	Slings
	4) Load Block	7,000 lbs	NA
	5) TN-40 and TN-40HT Spent Fuel Storage Cask	250,000 lbs (max load on hook)	TN-40 Cask Lift Beam

01435983

TABLE 12.2-40 LOADS HANDLED OVER SAFETY RELATED COMPONENTS, COMPONENTS REQUIRED FOR PLANT SHUTDOWN OR DECAY HEAT REMOVAL

SYSTEM		LOAD(S) HANDLED	WEIGHT		DESIGNATED LIFTING DEVICE
Containment Polar Cranes	1)	Pressurizer Missile Shields	40,500 lbs		Slings
	2)	Vessel Head*	187,000 lbs		Vessel Head Spreader Assembly and Vessel Head Lifting Device
	3)	Upper & Lower Internals	57,900 lbs 171,500 lbs	(upper) (lower)	Internals Lifting Rig
	4)	Vessel Studs (In handling box)	7,200 lbs		Slings
	5)	In-Service Insp. Tool	4,000 lbs		Slings
	6)	Reactor Coolant Pump			
		a) Motor	79,500 lbs		Slings
		b) Pump	55,200 lbs		Slings
		c) Fly Wheel	13,200 lbs		Slings
	7)	Load Blocks	15,000 lbs		NA
Spent Fuel Pool Crane	1)	Divider Gates	2,620 lbs		Slings
	2)	Pool Covers	4,550 lbs		Slings

* The lifted weight includes the weight of the vessel head plus additional equipment such as CRDMs, RPIs, radiation shield, head assembly upgrade package, lift rig, stud tensioner hoist, and additional weight for contingency.

TABLE 12.2-41 STRUCTURAL CRITERIA – D5/D6 BUILDING

05-049

Loading Condition	Reinforced Concrete	Structural Steel
1. Dead loads plus live loads plus wind load or seismic load (0.06g)	ACI 349-85 Allowable Values	AISC Allowable Values
2. Dead loads plus live loads plus seismic load (0.12g)	ACI 349-85 Allowable Values	1.6 Times AISC Allowable Values
3. Dead loads plus live loads plus R.G. 1.76 design tornado	ACI 349-85 Allowable Values	1.7 Times AISC Allowable Values

For further information on the Ultimate Strength Design methodology used to design the D5/D6 Building, such as load factors applied to various loading conditions, see Reference 43 and USAR Section 12.2.15. 05-049

TABLE 12.2-42 D5/D6 BUILDING DAMPING FACTORS

(Percent of Critical Damping)

Structure or Component	Operating Basis Earthquake or 1/2 Safe Shutdown Earthquake *	Safe Shutdown Earthquake
Equipment and large-diameter piping systems, piping diameter greater than 12 inches **	2	3
Small-diameter piping systems, diameter equal to or less than 12 inches	1	2
Welded steel structures	2	4
Bolted steel structures	4	7
Prestressed concrete structures	2	5
Reinforced concrete structures	4	7

93001

* In the dynamic analysis of active components, these values should also be used for SSE.

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

TABLE 12.2-43 D5D6 BUILDING TORNADO GENERATED MISSILES

Missiles	Dimension (meters)	Mass (Kilograms)	Velocity* (Meters/Sec)
Wood Plank	0.092 x 0.289 x 3.66	52	83
6 inch Sch 40 Pipe	0.168 Diameter x 4.58	130	52
1 inch Steel Rod	0.0254 Diameter x 0.915	4	51
Utility Pole	0.343 Diameter x 10.68	510	55
12 inch Sch 40 Pipe	0.32 Diameter x 4.58	340	47
Automobile	5 x 2 x 1.3	1810	59

98124

* Velocities are horizontal velocities. For vertical velocities, 70 percent of the horizontal velocities shall be used. 98124

TABLE 12.3-1 PRIMARY SHIELD NEUTRON FLUXES AND DESIGN PARAMETERS

Calculated Neutron Fluxes - Core Midplane

Energy Group	Incident Fluxes ($n/cm^2/sec$)	Leakage Fluxes ($n/cm^2/sec$)	
E > 1 Mev	2.94×10^9	1.0×10^2	98124
5.3 Kev < E ≤ 1 Mev	3.23×10^{10}	2.5×10^2	
.625 ev < E ≤ 5.3 Kev	1.91×10^{10}	5.0×10^2	
E ≤ .625 ev	4.45×10^9	2.5×10^4	98124

Design Parameters

Core thermal power, MWt	1650	98124
Active core height, inches	144	
Effective core diameter, inches	96.95	
Baffle wall thickness, inches	1.125	
Barrel wall thickness, inches	1.75	
Thermal shield wall thickness, inches	3.50	
Reactor vessel I.D., inches	132.00	
Reactor vessel wall thickness, inches	6.50	98124
Reactor coolant cold leg temperature, °F	544	
Reactor coolant hot leg temperature, °F	607	
Maximum thermal neutron flux exiting primary concrete	3×10^4 n/cm^2-sec	98124
Reactor shutdown dose exiting primary concrete	< 15 mr/hr	

TABLE 12.3-2 SECONDARY SHIELD DESIGN PARAMETERS

Core Thermal Power	1721.4
Reactor coolant liquid volume	6191 ft^3
Reactor coolant transit times:	
Core	0.9 sec.
Core exit to steam generator inlet	2.0 sec.
Steam generator inlet channel	0.6 sec.
Steam generator tubes to vessel inlet	5.3 sec.
Vessel inlet core	2.1 sec.
Total out of core	10.0 sec.
Full power dose rate outside secondary shielding	< 1 mr/hr

TABLE 12.3-3 ACCIDENT SHIELD DESIGN PARAMETERS

Core thermal power	1721.4 MWt
Minimum full power operating time	650 days
Equivalent fraction of core cooling	1.0
Fission product fractional releases:	
Noble gases	1.0
Halogens	0.5
Remaining fission product inventory	0.01

30-Day Dose to Control Room Personnel

	With Spray (Rem)	No Spray (Rem)	Limits
Thyroid	8.78	74.1	30
Whole Body	1.6	1.6	5
Beta Skin	26.7	26.7	30

98109

TABLE 12.3-4 REFUELING SHIELD DESIGN PARAMETERS

Total number of fuel assemblies	121
Minimum full power exposure	1000 days
Minimum time between shutdown and fuel handling	24 hours
Maximum dose rate adjacent to spent fuel pit	1.0 mRem/hr
Maximum dose rate at water surface	2.5 mRem/hr

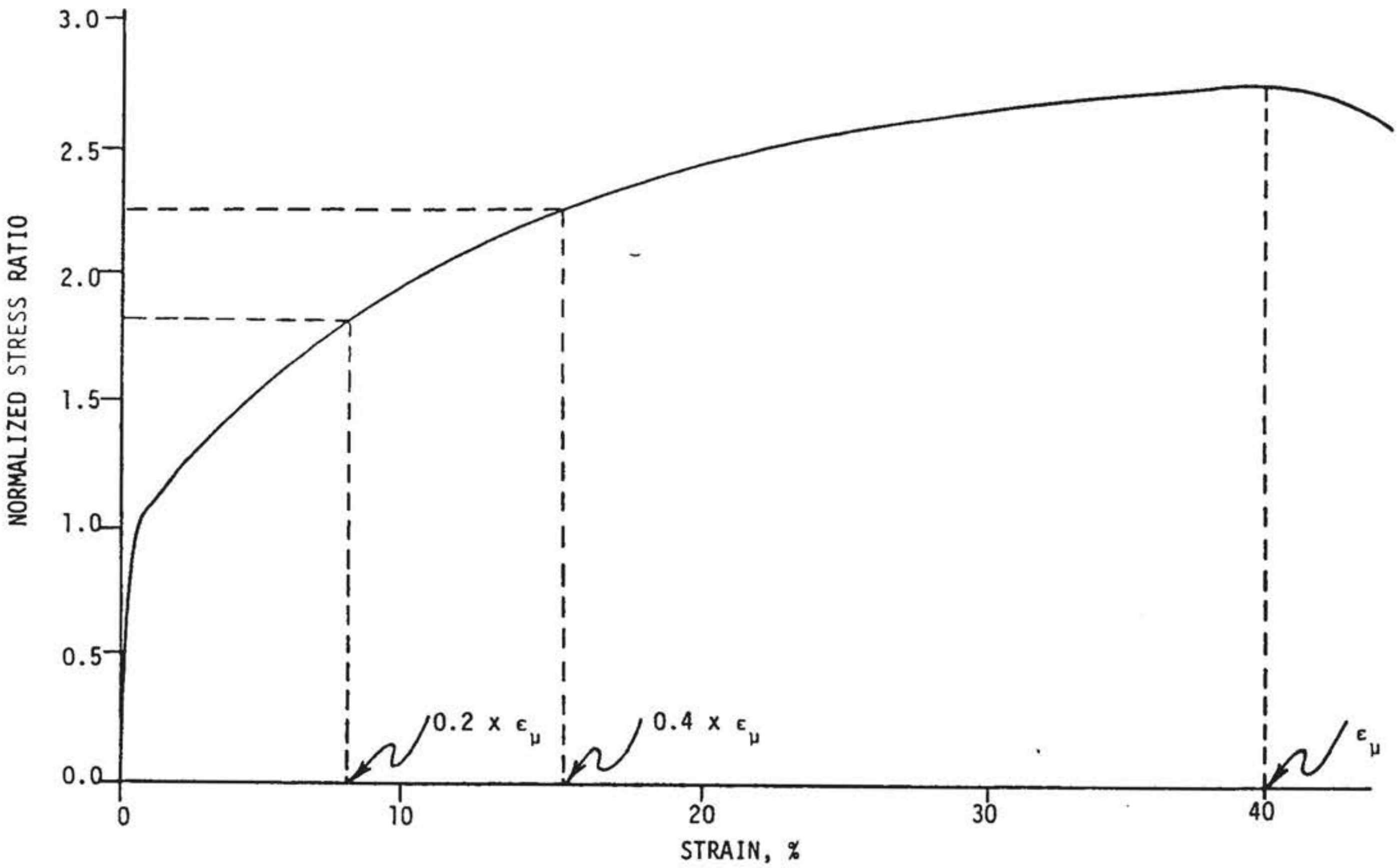
01-024

TABLE 12.3-5 PRINCIPAL AUXILIARY SHIELDING

Component	Concrete Shield Thickness, Ft. - In.
Demineralizers	3 - 9
Charging pumps	2 - 0
Liquid holdup tanks	2 - 9
Volume control tank	3 - 6
Reactor Coolant filter	2 - 6
Gas stripper	2 - 0
Gas decay tanks	4 - 0
Gas compressor	3 - 0
Waste evaporator	2 - 6
Liquid waste holdup tank	2 - 0
Safety Injection Pumps	1 - 4
Design parameters for the auxiliary shielding include:	

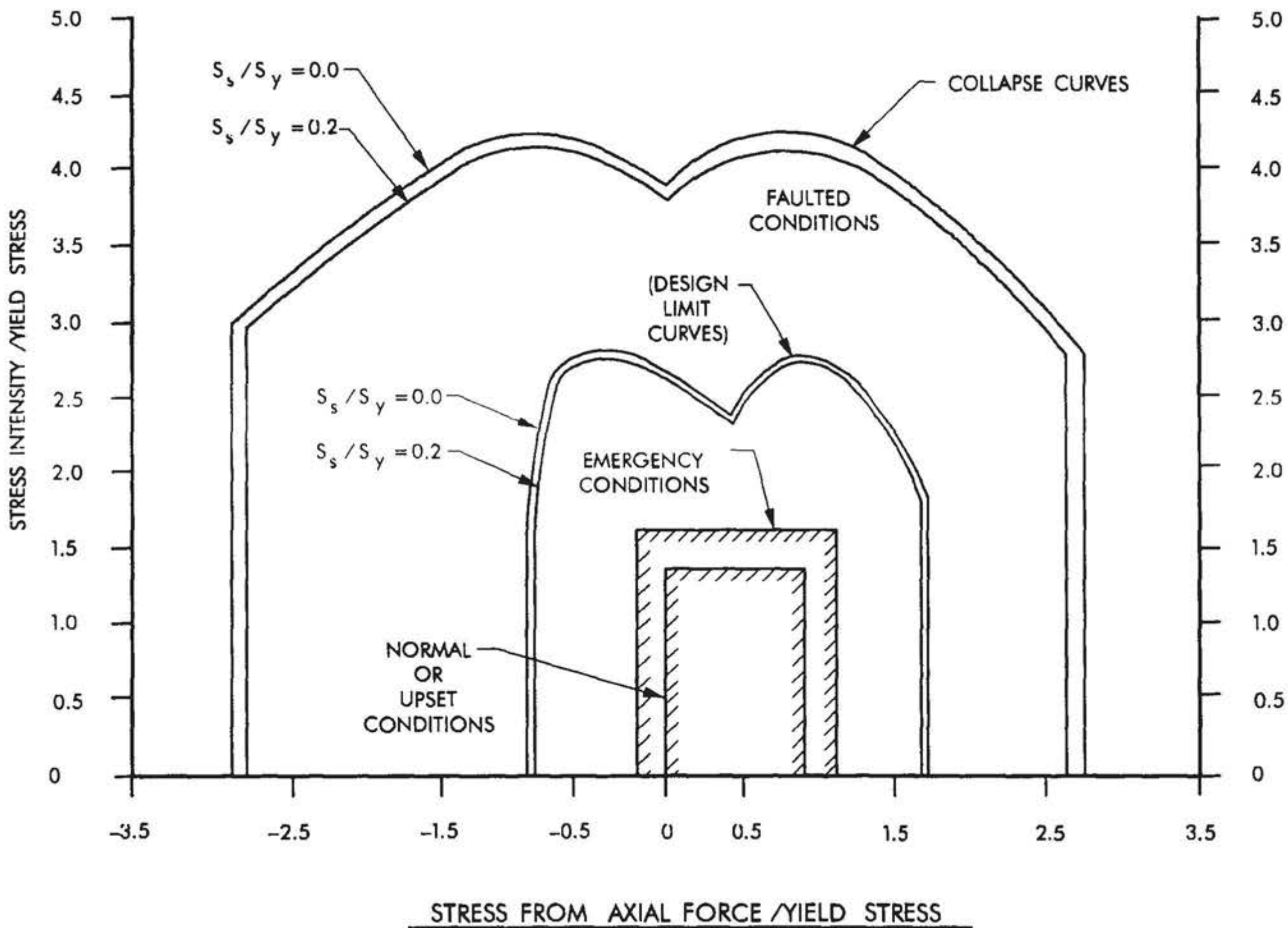
01-024

Core Thermal Power	1721.4 MWt
Fraction of fuel rods containing small clad effects	0.01
Reactor coolant liquid volume	6191 ft^3
Letdown flow (normal purification)	40 gpm
Effective cesium purification flow (intermittent)	4.0 gpm
Cut-in concentration deborating demineralizer	160 ppm
Dose rate outside auxiliary building	< 1 mRem/hr
Dose rate in the building outside Shield walls	< 2.5 mRem/hr



TYPICAL STRESS STRAIN CURVE STANDARD ASTM TENSILE TEST MATERIAL: 304 STAINLESS STEEL TEMPERATURE 600° F

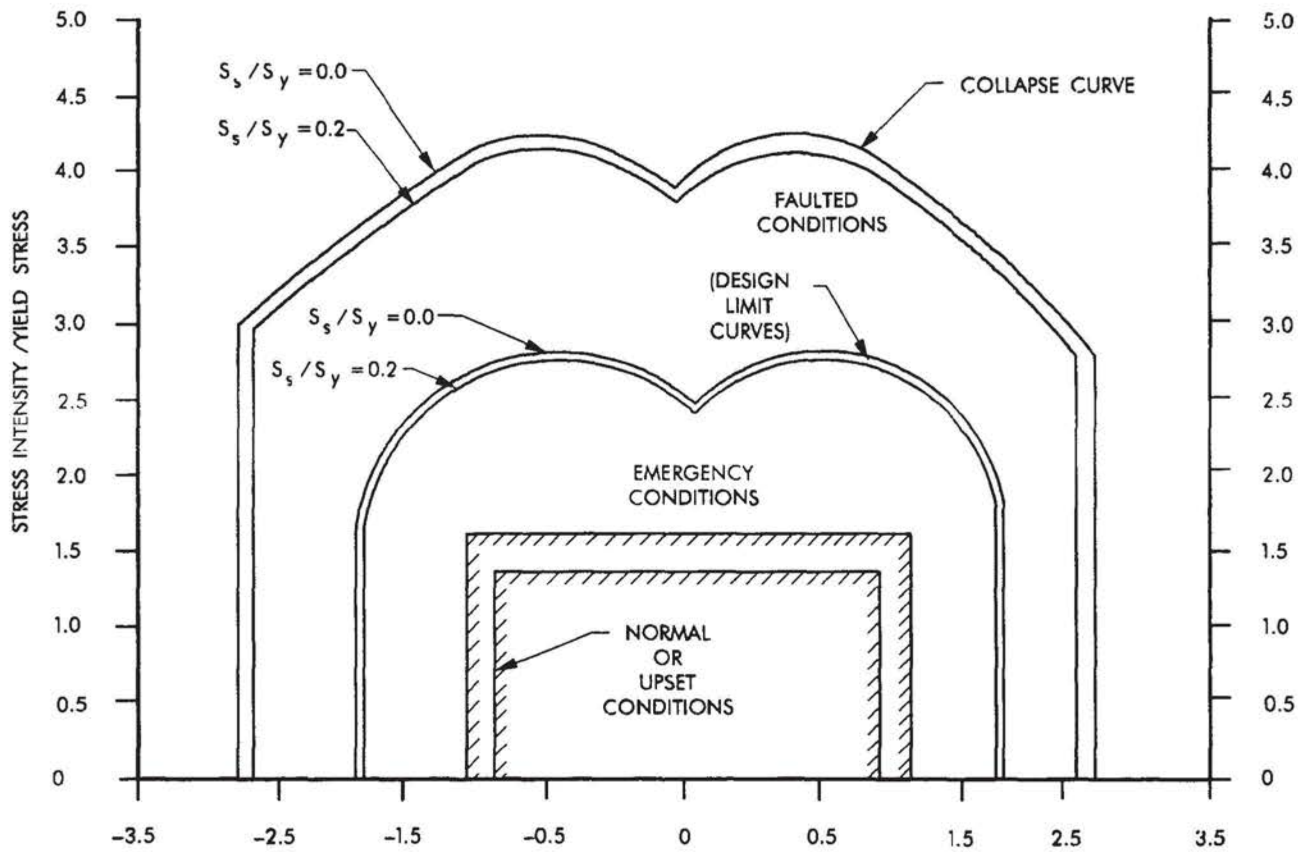
DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12201.DGN		FIGURE 12.2-1 REV. 18



MATERIAL: SA 376 Tp 316
CROSS-SECTION: HOLLOW-CIRCULAR
HOOP STRESS: 0.90 S_y

COMPARISON BETWEEN DESIGN AND COLLAPSE CONDITIONS

DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U1222A.DGN		**FIGURE 12.2-2A REV. 18**

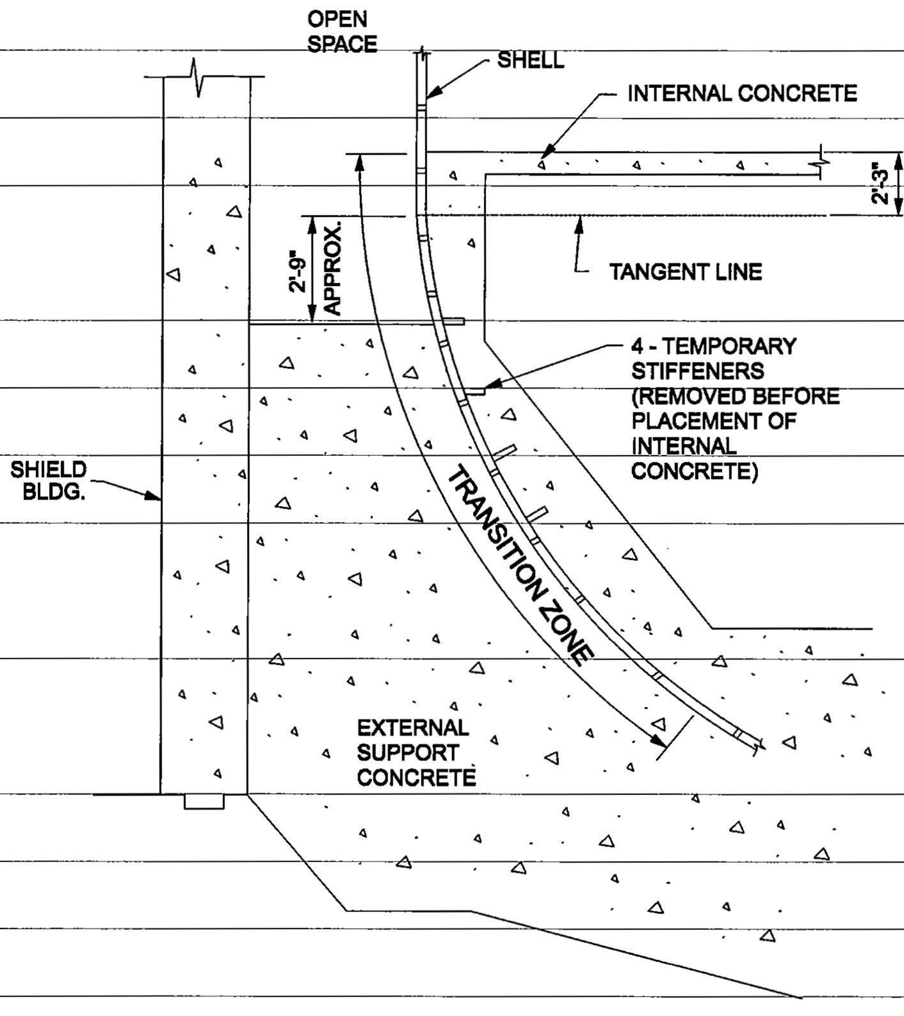


STRESS FROM AXIAL FORCE /YIELD STRESS

MATERIAL: SA 376 Tp 316
CROSS-SECTION: HOLLOW-CIRCULAR
HOOP STRESS: 0.00 S_y

COMPARISON BETWEEN DESIGN AND COLLAPSE CONDITIONS

DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12202B.DGN		**FIGURE 12.2-2B REV. 18**



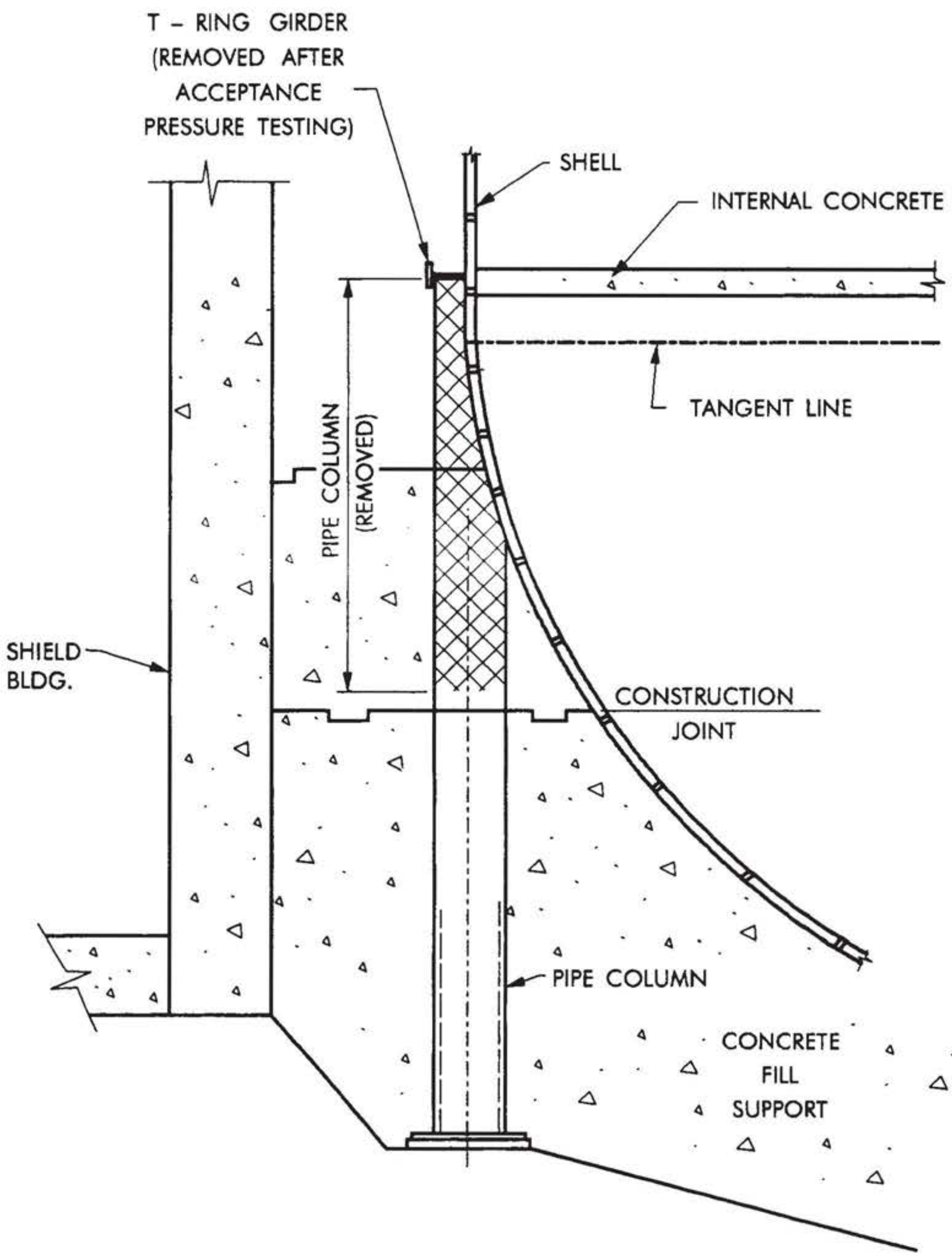
SHELL TRANSITION ZONE AND INTERNAL STIFFENERS

DRAWN BY: VLS	DATE: 02-02-07	NORTHERN STATES POWER COMPANY Xcel Energy PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING, MINNESOTA	SCALE: NA	REVISION: 29
PAGE. NO. NA	CAD-FILE U12203.DGN		FIGURE 12.2-3	

CAD FILE: J / CAD / PRI / USARS / U12203.DGN

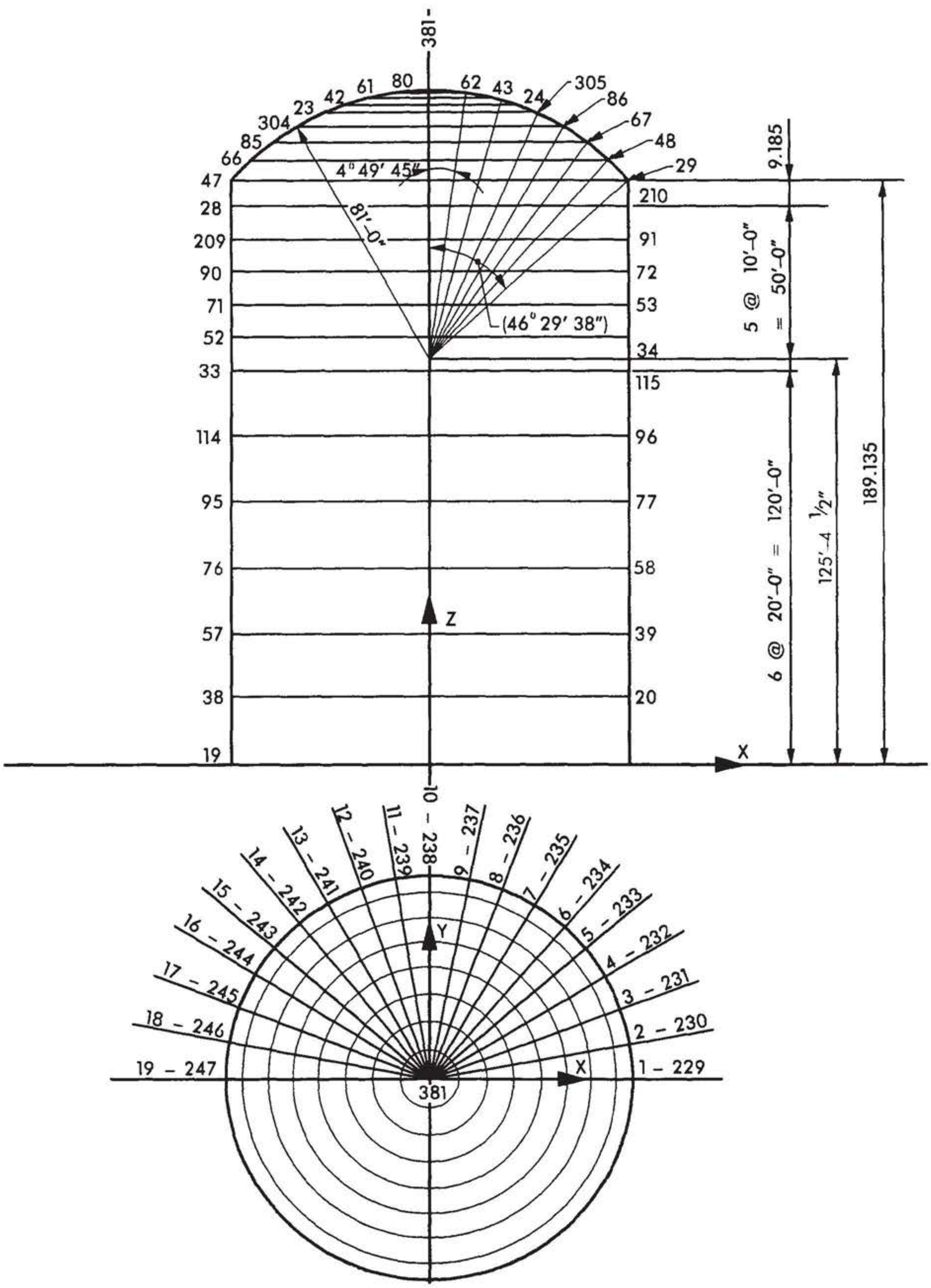
01069064

FIGURE 12.2-3



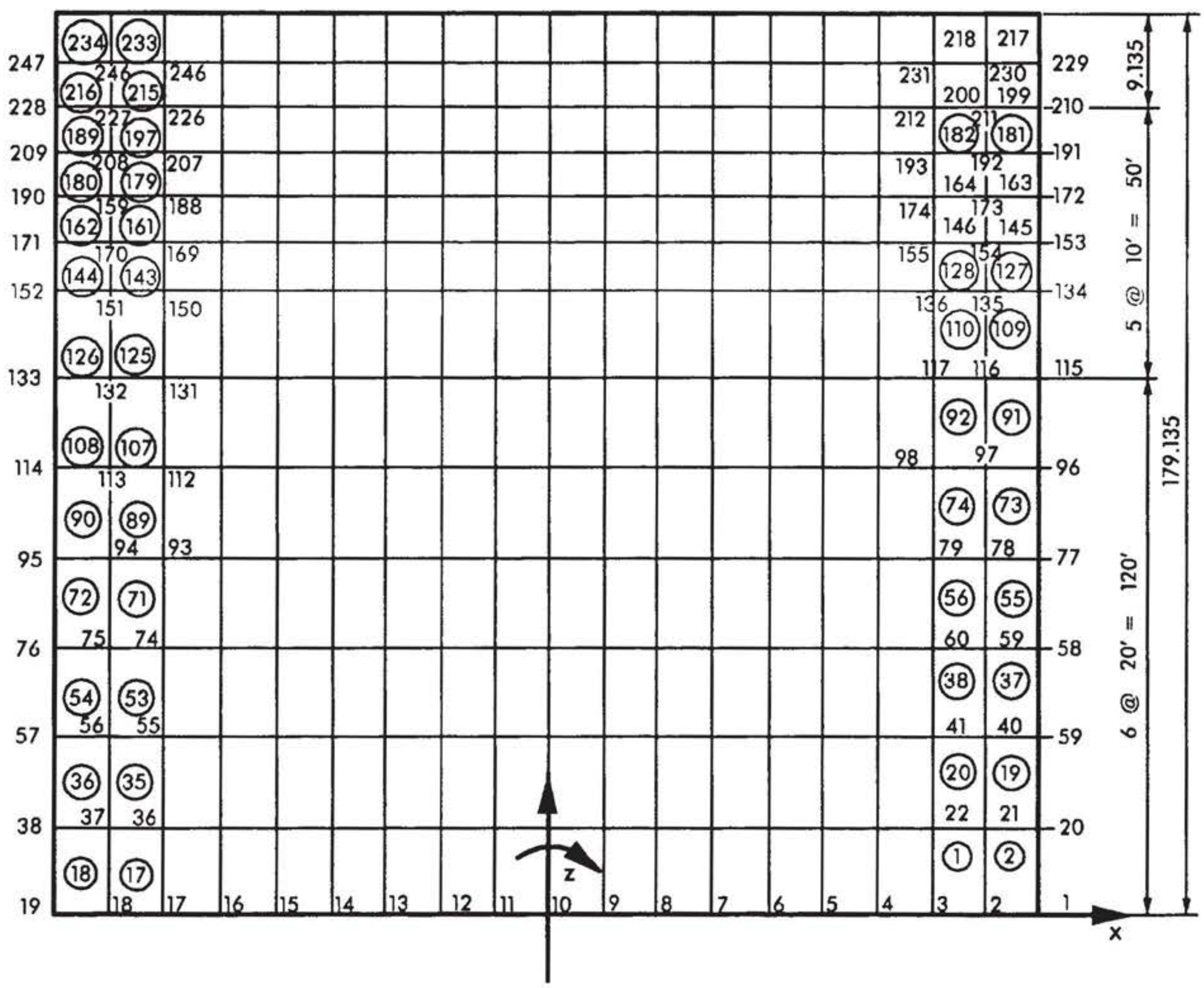
TEMPORARY SUPPORT STRUCTURE

DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12204.DGN		**FIGURE 12.2-4 REV. 18**



MODEL FOR FINITE ELEMENT ANALYSIS OF DOME

DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12206.DGN		**FIGURE 12.2-6 REV. 18**



MODEL FOR FINITE ELEMENT ANALYSIS OF DOME

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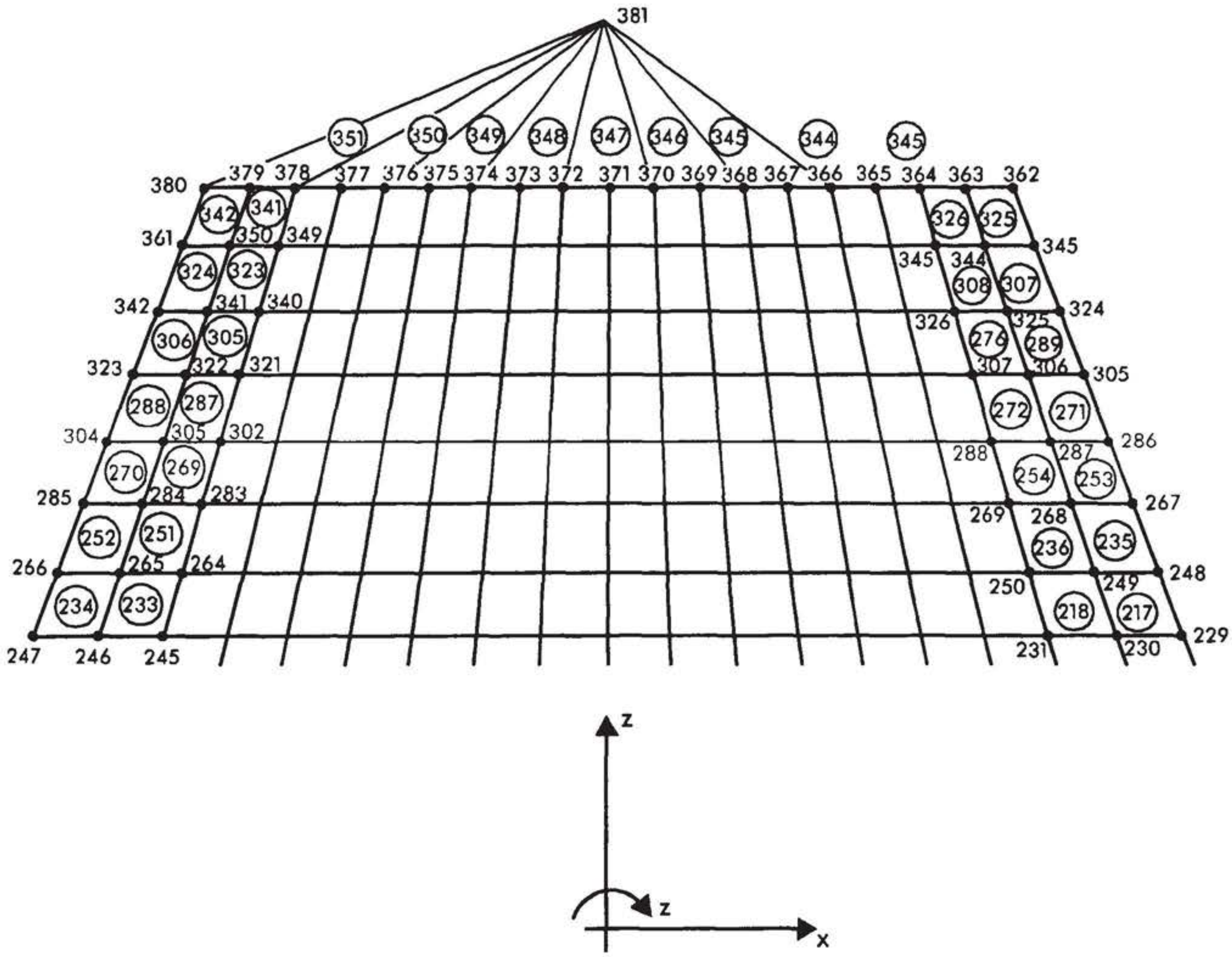
CHECKED	CAD FILE U1227.DGN		**FIGURE 12.2-7**	**REV. 18**



MODEL FOR FINITE ELEMENT ANALYSIS OF DOME

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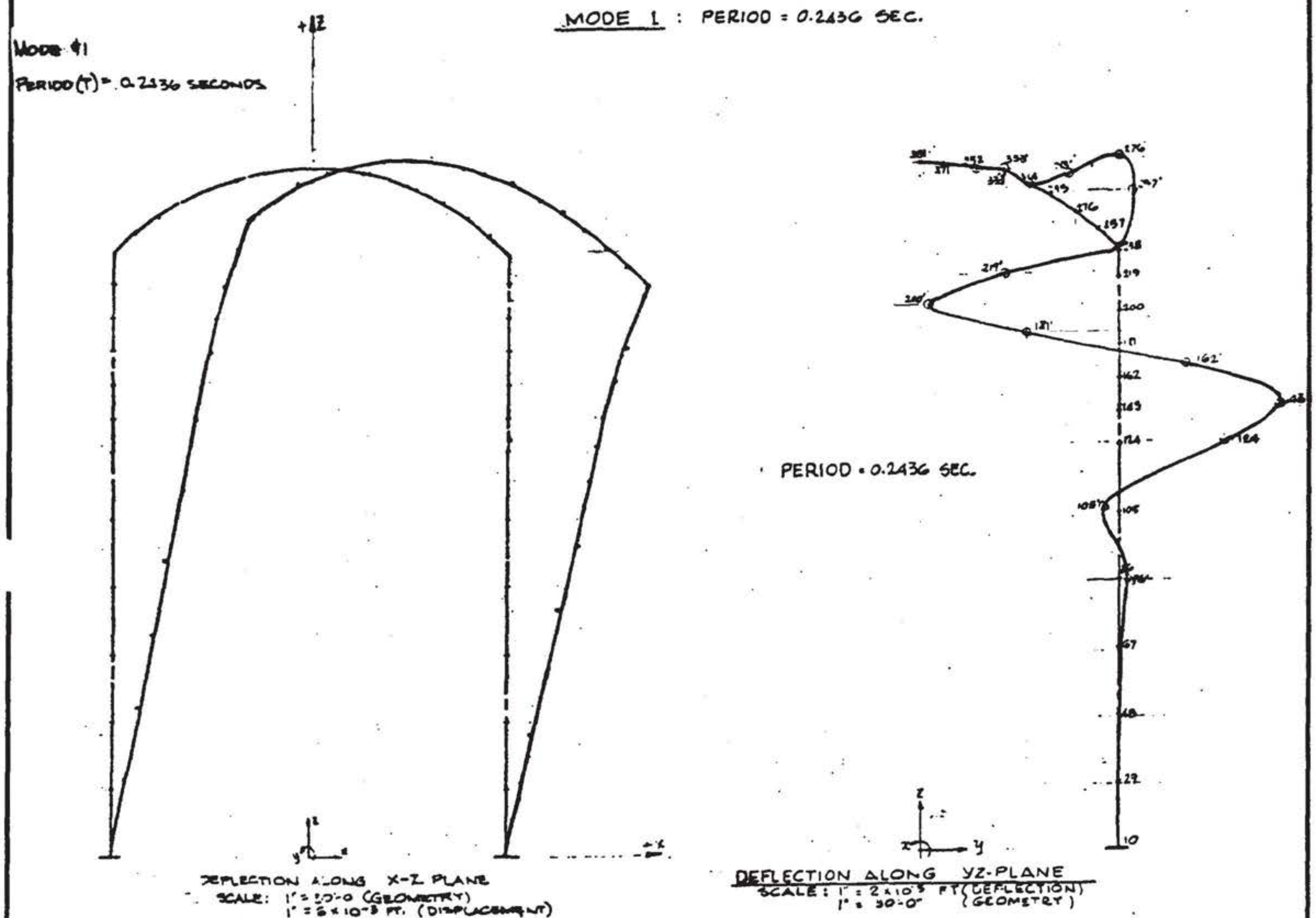
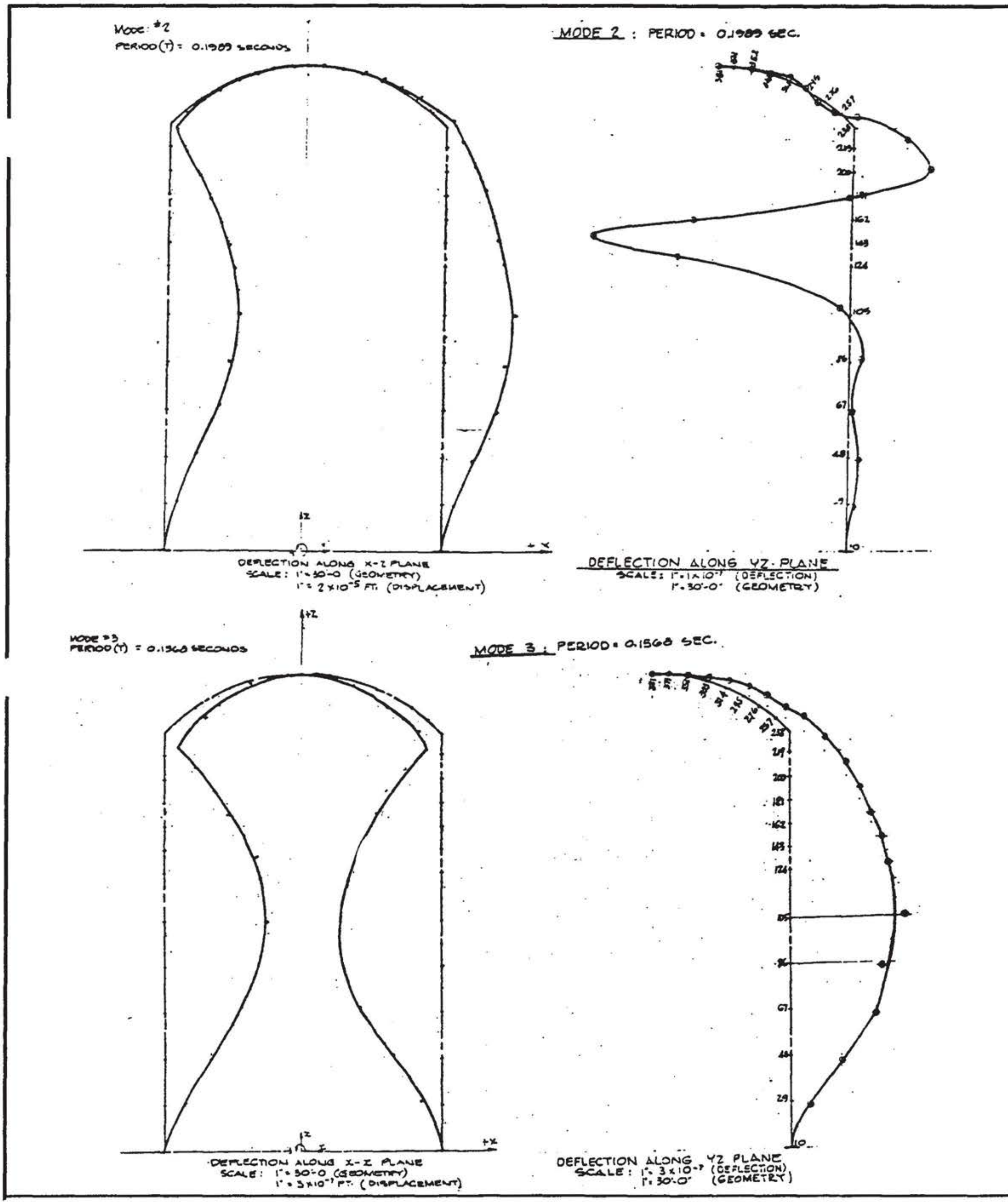
CHECKED	CAD FILE U12208.DGN	PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	**FIGURE 12.2-8**	**REV. 18**



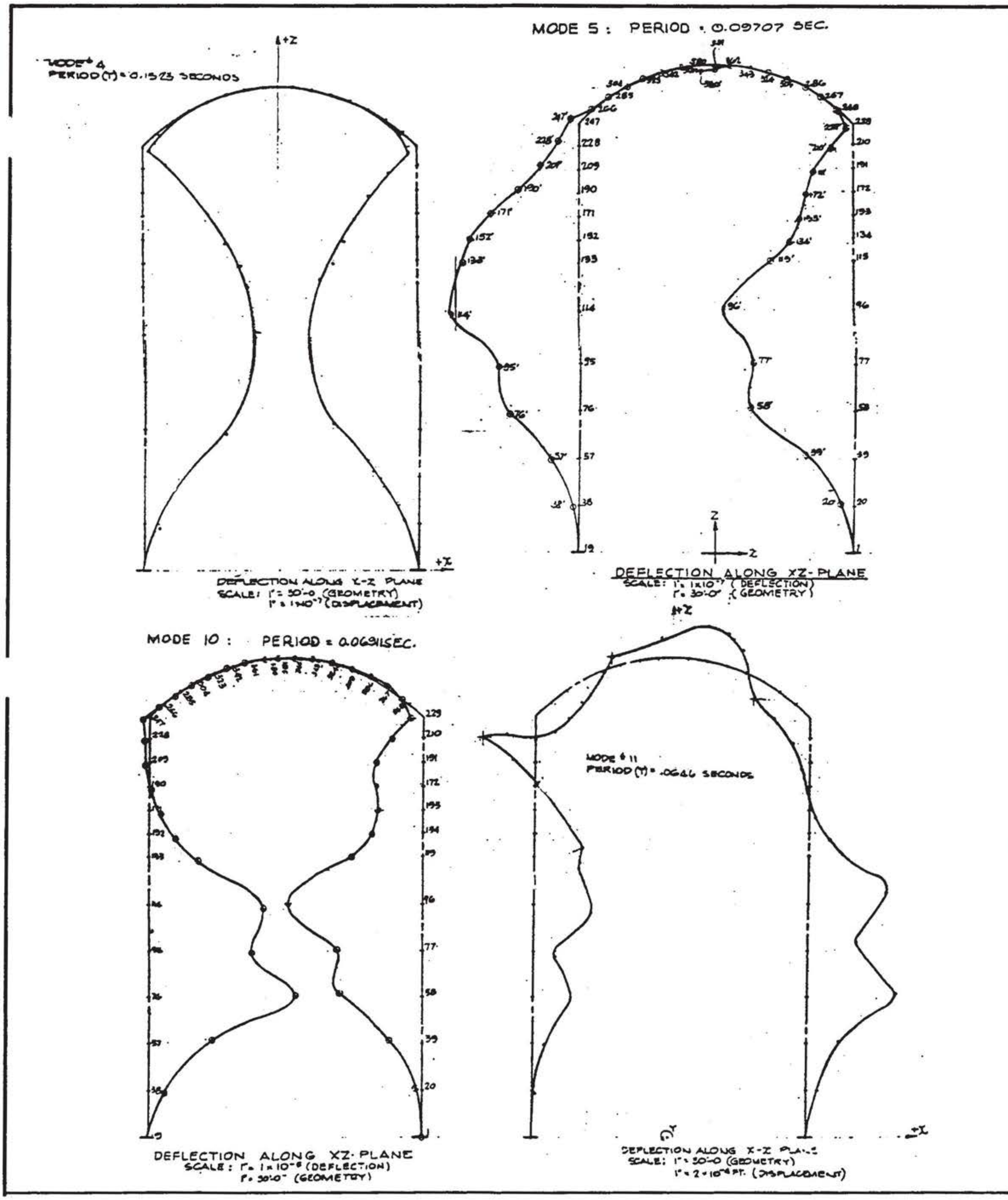
ANALYSIS OF DOME USING DYNAMIC VERSION OF SAP

DWN T. MILLER	DATE 6-23-89	NORTHERN STATES POWER COMPANY	SCALE: NONE
CHECKED	CAD FILE U1229A.DGN	PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	FIGURE 12.2-9A REV. 18



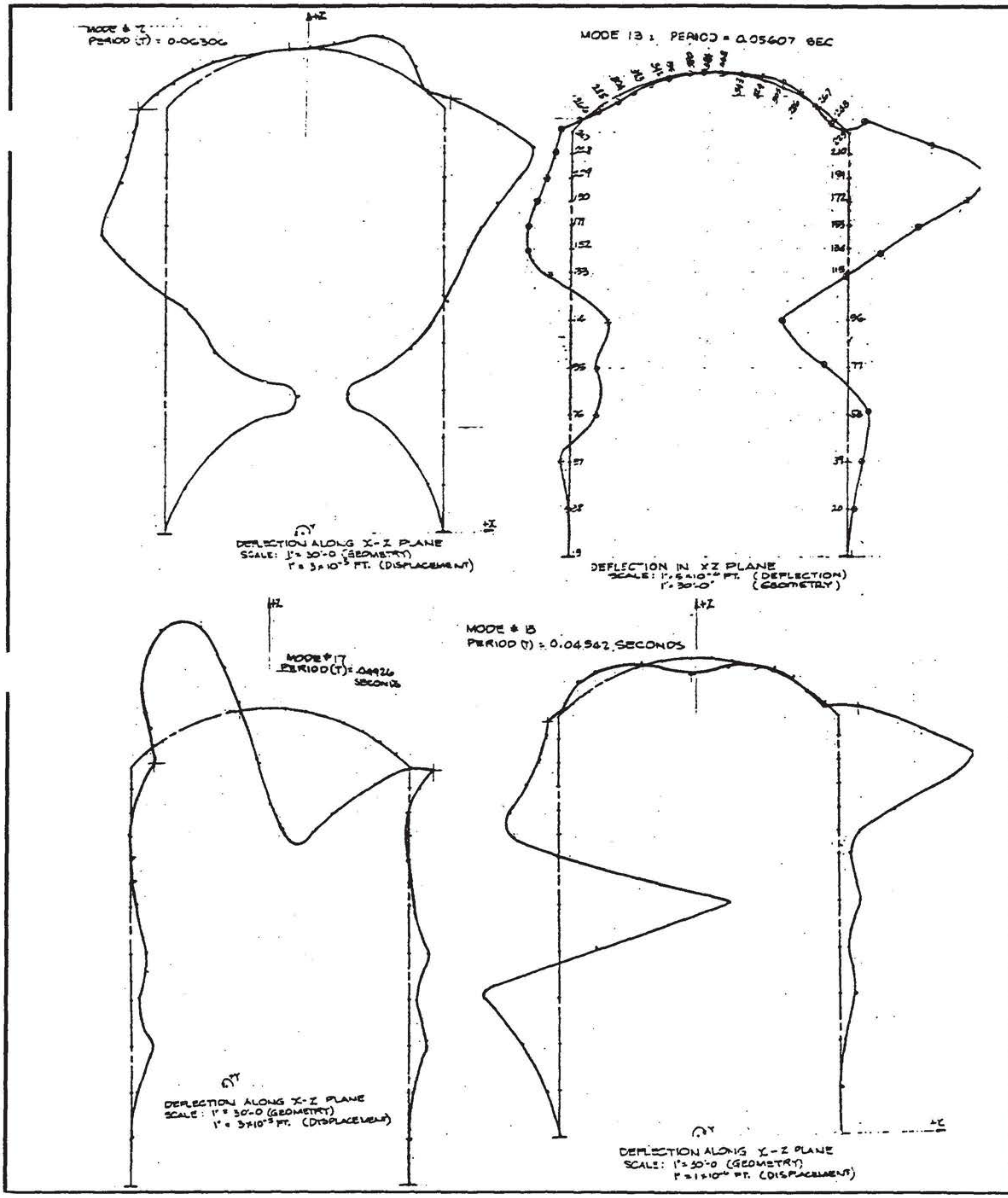
ANALYSIS OF DOME USING DYNAMIC VERSION OF SAP

DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12298.DGN		**FIGURE 12.2-9B REV. 18**



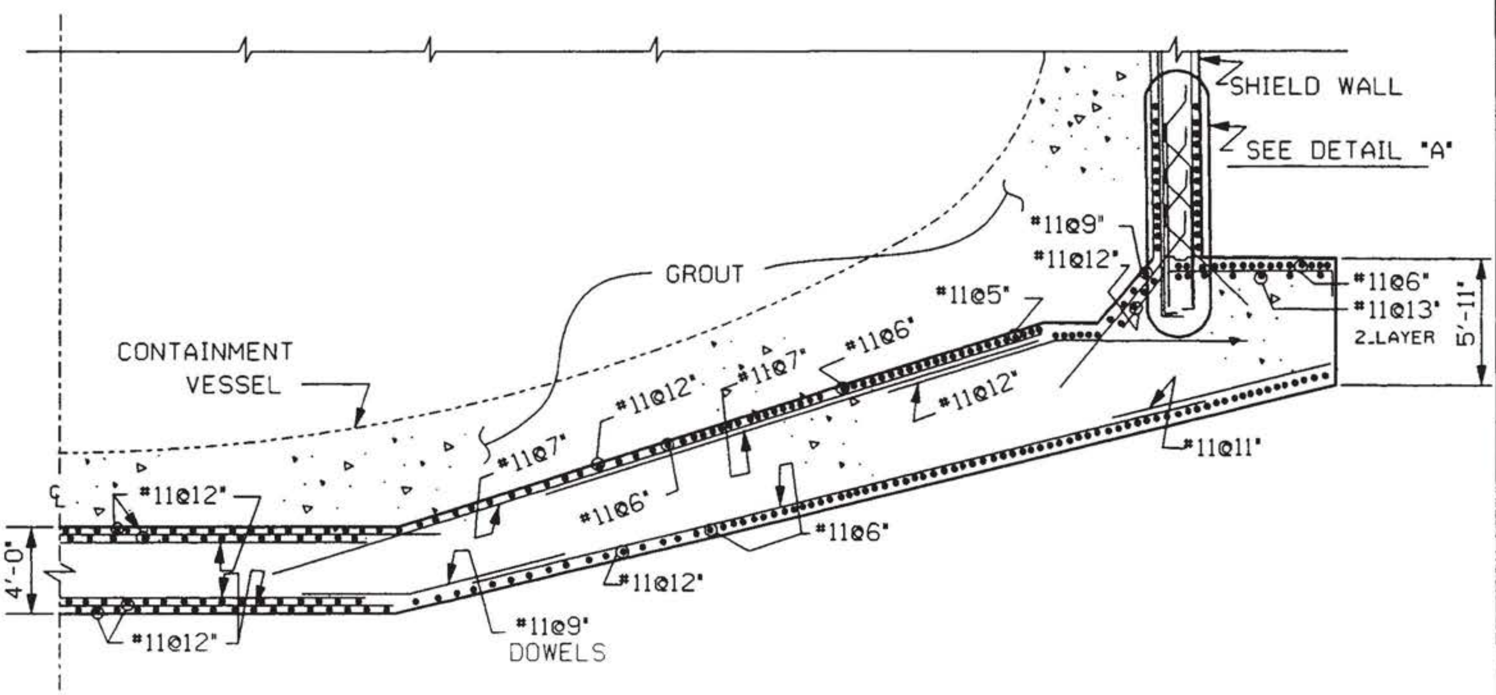
ANALYSIS OF DOME USING DYNAMIC VERSION OF SAP

DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U1229C.DGN		FIGURE 12.2-9C REV. 18

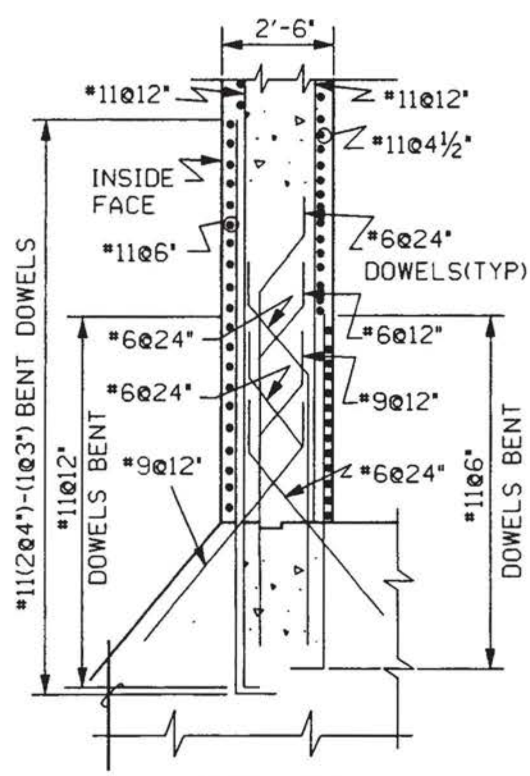


ANALYSIS OF DOME USING DYNAMIC VERSION OF SAP

DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12290.DGN		**FIGURE 12.2-9D REV. 18**



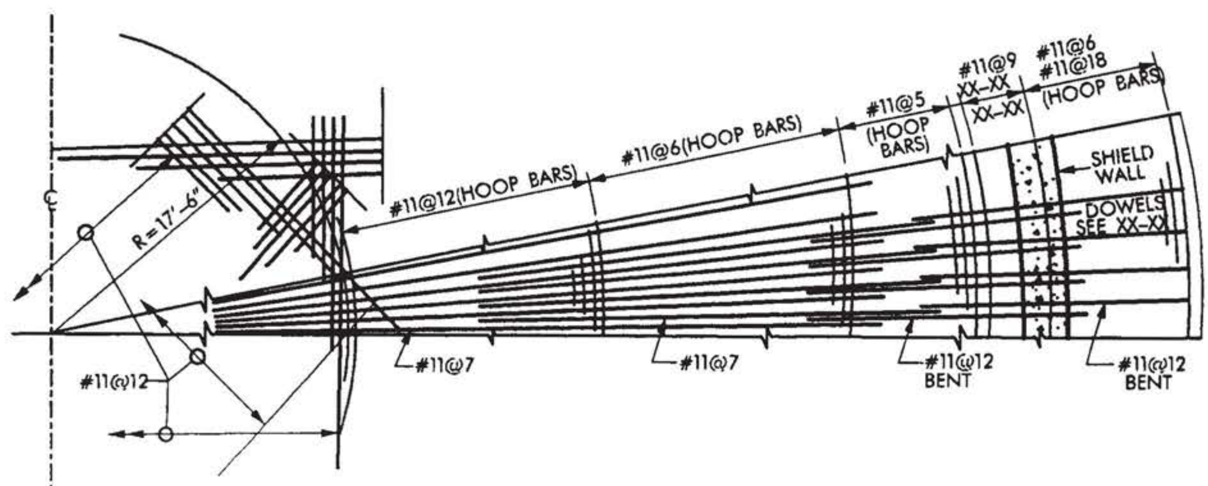
TYPICAL SECTION FOR BASE SLAB & WALL



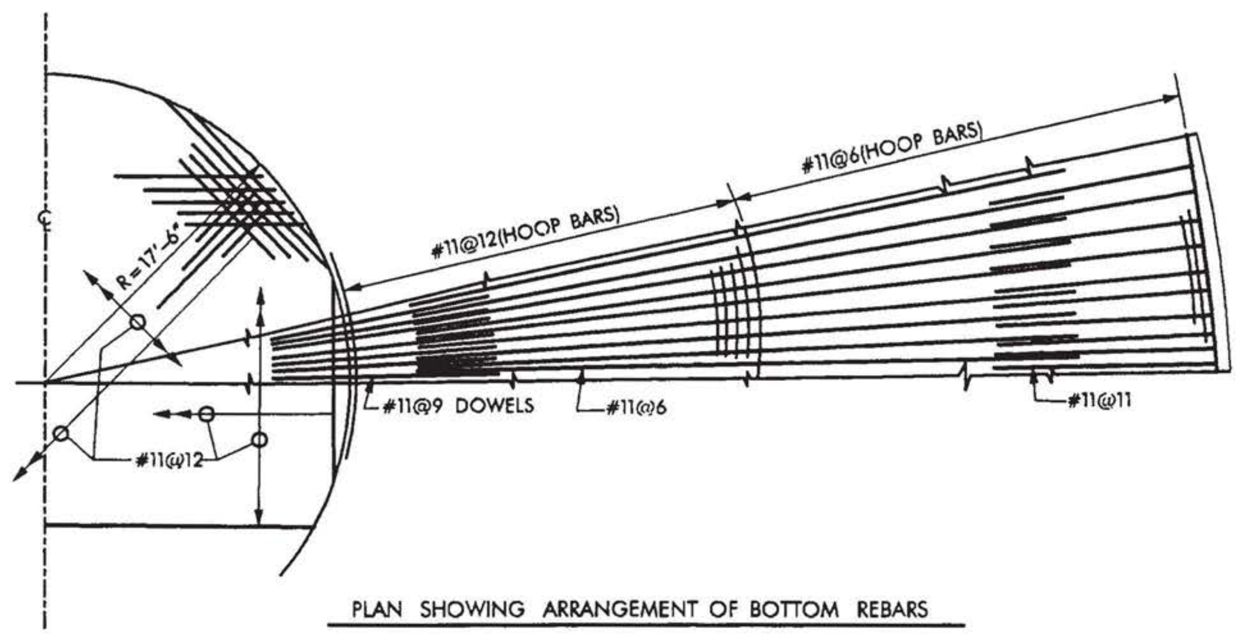
DETAIL "A"

SECTION OF BASE SLAB & WALL REINFORCEMENT DETAILS

DWN TAM	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12210.DGN		**FIGURE 12.2-10 REV. 18**



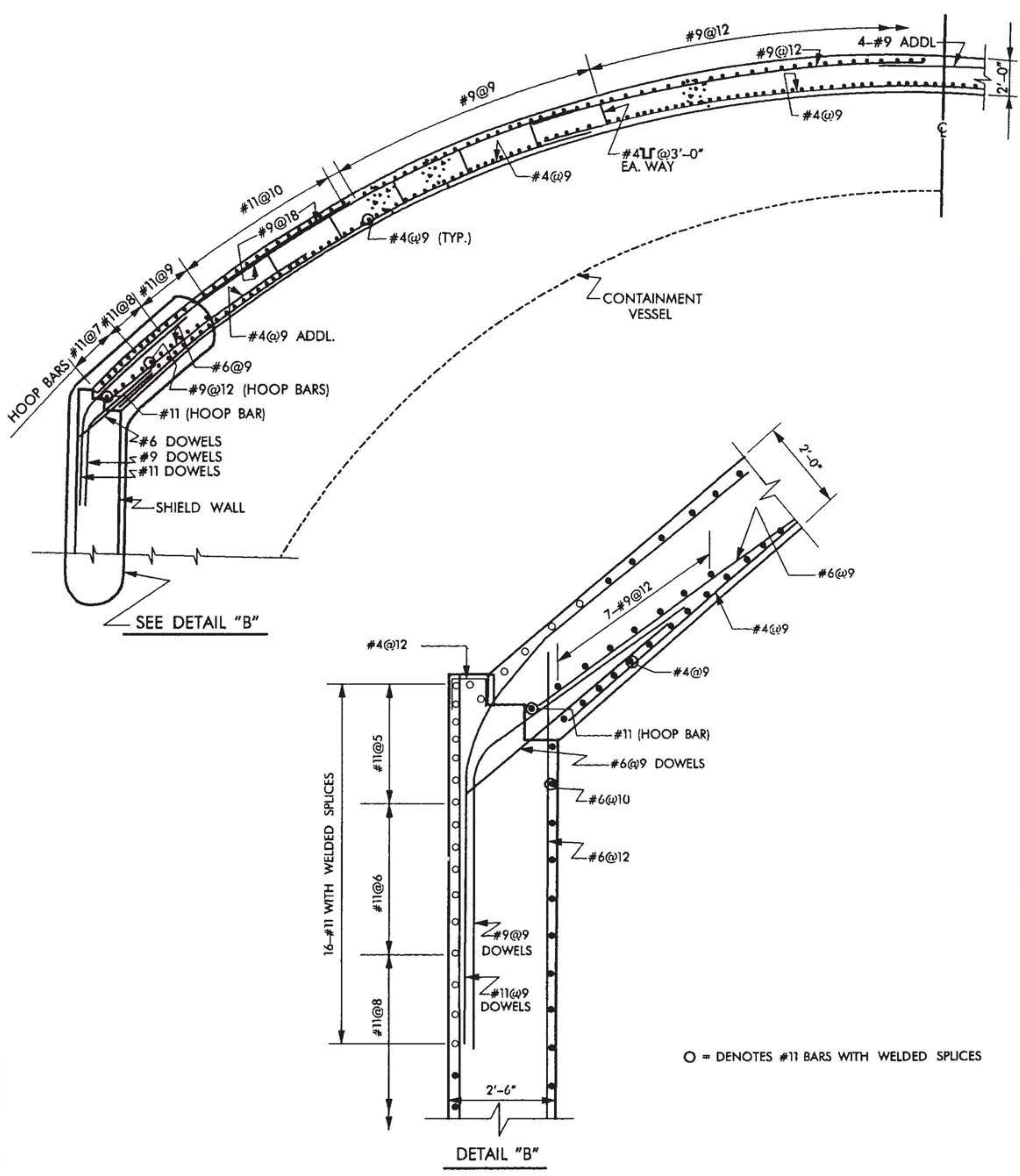
PLAN SHOWING ARRANGEMENT OF TOP REBARS



PLAN SHOWING ARRANGEMENT OF BOTTOM REBARS

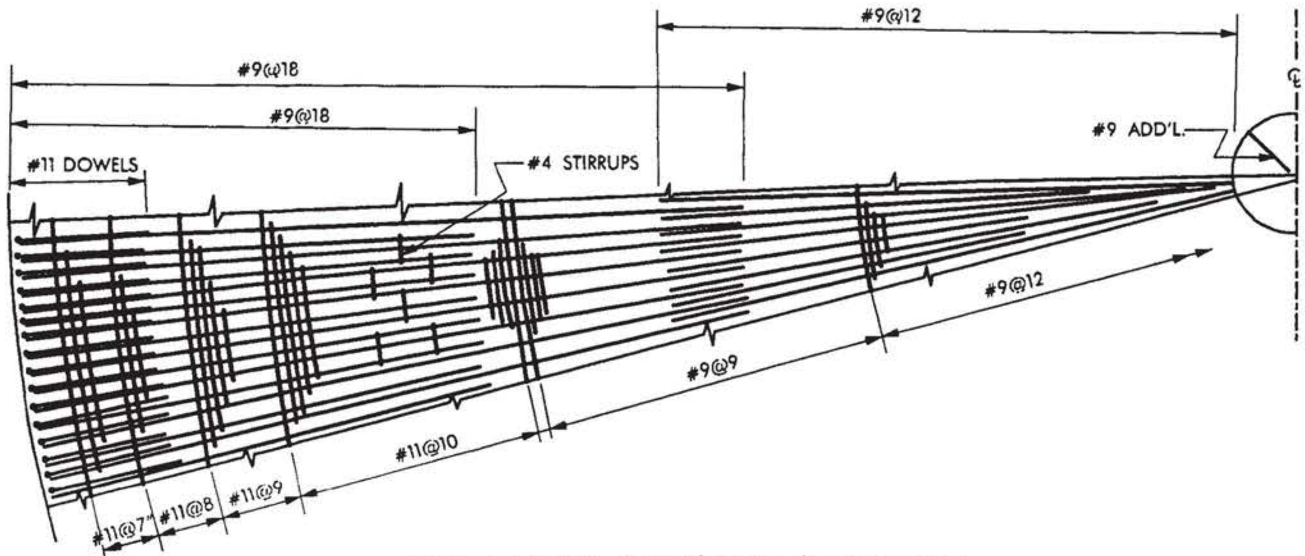
PLAN OF BASE SLAB REINFORCEMENT DETAILS

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12211.DGN		**FIGURE 12.2-11 REV. 18**

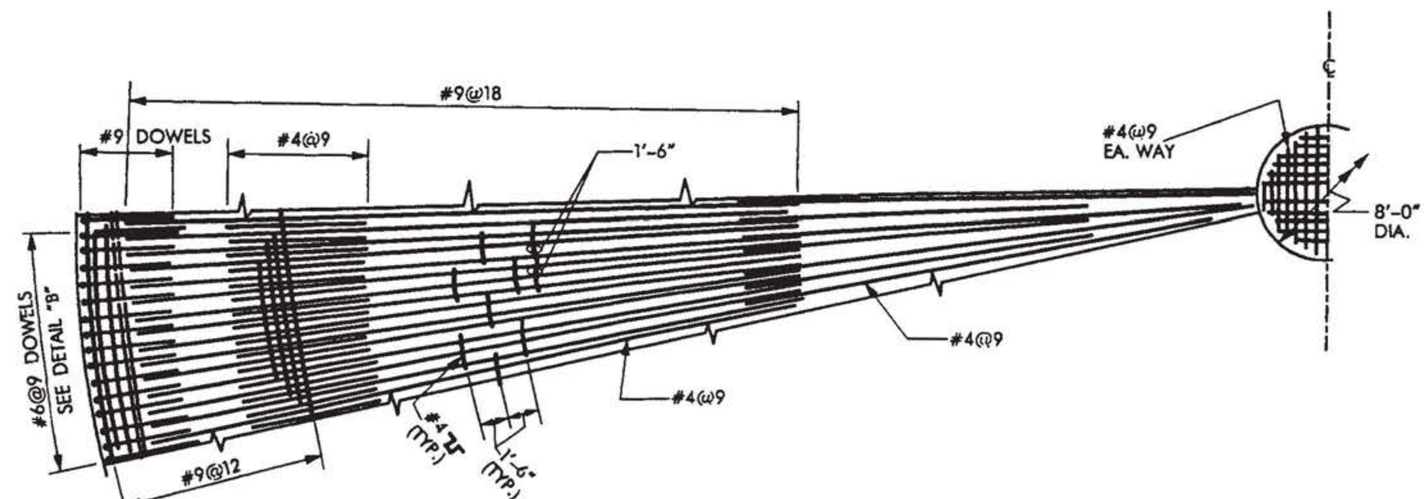


SECTION OF DOME SLAB & WALL REINFORCEMENT DETAILS

DWN GEW	DATE 4-17-03	NORTHERN STATES POWER COMPANY	SCALE· NONE
CHECKED	CAD FILE U12212.DGN	PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	**FIGURE 12.2-12 REV. 25**



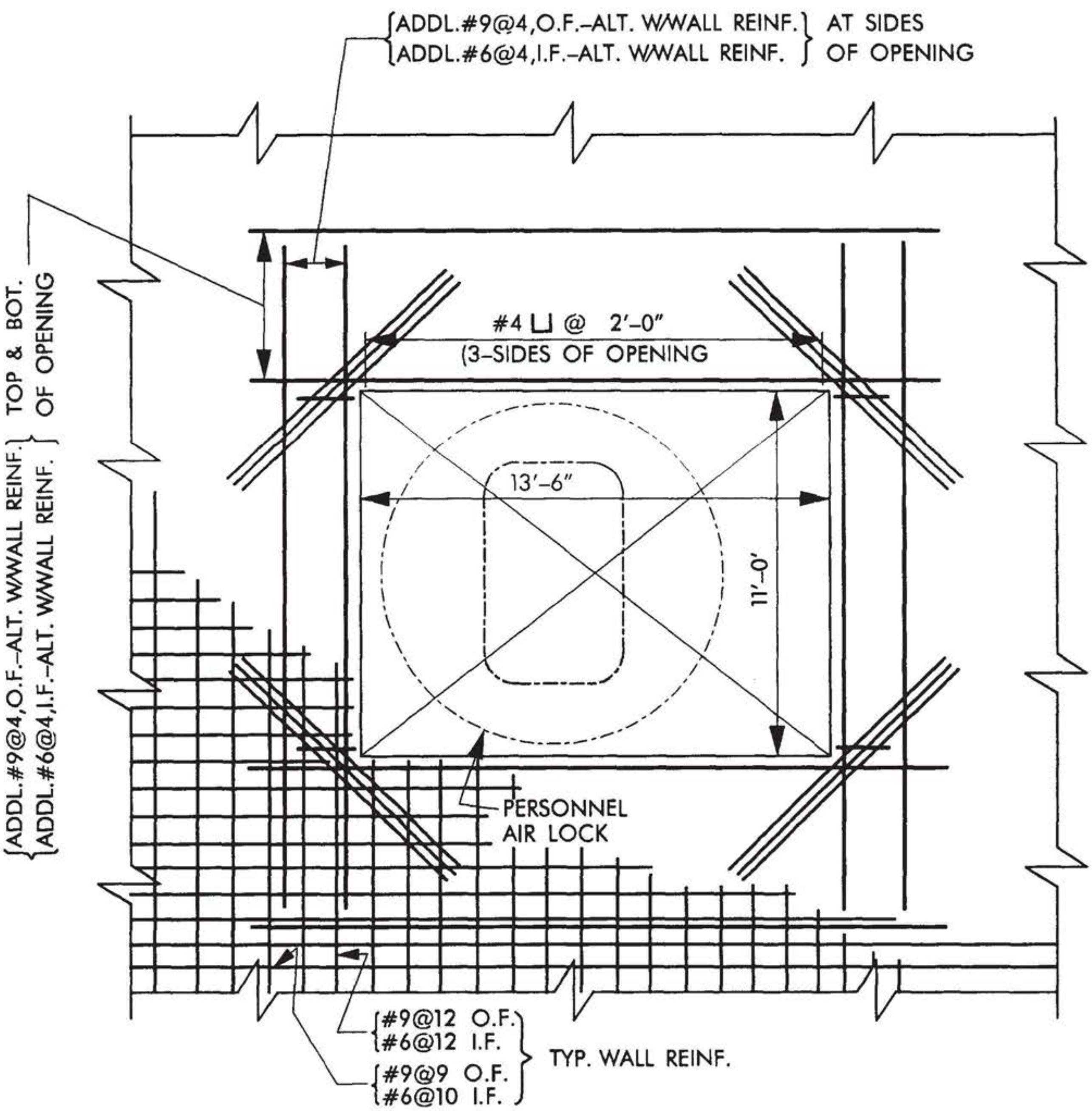
PLAN SHOWING ARRANGEMENT OF TOP REBARS



PLAN SHOWING ARRANGEMENT OF BOTTOM REBARS

PLAN OF DOME SLAB & WALL REINFORCEMENT DETAILS

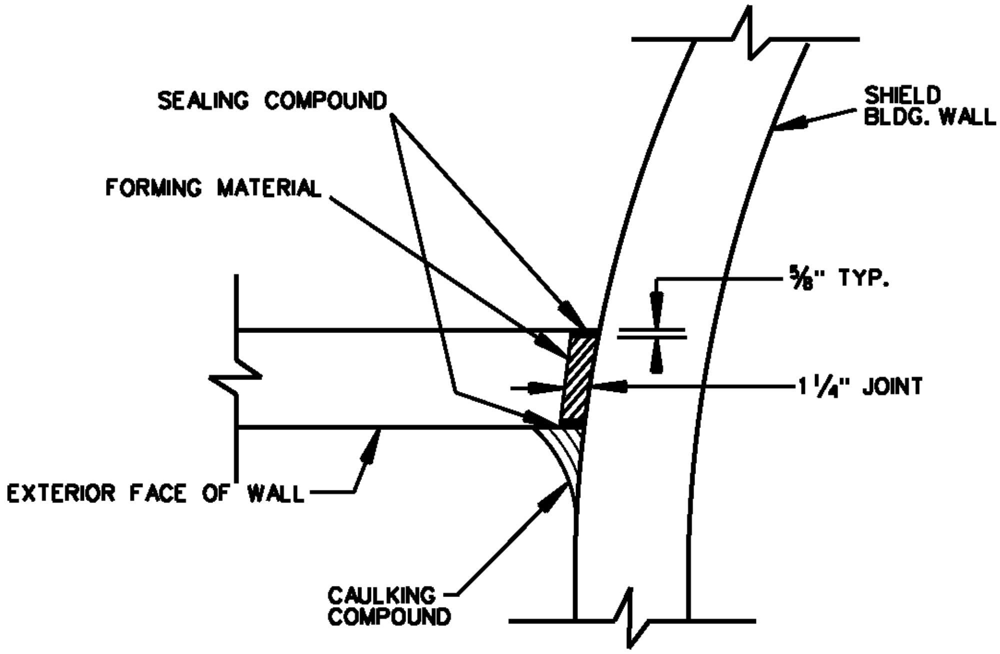
DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE	
CHECKED	CAD FILE U12213.DGN		FIGURE 12.2-13 REV. 18	



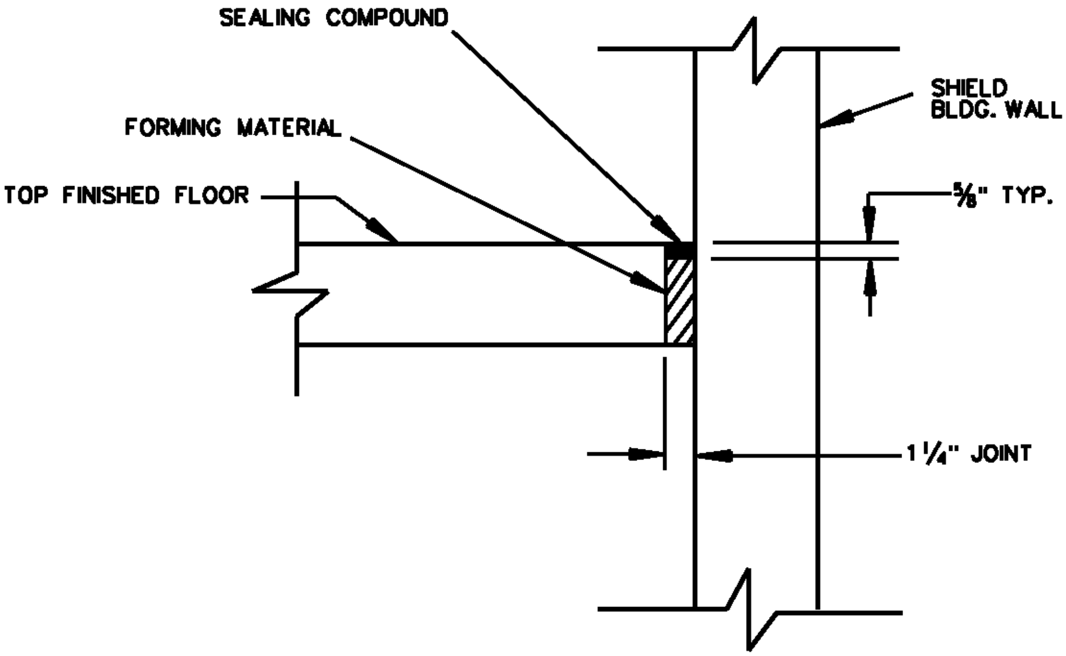
ELEVATION
(OUTSIDE VIEW)

LARGE OPENING IN WALL (TYP. ARRGMT. OF REINF.)

DWN GEW	DATE 6/23/99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE. NONE
CHECKED	CAD FILE U12214.DGN		**FIGURE 12.2-14 REV. 18**



JOINT BETWEEN AUXILIARY BLDG. AND SHIELD BLDG. WALLS

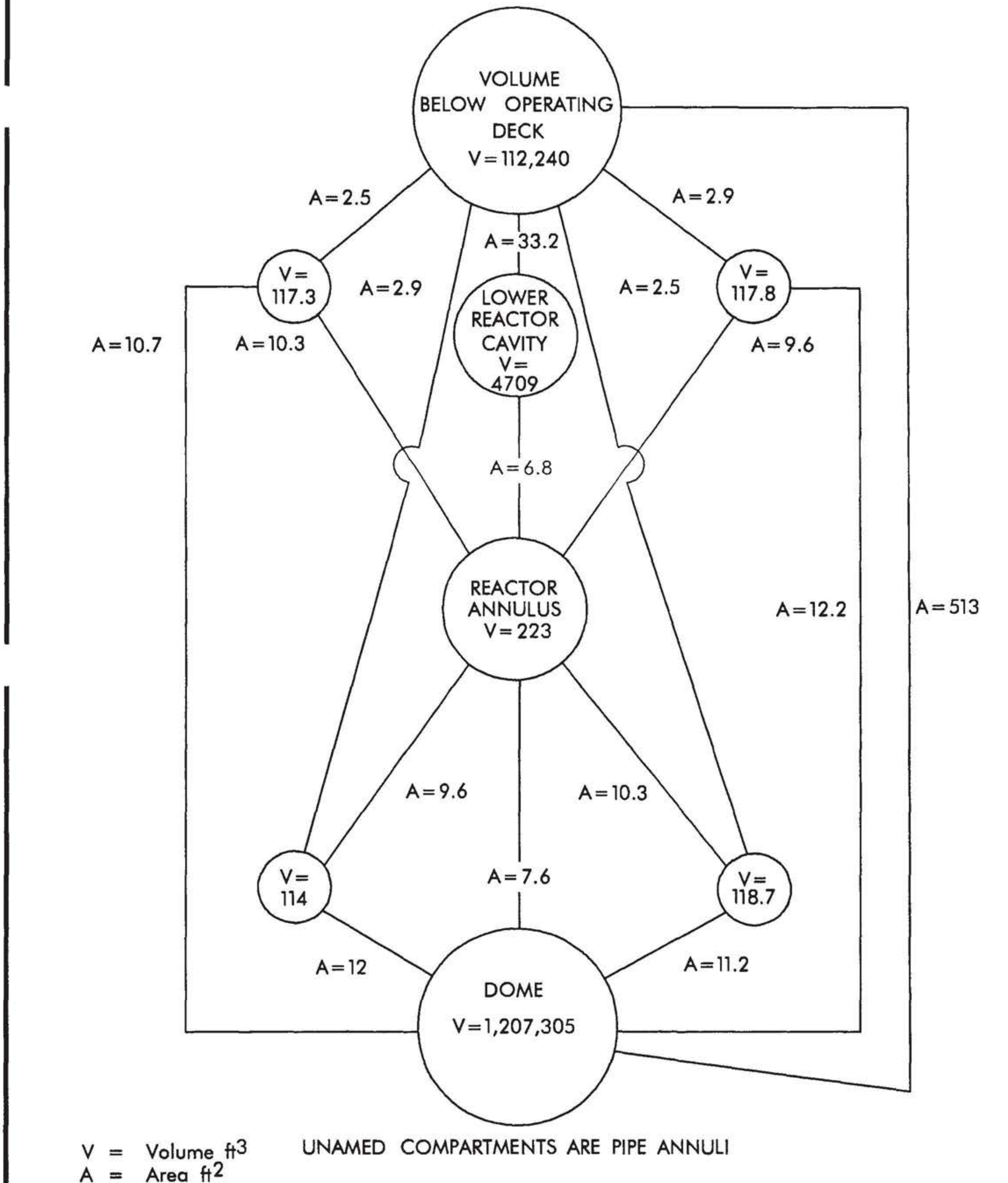


JOINT BETWEEN AUXILIARY BLDG. SLAB AND SHIELD BLDG WALL

01436159

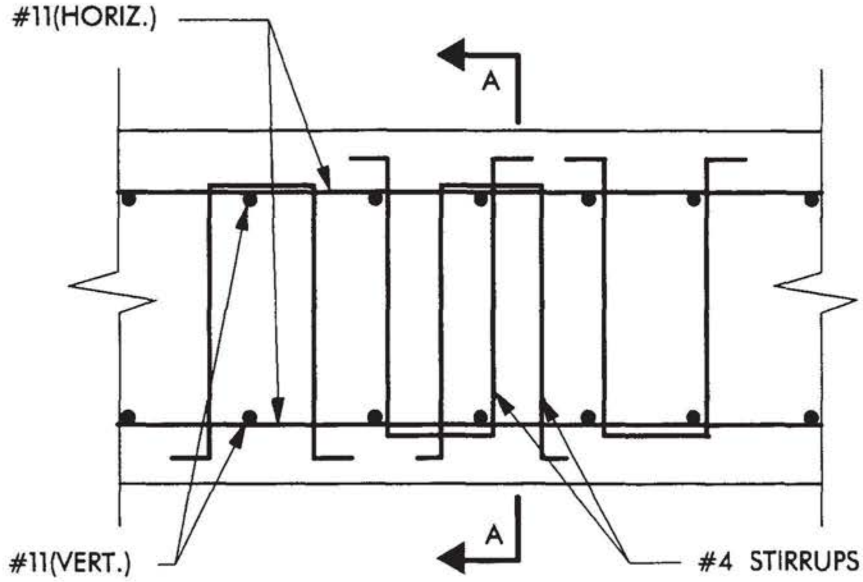
DETAILS FOR JOINTS BETWEEN REACTOR & AUXILIARY BUILDING WALL

DWN: LAB	DATE: 8-11-15	NORTHERN STATES POWER COMPANY Xcel Energy	SCALE: NONE
CHECKED:	CAD FILE: U12215.DGN	PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING, MINNESOTA	*FIGURE 12.2-15 REV. 34*

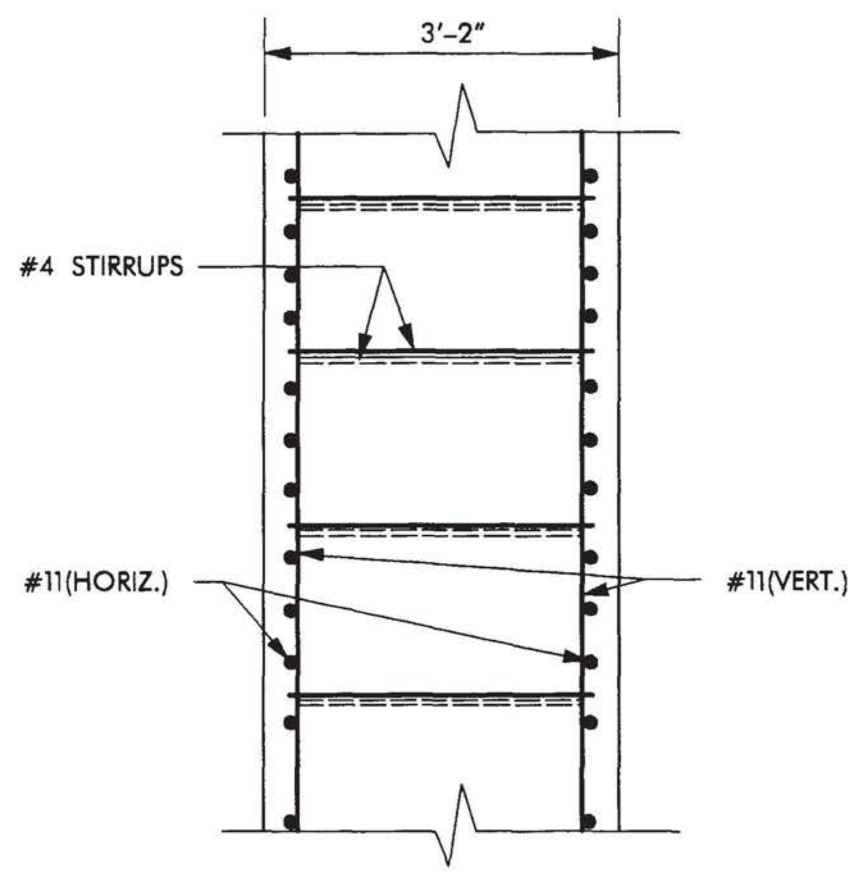


VOLUME AND FLOW PATH SCHEMATIC OF REACTOR CAVITY REGION

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE : NONE
CHECKED	CAD FILE U12216.DGN		**FIGURE 12.2-16 REV. 18**



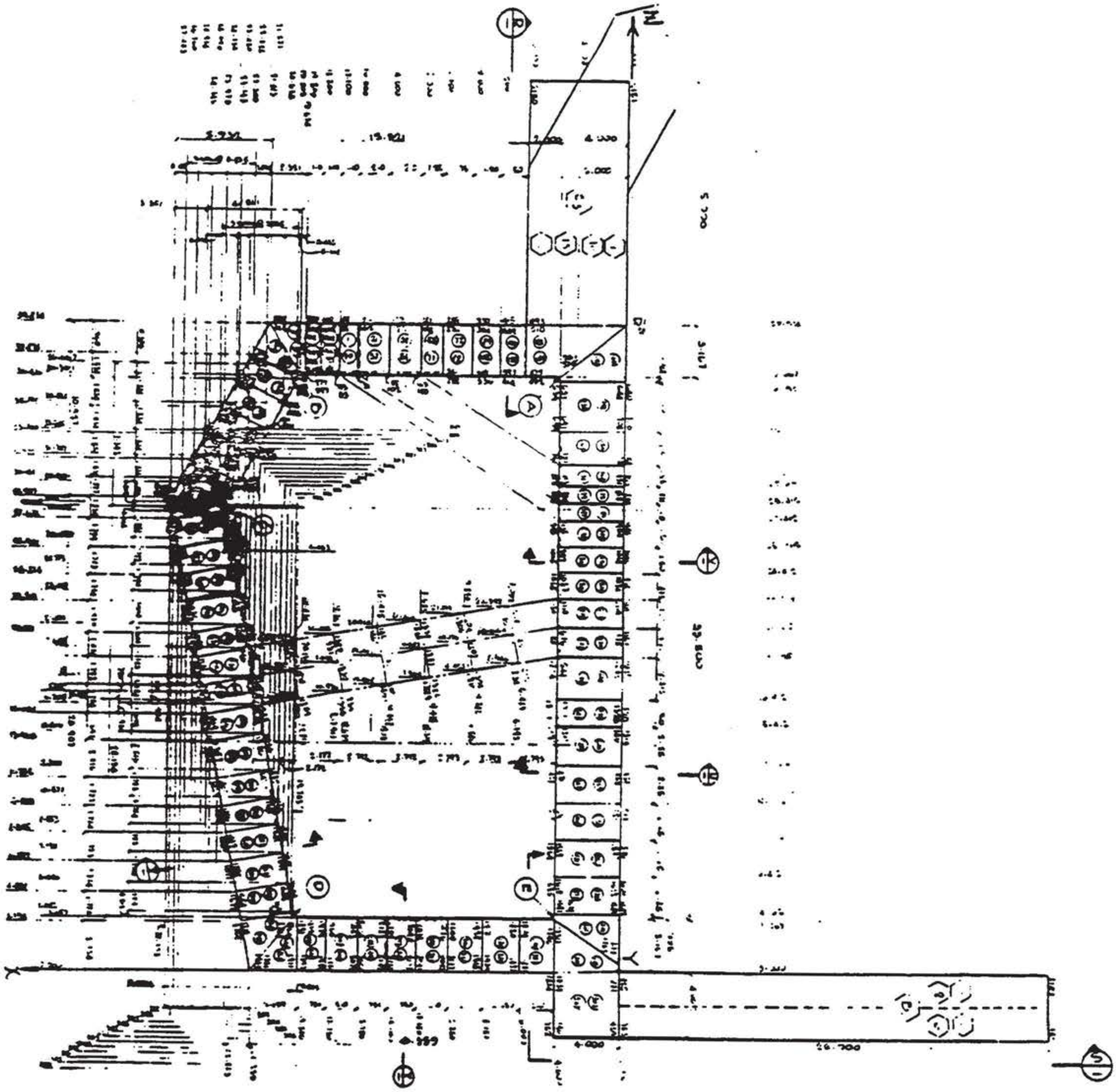
TYPICAL STIRRUP DETAIL IN WALLS



SECTION A–A

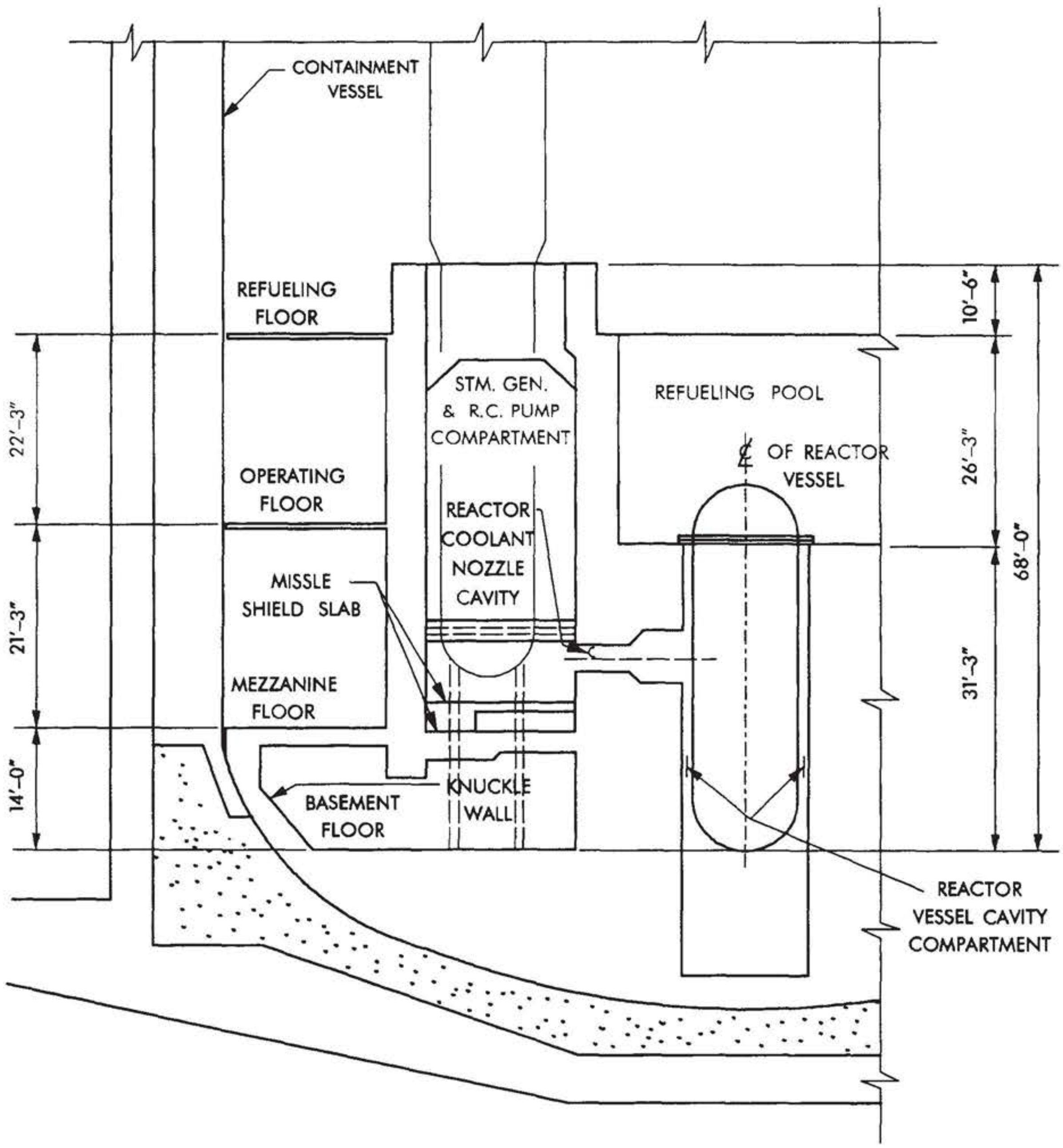
PLAN VIEW AND SECTION OF WALL SEGMENT SHOWING STIRRUP

DWN GEW	DATE 6–23–99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12217.DGN		**FIGURE 12.2-17 REV. 18**



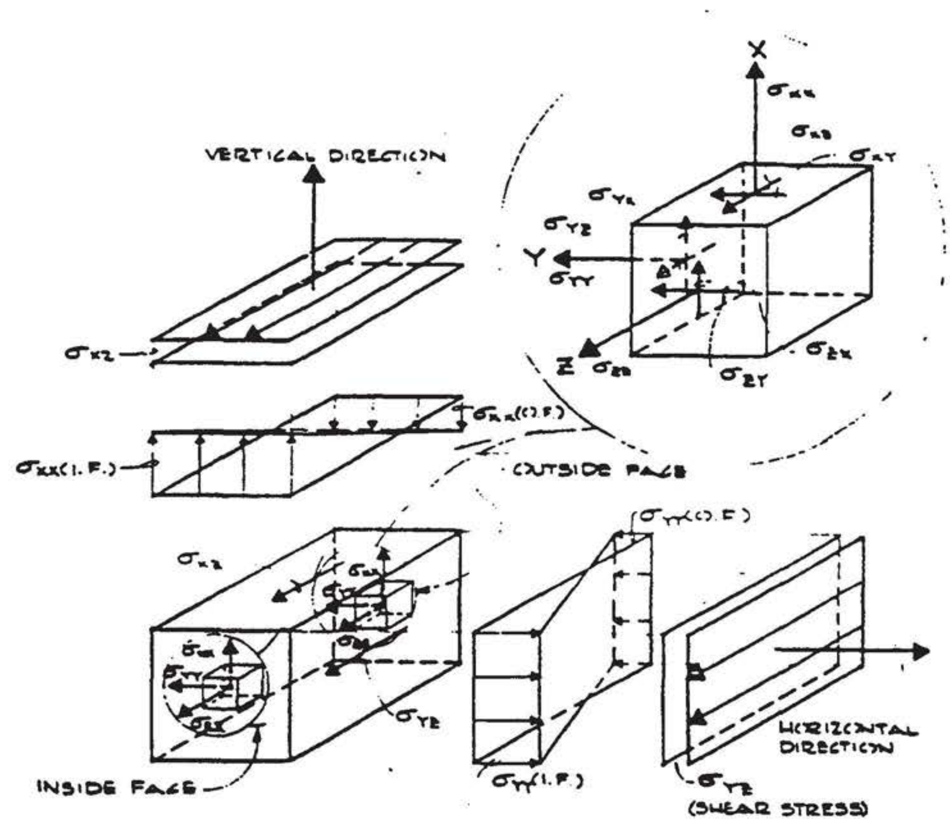
PLAN VIEW OF STRUCTURAL MODEL OF STEAM GENERATOR COMPARTMENT (HEIGHT 68 FT.)

DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12218.DGN		**FIGURE 12.2-18 REV. 18**



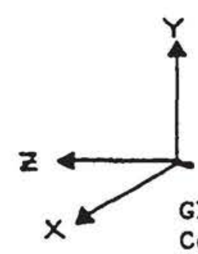
STEAM GENERATOR AND REACTOR CAVITY COMPARTMENT CONFIGURATION-SECTION

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12219.DGN		**FIGURE 12.2-19 REV. 18**



NOTES:

1. Inside Face: Facing Steam Generator and Reactor Coolant Pump.
2. Outside Face: Opposite of Inside Face of Wall.
3. σ_{xx}, σ_{yy}, σ_{zz} : Positive for Tension and Negative for Compression.

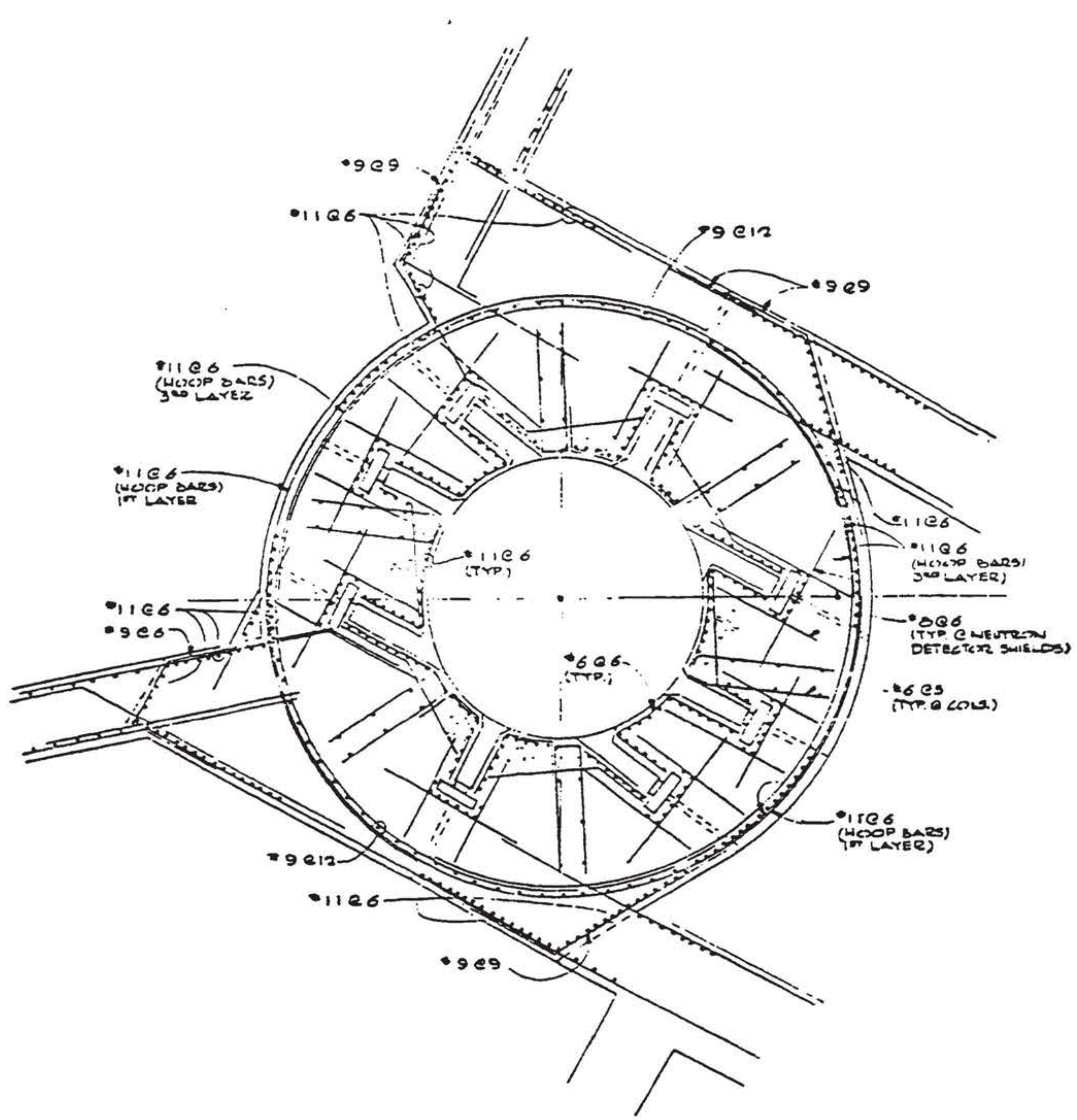


Global Coordinate System

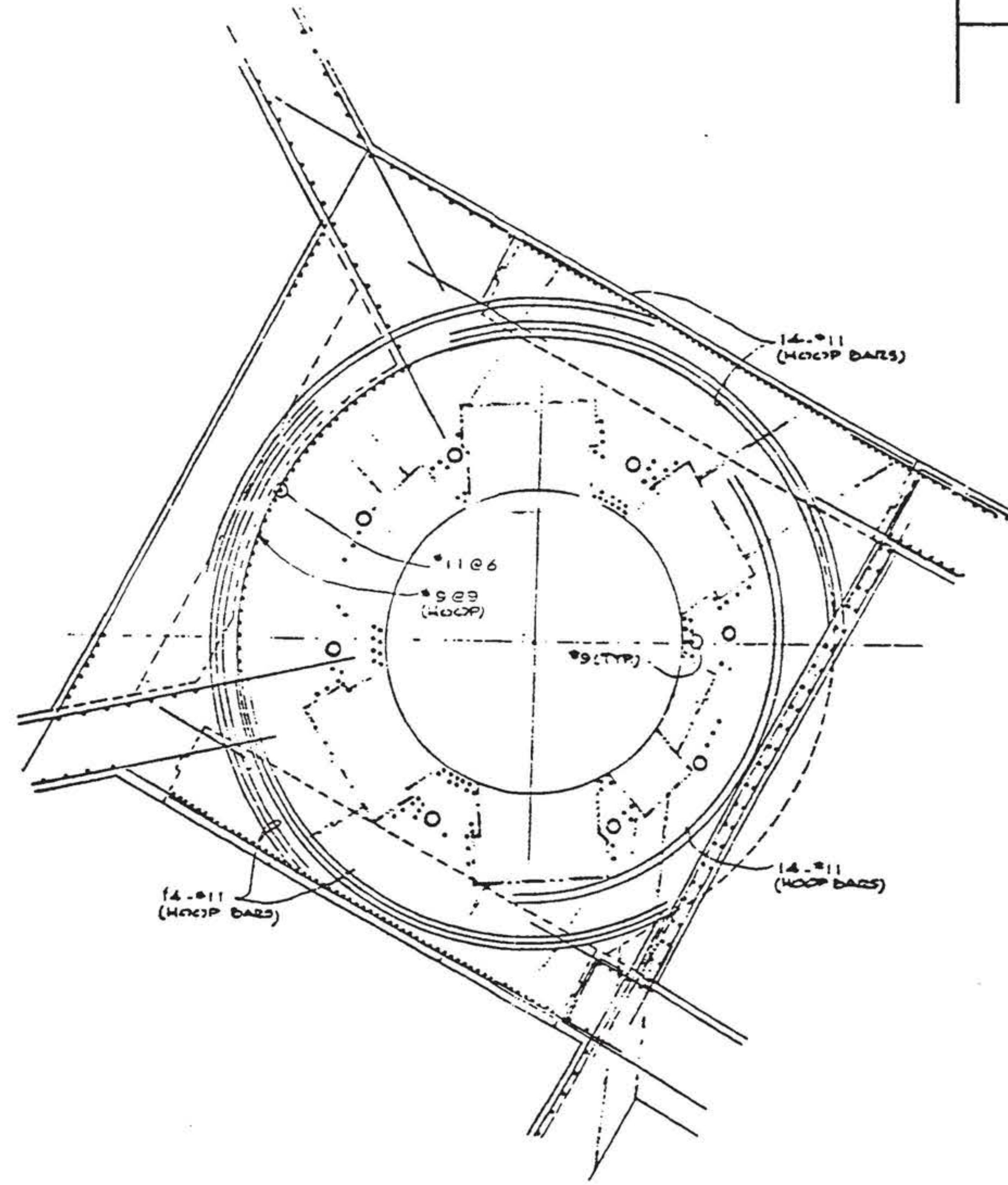
STRESS OUTPUT FOR THREE DIMENSIONAL BRICK ELEMENT IN LOCAL COORDINATE SYSTEM

DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12220.DGN		FIGURE 12.2-20 REV. 18

REVISIONS



HORIZONTAL SECTION
FROM EL. 697'-6 TO EL. 718'-8



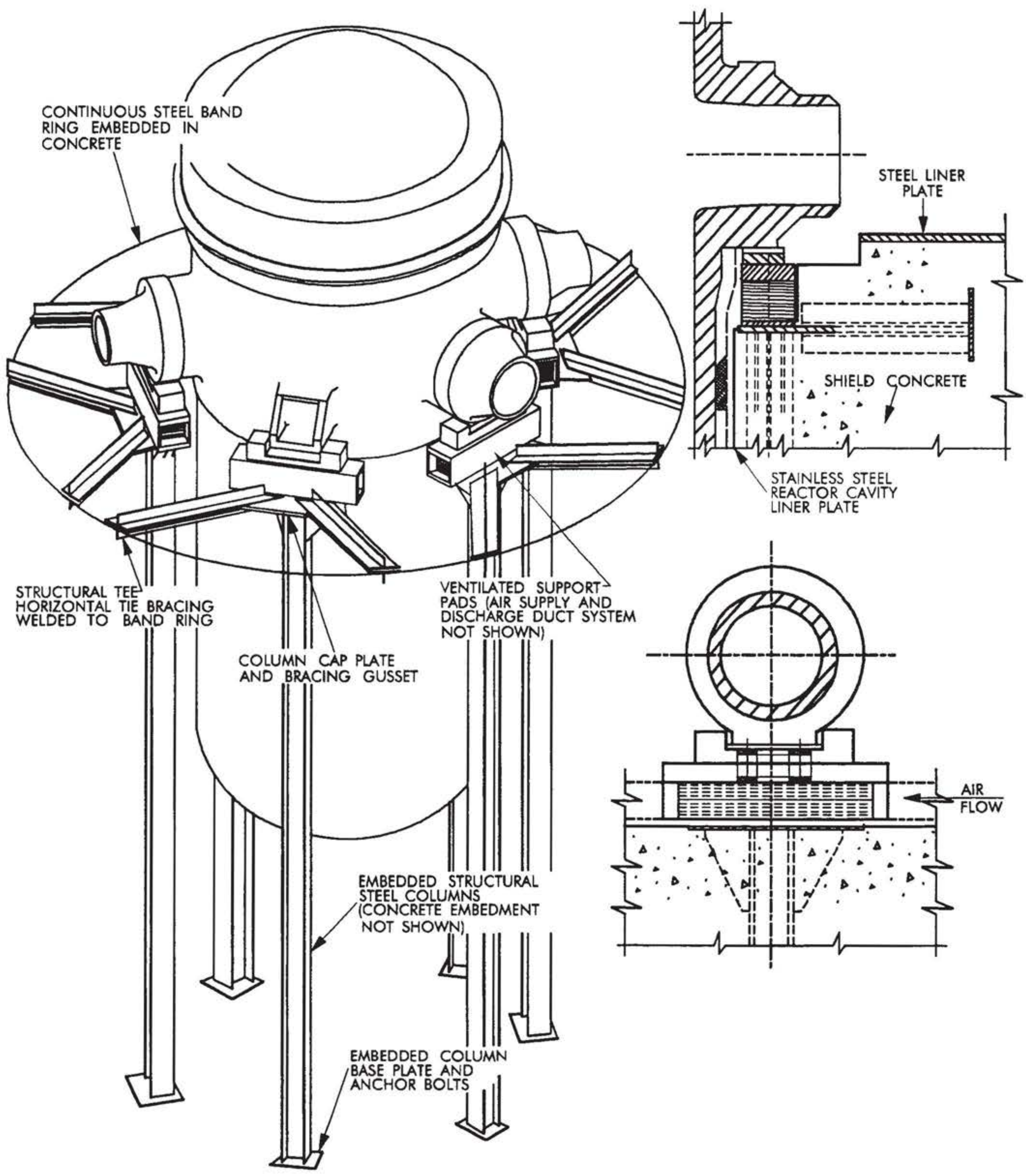
HORIZONTAL SECTION
FROM EL. 718'-8 TO EL. 728'-9

DWN GEW	DATE 6-23-99	SIGNIFICANT NO.						
CHECKED		GROUP	1	2	3	4	5	CL 6
PROJECT NO. ETNSUR								
APP'D & CERT.		REACTOR VESSEL BIOLOGICAL SHEILD TYPICAL REINFORCING						
CAD FILE U12221.DGN								

NORTHERN STATES POWER COMPANY
PRAIRIE ISLAND NUCLEAR GENERATING PLANT
RED WING, MINNESOTA

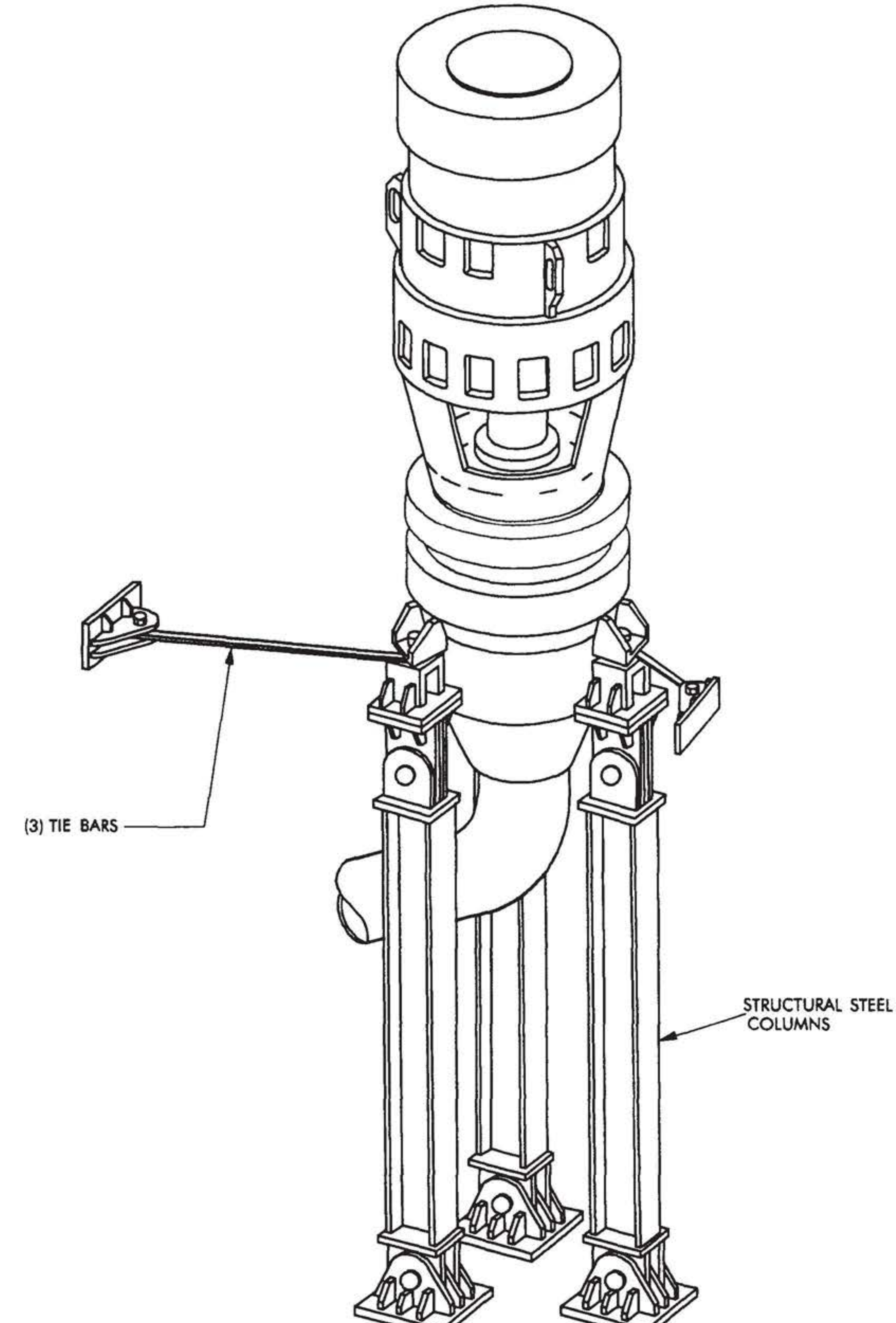
SCALE NONE

FIGURE 12.2-21 REV. 18



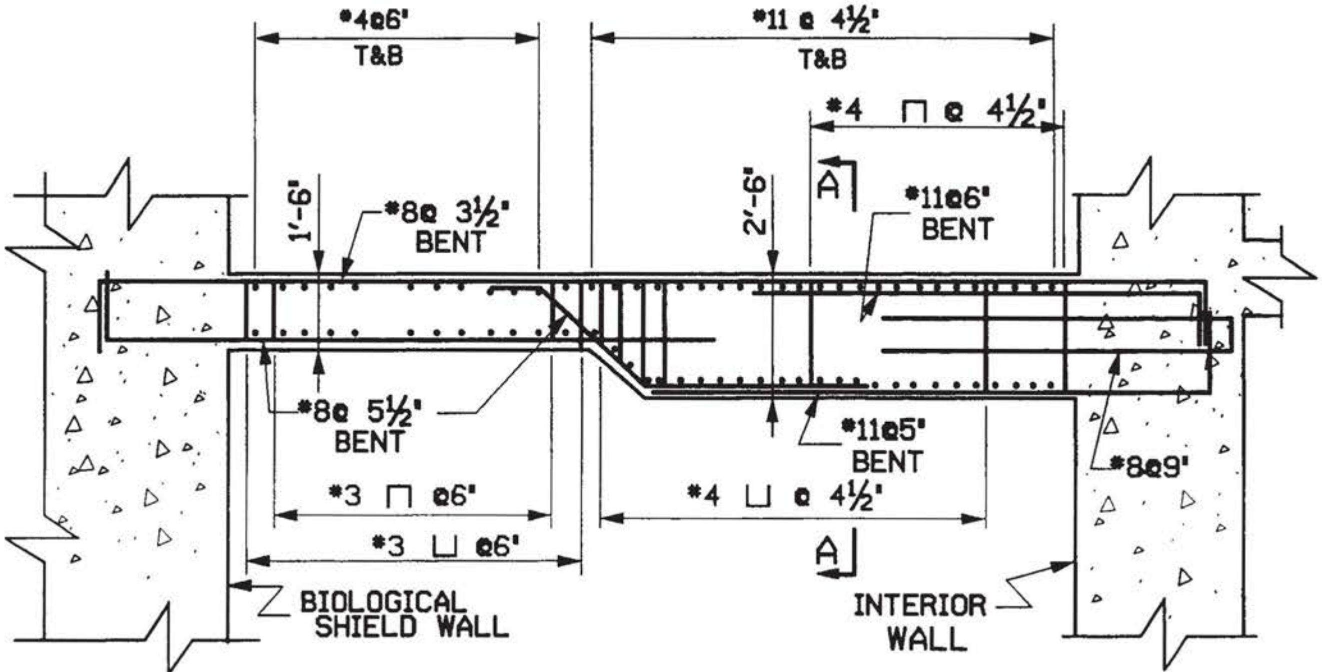
REACTOR VESSEL SUPPORTS

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY	SCALE: NONE
CHECKED	CAD FILE U12222.DGN	PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	**FIGURE 12.2-22 REV. 18**

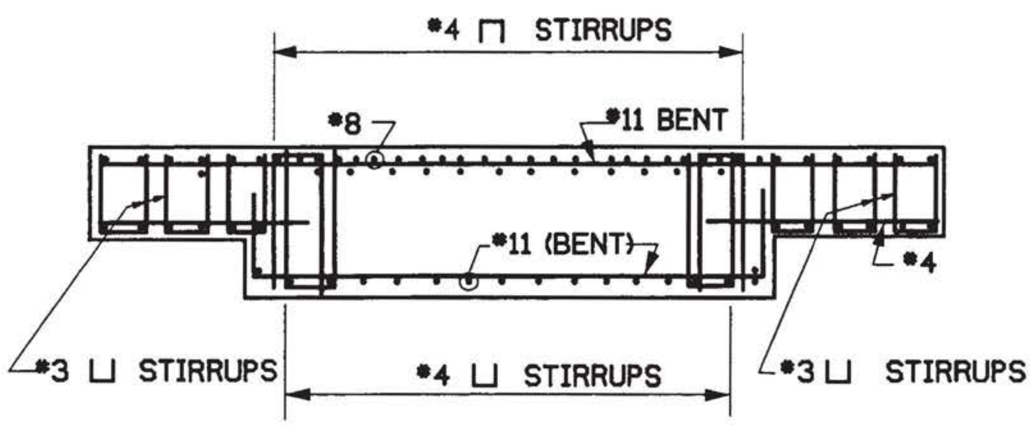


REACTOR COOLANT PUMP SUPPORTS

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12223.DGN		**FIGURE 12.2.23 REV. 18**



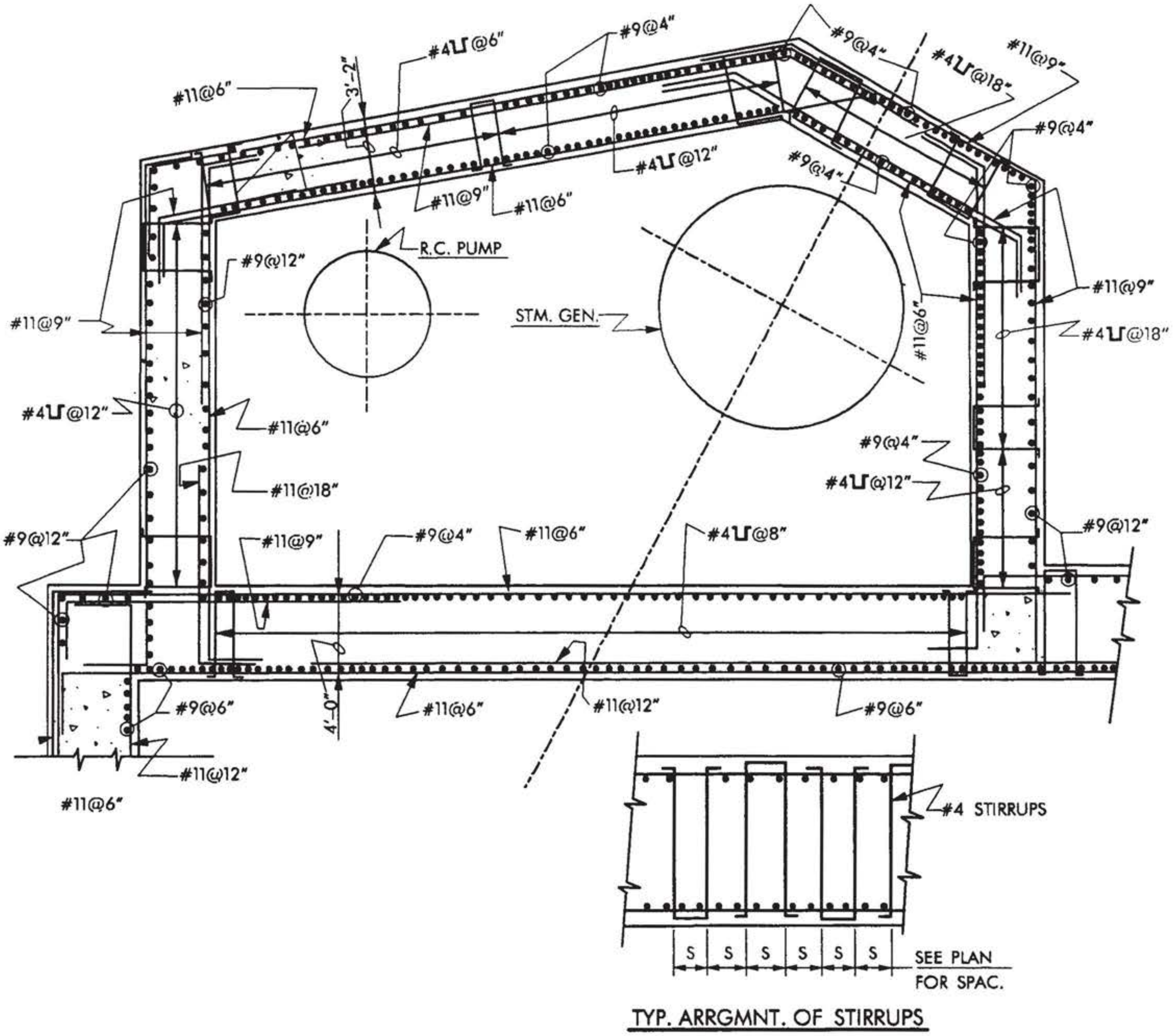
TYPICAL FLOOR SECTION



SECTION A-A

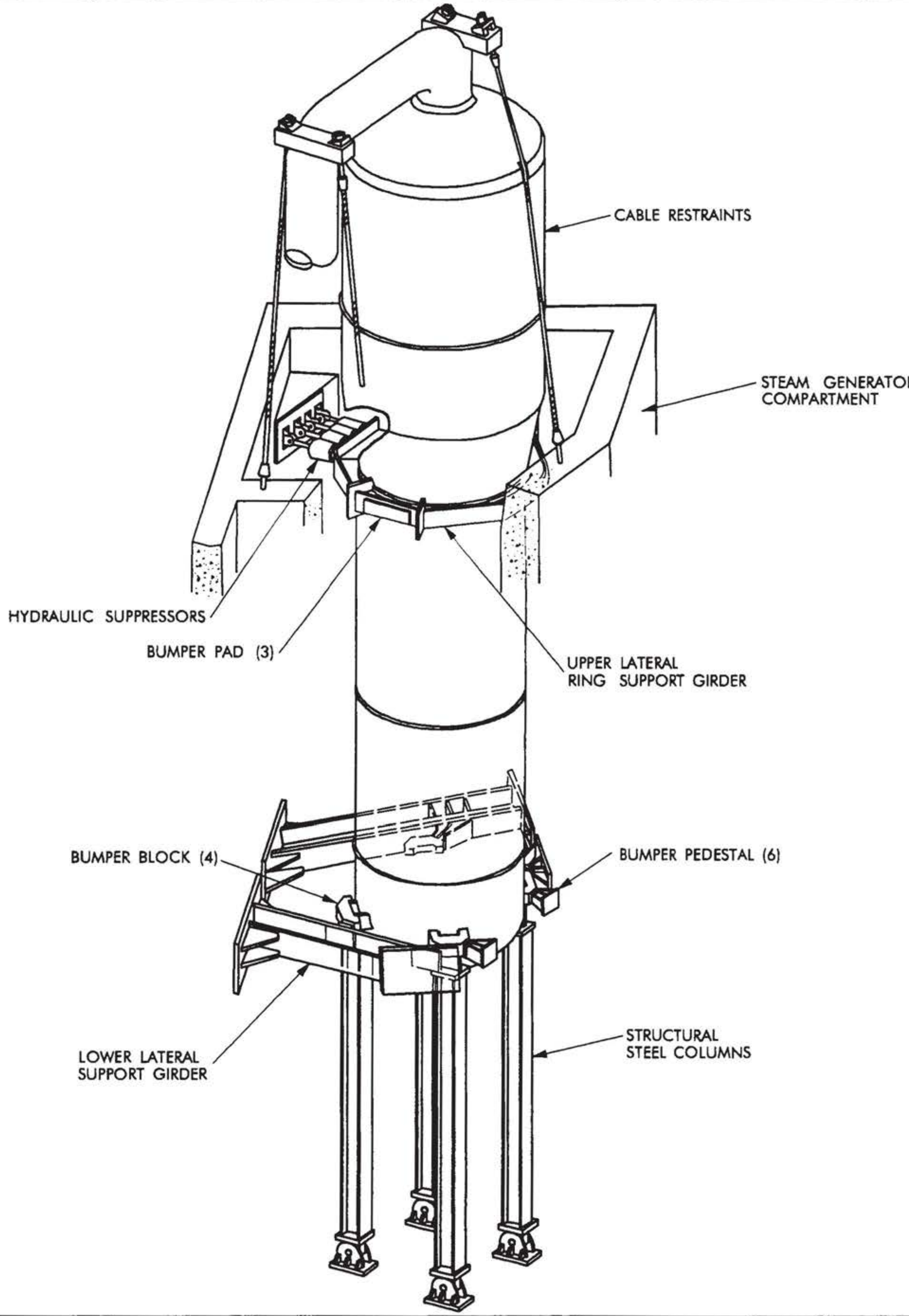
CONCRETE BEAM FLOOR (TYPICAL ARRANGEMENT OF REINFORCEMENT)

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12224.DGN		FIGURE 12.2-24 REV. 18



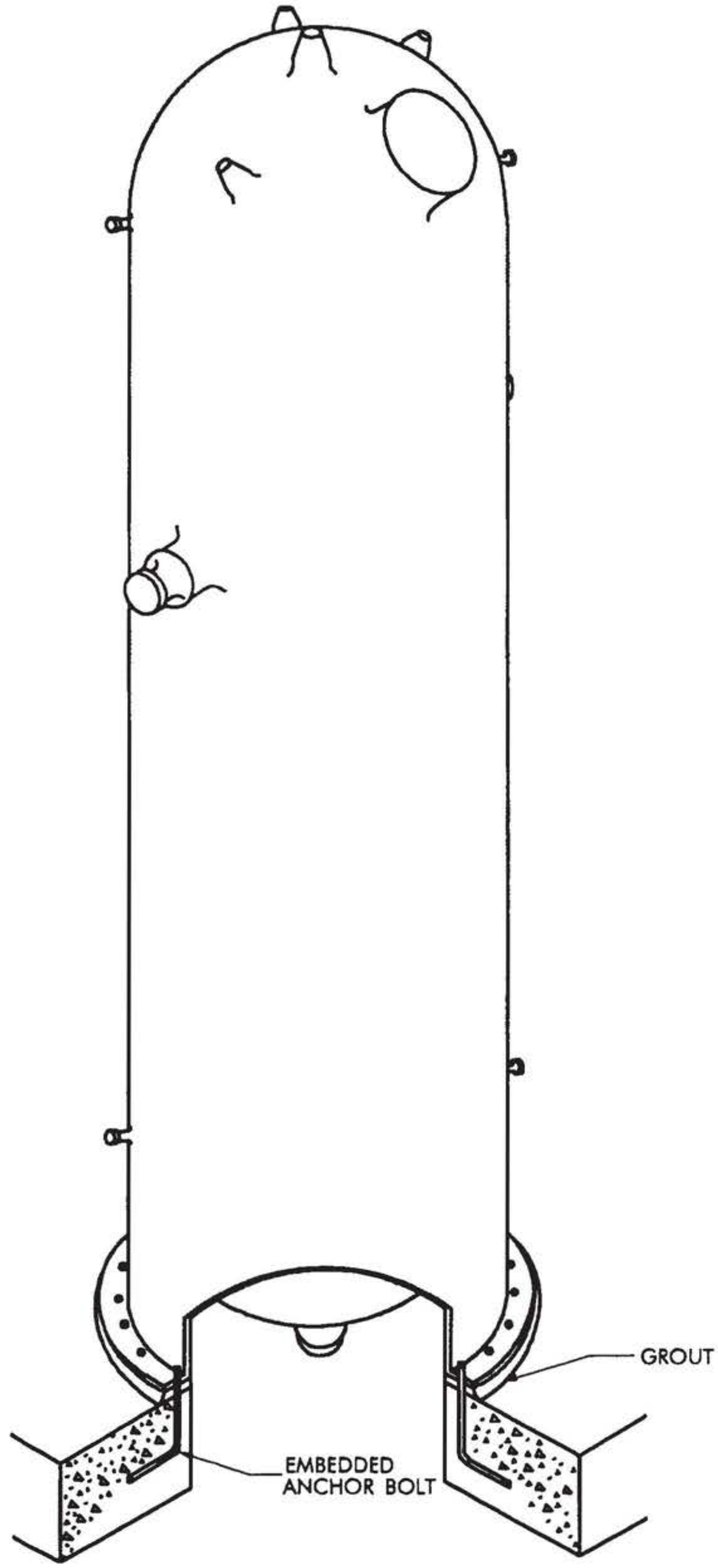
ENCLOSURE WALLS FOR STEAM GENERATOR AND REACTOR COOLANT PUMP (TYPICAL PLAN OF REINFORCEMENT)

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12225.DGN		**FIGURE 12.2-25 REV. 18**



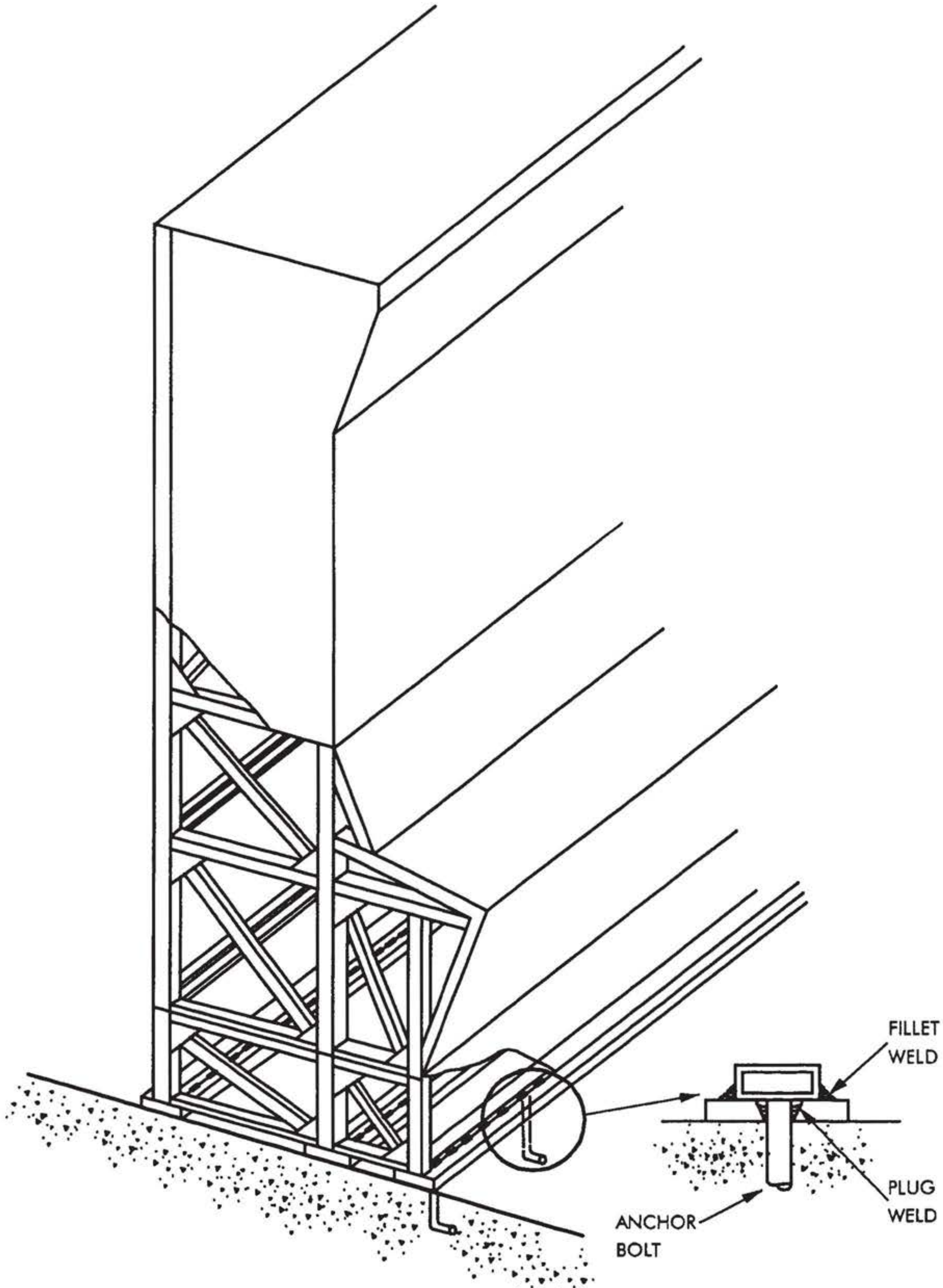
STEAM GENERATOR SUPPORTS

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE	
CHECKED	CAD FILE U12226.DGN		**FIGURE 12.2-26**	**REV. 18**



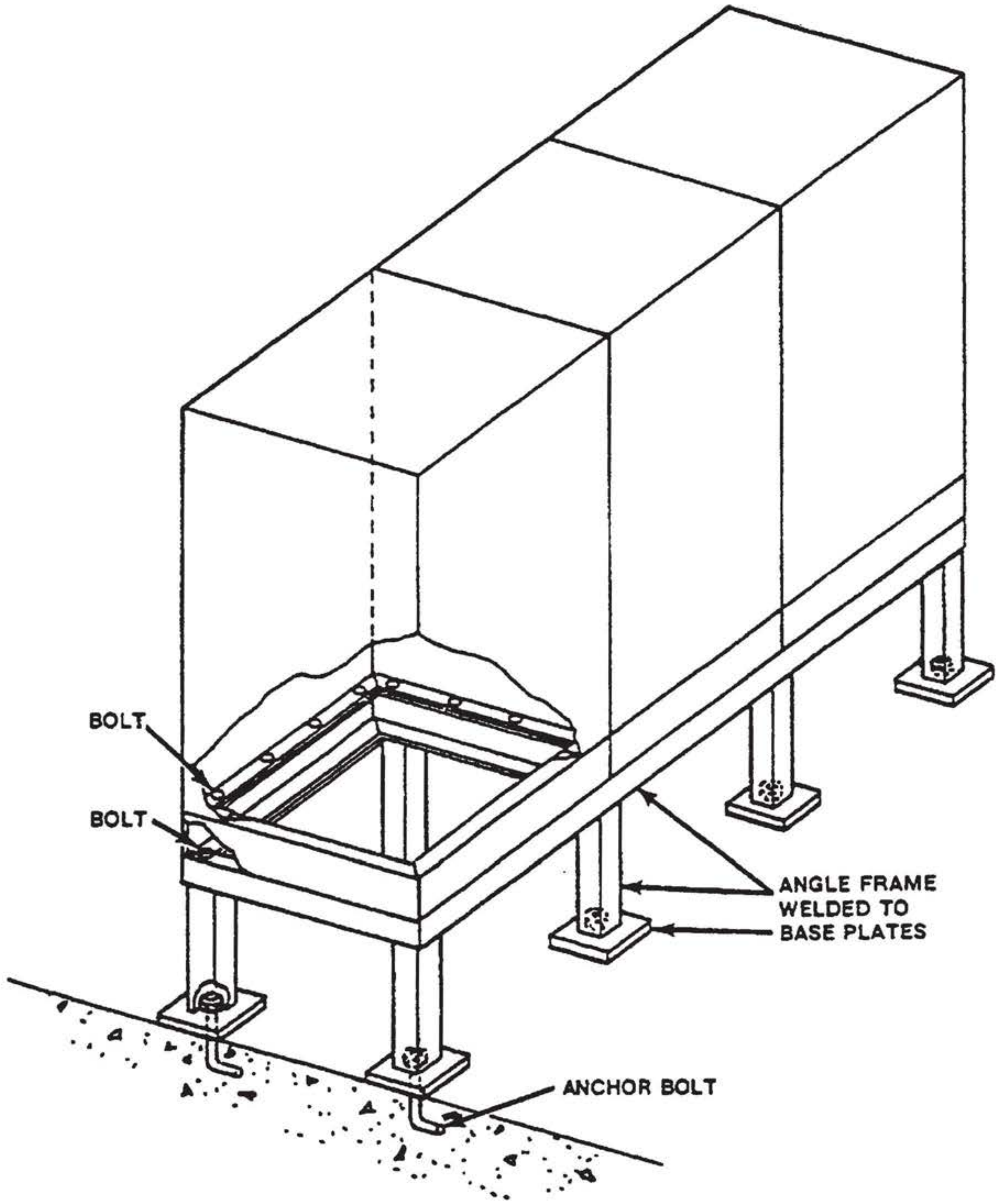
PRESSURIZER SUPPORT

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12227.DGN		**FIGURE 12.2-27 REV. 18**



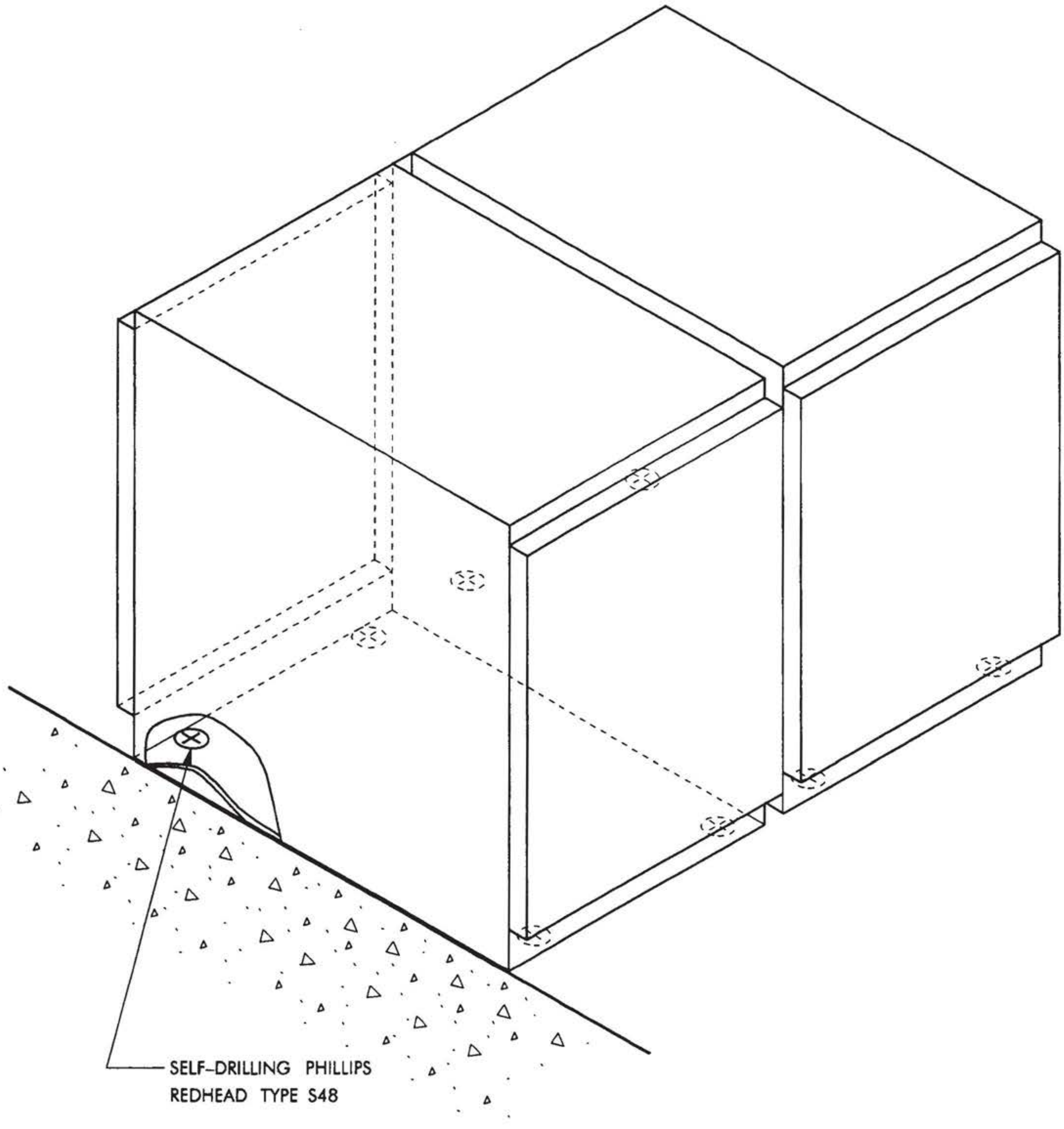
CONTROL ROOM EQUIPMENT NSS CONTROL PANEL SUPPORTS

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE	
CHECKED	CAD FILE U12228.DGN		**FIGURE 12.2-28 REV. 18**	



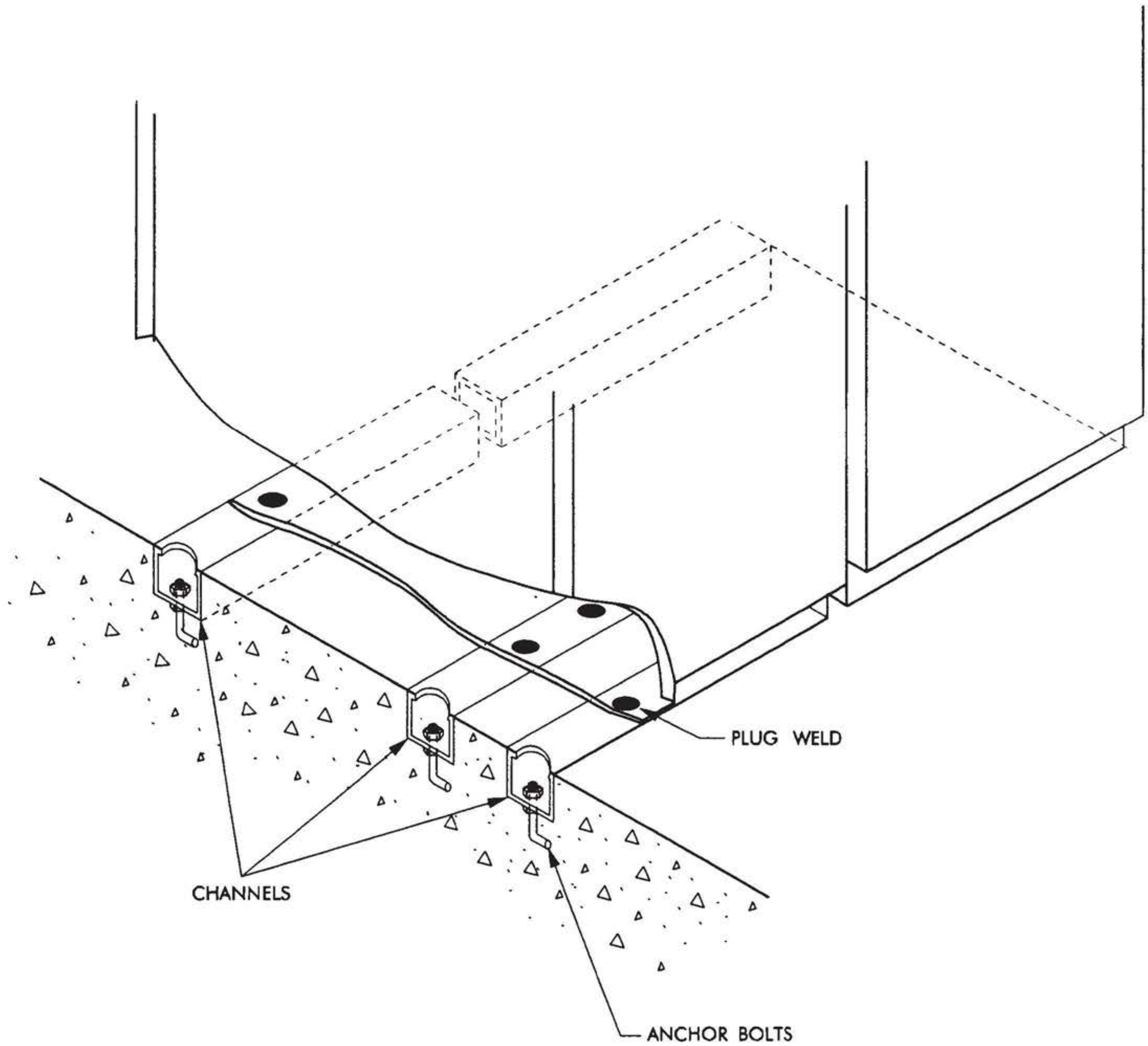
CONTROL ROOM EQUIPMENT MOUNTING FOR PROCESS CONTROL
RADIATION MONITOR AND NIS CONTROL PANELS

DWN T. MILLER	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12229.DGN		**FIGURE 12.2.29 REV. 18**



SWITCHGEAR MOUNTING DETAILS

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12231.DGN		FIGURE 12.2-31 REV. 18



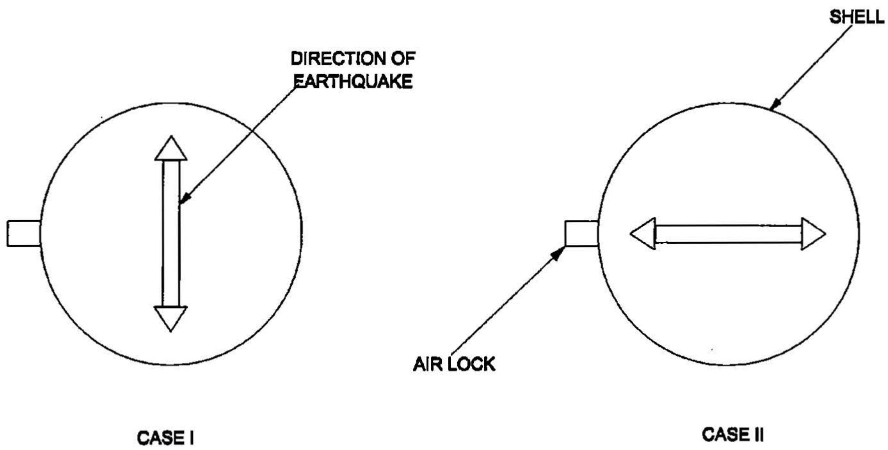
SWITCHGEAR MOUNTING DETAILS

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY	SCALE: NONE
CHECKED	CAD FILE U12232.DGN	PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	**FIGURE 12.2-32 REV. 18**

FIGURE 12.2.33

DELETED

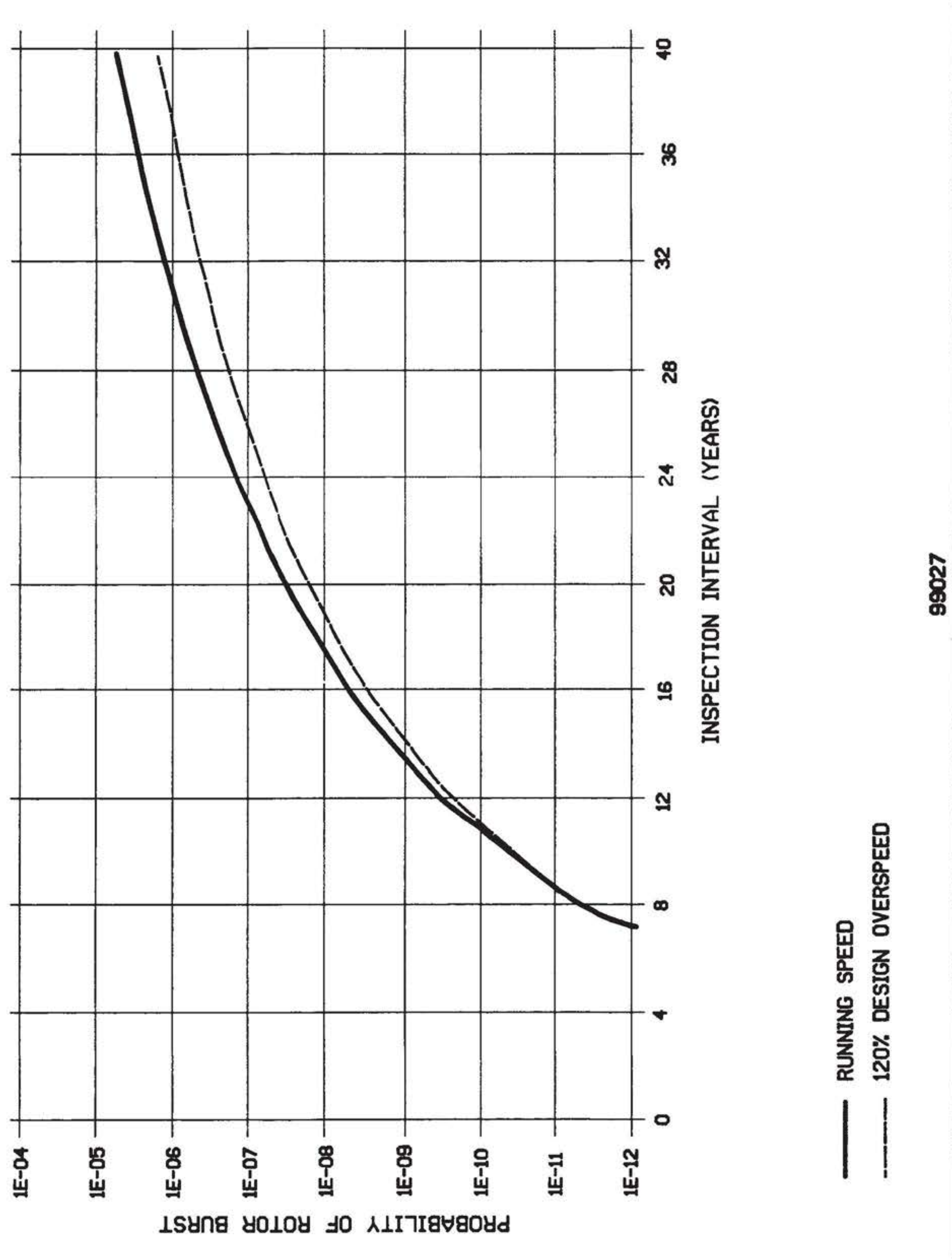
01409787



EFFECT OF AIR LOCK ON SEISMIC ANALYSIS

DWN GEW	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE
CHECKED	CAD FILE U12234.DGN		FIGURE 12.2-34 REV. 18

FIGURE 12.2-34



PROBABILITY OF TURBINE RUPTURE DUE TO STRESS CORROSION

DWN TAM	DATE 6-23-99	NORTHERN STATES POWER COMPANY PRAIRIE ISLAND NUCLEAR GENERATING PLANT RED WING MINNESOTA	SCALE: NONE	
CHECKED	CAD FILE U12230.DGN		FIGURE 12.2-38	REV. 18

Figure 12.3-1 withheld from public disclosure under 10 CFR 2.390

01521968

Figure 12.3-2 withheld from public disclosure under 10 CFR 2.390

01521968

Figure 12.3-3 withheld from public disclosure under 10 CFR 2.390 01521968

Figure 12.3-4 withheld from public disclosure under 10 CFR 2.390

01521968

Figure 12.3-5 withheld from public disclosure under 10 CFR 2.390

01521968

Figure 12.3-6 withheld from public disclosure under 10 CFR 2.390

01521968

Figure 12.3-7 withheld from public disclosure under 10 CFR 2.390

01521968