UCRL 15140

MASTER

Subcontract 6440709

POTENTIAL SEISMIC STRUCTURAL FAILURE
MODES ASSOCIATED WITH THE ZION NUCLEAR PLANT

SEISMIC SAFETY MARGINS RESEARCH PROGRAM
(PHASE I)
Project VI - Fragilities

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prepared for
Nuclear Test Engineering Division
LAWRENCE LIVERMORE LABORATORY
Livermore, California

October 1979

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1. INTRODUCTION

The Zion 1 and 2 Nuclear Power Plant consists of a number of structures. The most important of these from the viewpoint of safety are the containment buildings, the auxiliary building, the turbine building, and the crib house (or intake structure). Several methods of construction were used for these buildings including braced frame steel structures, reinforced concrete, and prestressed concrete. The auxiliary building and turbine buildings for Units 1 and 2 are structurally coupled and involve a combination of braced frame steel structure and reinforced concrete shear wall construction. Also, the turbine/auxiliary building is significantly nonsymmetric about one axis such that considerable torsional response to north-south excitation is expected.

The evaluation of the potential seismic failure modes and determination of the ultimate seismic capacity of the structures is a complex undertaking which will require a large number of detailed calculations. As the first step in this evaluation, a number of potential modes of structural failure have been determined and are discussed in this report.

In this report the emphasis is on those structures which support systems necessary to assure the safe shut down of the reactors and prevent the release of radioactive materials. Of primary importance are the reactor containment buildings, the combined turbine/auxiliary building, and the crib house. Further, emphasis is on seismically induced failure modes rather than on those resulting from Loss of Coolant Accident (LOCA) or other load combinations even though these other loads may be a secondary result from seismic excitation. In some cases where the consideration of additional load combinations may significantly affect the seismic response, these effects are also discussed.

The report is principally directed towards seismically induced failure of structures. To some extent, modes involving soil foundation failures are discussed in so far as they affect the buildings. However, failure modes involving soil liquefaction, surface faulting, tsunamis, etc., are considered outside the scope of this evaluation. Similarly, equipment anchorage failure involving shear or pullout of concrete anchor bolts or local equipment support failures are not discussed in this report. Although such details may be within the civil/structural scope of supply this type of failure results from equipment response. Potential failures involving rupture of storage water tanks which form an integral part of the structure are considered but metal fabricated tanks are discussed in a separate report for equipment.

A structural system as complex as the Zion facility requires consideration of a very large number of potential failure modes. This includes the evaluation of a great many structural elements and details to determine their load deformation characteristics and ultimate load capacities. These detailed calculations and dynamic system response analyses have not yet been performed for Zion. Consequently, damage and collapse acceleration levels corresponding to the various failure modes are not yet available. Neither has any attempt been made to rank the failure modes in order as to probable order of occurrence or severity of results. Thus, among the failure modes listed are a number which have a low probability of occurrence. Finally, it is expected that many of the failure modes will be interrelated and occur in conjunction with each other at the very high levels of seismic response anticipated.

2. CONTAINMENT BUILDING

2.1 CONTAINMENT VESSEL DESCRIPTION

The reactor containment buildings for Zion Units 1 and 2 are vertical circular cylinders with shallow domed roofs. They enclose the concrete internal structures, the reactor vessels, and reactor coolant systems. The containment vessels and the concrete internals are supported by independent flat circular foundation slabs which include a sump near the center to house the reactor vessel. The cylindrical portion of the containment vessel is prestressed by a post-tensioning system which consists of horizontal and vertical unbonded tendons. The horizontal hoop tendons terminate in one of the six equally spaced vertical buttresses which extend from the base slab to above the spring line of the vessel. The dome is prestressed by a three-way post tensioning system. Vertical and circumferential reinforcing steel is placed in the cylinder and the dome contains radial and circumferential reinforcing steel toward the outside diameter and reinforcing steel in a rectangular grid near the center. The foundation slab is conventionally reinforced with high-strength steel. Other than the vertical containment vessel tendons which extend through the base slab, no prestressing is used for the base slab. The entire structure is lined with 1/4 inch welded steel plate to provide vapor tightness.

The containment structures for Units 1 and 2 are essentially identical in design and construction. The dimensions for the containment are:

Outside diameter	147'-0"
Wall thickness	3'-6"
Vessel Height	214'-8"
Dome Thickness	2'-8"
Base Slab Thickness	9'-0"
Base O.D.	157'-0"

A vertical section of the containment vessel excluding the concrete internals and equipment is shown in Figure 2-1. Grade elevation is 591'-0" and the top of the base slab is located at elevation 565'-0". Near the center of the slab, a sump which houses the reactor vessel extends from elevation 565'-0" down to elevation 539'-0". The inside diameter of the sump is 21 feet and the wall thickness at this location is 16 feet. The floor slab under the sump is 9 feet thick.

The pour sequence of the foundation slabs for Units 1 and 2 was somewhat different as shown in Figure 2-2. The same figure shows the vertical construction joint (shear key) used. A three foot thick reinforced concrete slab was poured on top of the foundation slab. The purpose of this slab is to protect the 1/4 inch steel liner from being damaged by missile impact and provide protection for ducts and piping. However, this slab also extends vertically downward inside the reactor vessel sump and provides additional horizontal shear capacity for the concrete internal structure.

The cylindrical wall of the containment vessel is 3'-6" thick and is prestressed by post-tensioned vertical and horizontal tendons. The horizontal hoop tendons are anchored at the six equally spaced vertical buttresses and the vertical tendons are anchored at the ring girder and bottom of the base slab. The horizontal tendons are located just inside of the outer reinforcing steel as shown in Figures 2-3 and 2-4. The vertical tendons are located in two planes 1'-5" to 2'-0" apart. One plane is just inside the horizontal tendons and the other just outside the inner reinforcing steel. The vertical tendons are staggered in the circumferential direction as shown in Figure 2-11. Nominal bonded reinforcing steel is provided in the wall to distribute strains due to shrinkage and thermal effects. Figure 2-3 also shows the intersection of the wall and the foundation slab. Additional reinforcing steel and stirrups were provided at the intersection to handle the discontinuity stresses.

The configuration of the tendons in the dome is based on a three-way tendon system consisting of three groups of tendons each oriented 120 degrees with respect to each other. These tendons are anchored in the concrete ring girder at the intersection of the dome and cylindrical wall as shown in Figure 2-4. One group of tendons is located in the mid-plane of the 2'-8" thick dome. The second group is located half on the inside and half on the outside of the first group, and the third group is located half on the inside and half on the outside of group 2.

The 1/4 inch thick steel liner plate is fabricated from A442 Grade 60 carbon steel. This provides for leak tightness of the containment building and is anchored to the vessel structure by means of a horizontal and vertical steel angle grid system. The liner was designed for concrete creep and shrinkage, prestress, thermal, and internal pressure loadings.

2.2 CONCRETE INTERNAL STRUCTURE DESCRIPTION

Located within the containment vessels are the concrete internal structures. These structures are conventionally reinforced and support the reactor vessel, the steam supply system, the fuel handling pool, and the polar crane. The concrete internal structure consists of a ring wall, operating floor, fuel handling pool, and the reactor biological shield wall. The ring wall is 3'-9" thick with an outside diameter of 106 feet and extends upward from the floor slab at elevation 568 ft to the operating floor at elevation 617 ft. On the operating floor immediately above the ring wall is located the polar crane. Figure 2-5 shows a vertical section through the internals and containment vessel. Figure 2-6 shows the location of the major items of equipment including the reactor vessel, the steam generators, the reactor coolant pumps, and the polar crane.

Inside the ring wall, the reactor biological shield, which is a 8'-6" thick wall with a 34 ft outside diameter, extends downward from approximately elevation 591 ft through the base slab to elevation 542 ft. Below the base slab, the well is surrounded by massive concrete shielding as shown in Figure 2-5.

The floor of the fuel handling pool is continuous with the biological shield at elevation 591 ft., and steps down in two levels so that at the edge of the containment vessel adjacent to the fuel handling building, the floor of the fuel pool is at the same elevation as the bottom of the fuel transfer channel in the fuel handling building as shown in Figure 2-7. Short concrete walls extending upward from the base slab support the floor of the fuel pool (Figure 2-8). The vertical walls rising from the floor of the pool are continuous with the operating floor slab and form the pool boundaries.

The only location where the concrete internals are structurally connected to the containment vessel is at the base of the internal structure (Elevation 568 ft.). One foot square by 2 inch deep shear keys connect the ring wall to the 3 foot thick slab above the liner, and 1-3/8" diameter anchor bolts tie the wall into the 9 foot thick foundation slab. A detail of this connection is shown in Figure 2-9. This detail is designed to transmit loads from the internals to the foundation directly without affecting the liner.

There are three major floors inside the containment building. At elevation 568'-0" is located the 3 foot thick reinforced concrete slab poured on top of the liner and foundation slab. There is a 1 inch gap between this slab and the vessel wall which is filled with self expanding cork. The floor plan is shown in Figure 2-8. At elevations 550 ft. and 617 ft. (operating floor), the floor slabs are constructed of poured in-place concrete slab, steel grating, or concrete slab supported by steel framing. For the floor slabs located between the ring wall and the

containment wall, a 2 inch radial gap exists. Thus, the floor slabs are not designed to act as load transmitting structures between the containment wall and the concrete internal structure.

The 16 inch diameter feedwater pipes and the 34 inch diameter main steam pipes enter the reactor building at elevation 584'-7" through pipe sleeves. The pipes are enclosed in the underground pipe tunnel before entering the containment.

2.3 CONTAINMENT BUILDING FAILURE MODES

The containment building will respond to lateral seismic excitation with combined flexure and shear in the structure. Axial stresses resulting from flexure are distributed according to the first harmonic as long as the response of the structure remains linear and discontinuities such as the equipment hatches are neglected. The tangential shear through the cylindrical shell is also distributed according to the first harmonic for elastic lateral response although the location of maximum shear stress is rotated 90° from the location of maximum tension/compression stresses. Axial stresses are reacted by the concrete in compression and by the vertical prestressing tendons and reinforcing steel in tension once the preload has been overcome.

Transverse shear is reacted by both radial and tangential shear components in the shell which result from vertical reinforcing steel dowel action and concrete aggregate interlock. Axial (VQ/It) shear stress is resisted by horizontal reinforcing steel and concrete aggregate interlock. This shear must be transferred across the vertical buttress plates which extend the entire length of the cylinder.

Internal concrete structure shear loads are transferred through the shear keys under the ring wall to the 3 foot thick slab above the liner and through the liner to the foundation slab by means of the 1-3/8 inch diameter anchor bolts. Rocking response of the internals must also be resisted by the anchor bolts once the gravity forces are overcome.

Horizontal structure inertia loads are transferred to the soil by lateral soil pressure on the vertical faces of the foundation slab, tendon gallery, and reactor vessel sump, as well as friction on the horizontal surfaces. Rocking and vertical response is resisted by vertical soil pressure.

2.3.1 Foundation Failure Modes

There are a number of potential failure modes associated with degradation and failure of the soil foundation material. Because of the higher soil loading imposed by the reactor building, soil foundation failures are more likely to occur with regard to this structure than the more conventional structures such as the auxiliary and turbine buildings. These modes of failure can, in some cases, lead to loss of safe shut down capability, even though the individual structures may be essentially undamaged. This could occur in cases of large relative deformation between adjacent structures or between the structure and undisturbed soil which would lead to rupture of coolant lines or electrical control conduit. Although the emphasis of this evaluation is on structural failure modes, modes involving foundation failure are discussed briefly for completeness.

The containment structure is embedded approximately 35 feet below grade. Near the periphery of the foundation slab there is located the vertical prestressing tendon gallery with the bottom of the slab at elevation 548'-6". Near the center of the foundation slab, the reactor vessel well extends downward from elevation 565 ft. to elevation 530 ft. With this amount of lateral soil resistance together with the friction between the foundation slab and supporting soil, it is expected that sliding of the containment structure will not be a critical mode of failure. Similarly, the embedment will provide additional resistance to overturning although gaps may open between the structure and soil in the event of high levels of response. Even without the embedment, the probability of overturning is remote assuming competent foundation material. This is because the rocking period of the structure becomes

very long at large excursions compared to the earthquake forcing frequencies. Resonance cannot occur and the structure tends to become an isolated dynamic system rocking about its c.g. rather than overturning.

The rocking of the containment building may, however, cause soil failure at the toe of the foundation slab. The toe pressure is caused by the weight of the structure, the vertical force due to vertical ground motion, and the overturning moment due to horizontal ground motion. The toe pressure will increase significantly once uplift of the foundation slab occurs. After several cycles, large structure displacements can occur due to successive degradation of the soil as shown schematically in Figure 2-10. The most likely consequences of this type of failure are piping and electrical conduit failure resulting from large structure to structure or soil to structure deformations. Associated with this mode of failure are likely to be structural failures, particularly in the tendon gallery and possibly including base slab cracking around the construction joints once base slab uplift occurs. Failure of a tendon gallery wall (shown schematically in Figure 2-17) can result in a further rapid deterioration of the toe soil properties due to lack of soil confinement. Possible structure damage resulting from tendon anchor failure may occur.

Other potential failure modes such as soil liquefaction and surface faulting are also classed as foundation failure modes. Since the base slab of the containment structure is below the water table and the site includes layers of sand as well as glacial till, the potential for soil liquefaction at high ground acceleration levels needs to be considered. The possibility of surface faulting for the Zion site is expected to be negligible.

2.3.2 Structure Failure Modes

Structural failure of the containment building can occur in the containment shell, the base slab and sump, and the concrete internals. Seismic response results in high bending moment and tangential shear

loads as well as radial shear at the junction of the containment vessel wall and the base slab. Vertical response will also add to the stresses resulting from dead weight and flexure, particularly if base slab uplift due to rocking occurs. In the design verification, considerable effort is normally devoted to areas of discontinuities such as around large penetrations, the junction of the wall and base slab, and the ring girder. Although these are also critical areas when considering failure modes, the discontinuity area stresses based on elastic response are normally proportionately more critical than ultimate load evaluations where secondary stresses are less important than membrane and primary bending stresses.

The containment vessel is one structure where the addition of other dynamic loads can significantly influence the seismic capacity. If Loss of Coolant Accident (LOCA) internal pressure is present during the earthquake (or aftershock) a very substantial amount of the prestress capacity will be required to withstand the pressure loads. Consequently, a much lower strength capacity will be available to withstand the seismic loads. This is true not only for the capacity of the vertical system required to resist flexure and transverse shear but also the horizontal system. Typically the horizontal preload system does not need to resist large increases in load as the result of flexural loads. However, in Zion the circumferential preload is apparently required to transfer the VQ/I shear across the vertical buttress plates (Figures 2-11 and 2-12).

Below are summarized the more likely failure modes of the reactor building structure. Table 2-1 contains a somewhat more detailed description. The following two subsections discuss these potential failure modes in more detail for the containment vessel and internal structures respectively, together with some of the consequences.

1. Shear and diagonal tension cracks in the Containment Vessel (Fig. 2-13).
2. Crushing and spalling of concrete in the vessel wall (Fig. 2-14).
3. Yielding and failure of the reinforcing steel and loss of prestress.
4. Gross shear failure of vessel wall due to loss of dowel action and aggregate interlock (Figs. 2-14 and 2-15).
5. Axial shear failure along buttress plates (Fig. 2-12).
6. Buckling of vessel wall.
7. Shear failure in the foundation slab due to uplift.
8. Failure of tendon gallery wall and collapse of gallery (Fig. 2-17).
9. Shear failure of concrete internal structure anchor bolts at liner interface.
10. Shear failure of internal structure ring and shield walls.
11. Failure of the concrete structure enclosing the pressurizer.
12. Failure resulting from impact of adjacent structures or equipment (Fig. 2-21).

2.3.2.1 Containment Vessel Failure Modes

In the Zion containment vessels the tendons are stressed to approximately 60 to 65% of ultimate strength over the life of the structure. For low levels of seismic response, the wall will behave

essentially elastically. The concrete is effective in resisting shear and flexural tensile stress in this case. Only after the applied flexural tensile stress exceeds the prestress and the concrete cracks will the bonded reinforcing steel experience any seismic load. The increase in load in the tendons will be small due to the very small increase in strain compared to the preload strain. This occurs because the strain resulting from a crack width is distributed over the length of the unbonded tendon. As the load is increased and the cracks widen, yielding will occur in the reinforcing steel and liner. When the inertia loads are reversed buckling of the reinforcing steel and liner can occur and failure of the liner integrity can result since the steel alone must resist the compressive forces (Figure 2-14). Local spalling of the concrete outside of the reinforcing steel will result in loss of confinement for the steel and accentuate the failure as shown in Figure 2-15.

The strain at which concrete crushing occurs is normally in the range of 0.003 to 0.004. The design strength of the concrete for the Zion containment vessels is 5,500 psi. Crushing of the concrete can lead to the following consequences:

1. Reduction of the dowel action capacity of the reinforcing steel to resist shear.
2. Buckling of the reinforcing steel.
3. Loss of prestress.

Lateral seismic loading on the containment vessel creates both in-plane (tangential) and out-of-plane (radial) shear in the containment wall. The out-of-plane shear stress is resisted by stirrups placed in the wall close to the wall-base slab junction. The in-plane shear stress in the Zion vessel is designed to be primarily resisted by the prestressed concrete and vertical wall reinforcing.

The tangential shear distribution is distributed in the cylindrical shell according to the first harmonic for elastic lateral loads. As inelastic response levels are reached, the distribution changes. This shear "yielding" occurs due to reduction in dowel stiffness and loss of aggregate interlock as the cracks widen. Any loss of prestress will result in a significant reduction of shear resistance capacity since only the gravity and vertical response loads are available for aggregate "friction". The tangential shear must then be resisted to a larger extent by the bonded reinforcing steel. The dowel action of the reinforcing steel depends on whether the concrete can confine the steel bars. Failure of dowel action can result from either crushing of the concrete or bond splitting along the bar (Fig. 2-15). Initial consequences of shear type failure will be potential failure of the liner (Figure 2-16) and pipes. As the deformation is increased, eventually the point is reached where vertical collapse of part or all of the structure can occur.

Buckling of the vessel wall is considered an unlikely mode of failure since the 3'-6" wall thickness provides a diameter to thickness ratio which will tolerate large amounts of eccentricity and still provide for critical stress above the crushing strength of the concrete. However, if substantial amounts of concrete outside of the reinforcing steel radii should spall off due to shear, local buckling could conceivably occur before the crushing stress of the remaining concrete core is reached.

At response levels where uplift of the base slab occurs, the load distribution under the slab changes significantly. Although the containment vessel shell and concrete internals will provide substantial stiffening for the base slab provided they are intact, the possibility of base slab shear failure, probably across a construction joint, exists for very high levels of response. This would lead to loss of liner integrity and lack of support for both the vessel and internals with eventual structural collapse.

Collapse of a tendon gallery wall is also a potential mode of failure. This can occur either across the construction joint at the interface with the base slab or at some other location as shown in Figure 2-17. This type of failure can occur due to the increased lateral soil pressure resulting both from the loads induced by swaying of the structure as well as the increased vertical toe pressure in the soil due to rocking, particularly if base slab uplift occurs. The consequences of this mode of failure are that soil degradation is greatly increased with the potential for pipe failure, etc., as previously discussed and structural failure due to failure of the vertical prestress tendons can occur.

2.3.2.2 Concrete Internal Structure Failure Modes

The concrete internal structures of the Zion Station containment consists of a ring wall, the reactor biological shield wall, the fuel handling pool, and the operating floor. The reactor coolant system which consists of the reactor vessel, the steam generators, the pressurizers, and the reactor coolant pumps are located within the ring wall and laterally supported by the ring wall and the shield wall. Figure 2-18 shows the location of the major equipment items. The polar crane is also supported by the ring wall.

A major structural failure of the concrete internal structure could lead to a total failure of the reactor coolant system due to loss of support for major components or missile impact on the coolant system with consequent failure of the pressure boundary. Thus, attention is focused on the failure of any structural elements of the concrete internal structures which could lead to such an event.

The ring wall and shield wall were constructed of reinforced concrete with wall reinforcing in two perpendicular directions. No helical seismic shear reinforcing was observed from the structural drawings. Thus, the seismic induced shear will be resisted by aggregate interlocking of the concrete and drawl action of the wall reinforcing

steel across any concrete cracks which form. Although possible, it is judged that flexural failure will not be a controlling failure mode for the ring wall and shield wall because of the low height to diameter ratio. Rather, it is expected that the controlling seismic failure mode for the internal structure will be shear failure, especially since the internals do not bear against the containment wall.

The shear transfer path between the concrete internal structure walls and the foundation slab is shown in Figure 2-9. There are 12" x 12" x 2" deep keys at 2'-0" on center at the base of the concrete wall. Shear forces from the internal structures are transferred through these keys to the 3-ft. base slab. At higher ground accelerations, the keys may be sheared off. However, the dowels and vertical wall reinforcing could still remain effective in providing a clamping force for shear friction. As it was pointed out earlier in Section 2-2 describing the internal structure, a 1-inch gap filled with cork isolates the base slab from the containment vessel wall. Thus, the shear force from the 3-ft. thick slab must be transferred to the 9-ft. thick foundation slab through the 1-3/8-inch diameter dowels, friction, and the shear capacity of the vertical (1-ft. thick) continuation of the 3-ft. thick slab at the sump. The possible failure mode is the shear failure of the weld or 1-3/8-inch diameter dowels at the interface of the 3-ft. slab and 9-ft. slab and shear failure of the vertical portion of the 3-ft. thick slab in the sump. This will result in loss of liner integrity and possibly pipe and conduit failure.

A reinforced concrete structure at the operating floor encloses a portion of the pressurizer which is above the operating floor (Fig. 2-19). The pressurizer enclosure has 1 foot thick poured-in-place concrete walls on three sides. The walls are approximately 39-ft. tall. The fourth wall consists of several pieces of removable concrete panels. The roof is constructed of a 1-foot thick removable concrete slab bolted down to the two walls which are perpendicular to the roof slab span (Fig. 2-20). No diaphragm action is provided by the roof slab due to lack of

roof connection to the other two walls and the discontinuity at the center of the roof slab. Because of the open section considerable torsional response is expected. The failure mode of the wall will be mainly due to yielding of the wall reinforcing and eventual collapse. Damage of the pressurizer, steam generator and feedwater piping could result from the collapse of these walls and the roof.

2.3.2.3 Impact Failure Modes

Finally, impact between the containment vessel and the concrete internals, the containment vessel and the auxiliary building, or the containment vessel and the polar crane can lead to loss of integrity of the liner and provide for initiation of structural failure. Although the liner will likely constrain any fractured chunks of concrete which could act as missiles within the containment, the liner may be breached by the impact. If the impact is severe, some reinforcing steel or prestress tendons may be lost and the failure may propagate from the local impact area by several of the means previously described. Figure 2-21 shows some of the likely impact areas on the containment vessel.

Structural Element Description	Structural Response Description	Damage
Longitudinal guide walls	Combined lateral and torsional response due to horizontal ground motions	Tension yielding of the vertical reinforcing and crush of the extreme concrete at the base of the wall. Reduction in the shear capacity, eventual vertical collapse.
Operating floor concrete slab	Combined lateral and torsional response due to horizontal ground motions	Concrete slab shear failure at the junction of the floor slab and foundation walls - transfer of load to out-of-plane bending and eventual collapse
Service water pump enclosure:	Response to the E-W direction ground motions	Loss of roof diaphragm; loss of roof level lateral support of the east and west side walls collapse of walls and roof and subsequent pump damage
a. Concrete roof diaphragm		a. Shear failure at the junction of the roof slab and end shear walls; shear failure of the roof slab at a section where roof opening is located; tension yielding of the steel rebars at the flange of the roof diaphragm
b. North and south end walls		b. Flexural failure; shear failure across the construction joint; shear failure across the plastic hinge zone
c. East and west side walls	Response to the E-W ground motion	c. Formation of hinge at the base of the wall due to out-of-plane bending
Vertical concrete walls at the intake end of the crib house	Response to the E-W ground motion	Formation of hinge at the base of the walls due to out-of-plane bending; collapse of these two walls results in the partial collapse of the north and south foundation walls near the intake end possible flow blockage
Horizontal reinforced concrete strut (N-S direction) at the open area of the crib house	Response to concurrent horizontal and vertical ground motions	Exceedence of the ultimate compressive capacity of the strut under the interaction of axial compression and biaxial bending. Failure of the strut can lead to collapse of the north and south side foundation walls at the open area.
48-inch diameter underground service water pipes	Response to seismic wave action or relative displacement at the anchor points	Tensile or shear failure or buckling of the pipes
Aboveground structures:		
a. Masonry block walls	a. Response to ground motion perpendicular to its plane	a. Cracking at the base of the masonry walls; rigid body rocking of the wall and collapse
b. Roof top trolley supporting steel frame	Response to the E-W ground motion	b. Collapse of the frame due to lack of lateral bracing
		No damage to the safety-related equipment is likely to result from the above ground structures failure

TABLE 2-1. POTENTIAL CONTAINMENT BUILDING STRUCTURAL FAILURE MODES

FIGURE 2-1. SECTION OF REACTOR CONTAINMENT BUILDING

FIGURE 2-2. CONCRETE POUR SEQUENCE OF THE FOUNDATION SLABS

FIGURE 2-4. TYPICAL DETAIL AT THE INTERSECTION OF THE CONTAINMENT WALL AND DOME

FIGURE 2-5. ZION REACTOR BUILDING AND INTERNAL STRUCTURES

FIGURE 2-6. E-W SECTION OF CONTAINMENT BUILDING

FIGURE 2-7. PLAN OF THE FUEL HANDLING POOL

FIGURE 2-8. FLOOR PLAN AT ELEVATION 568 FEET
(SECTION A-A OF FIGURE 2-5)

FIGURE 2-9. CONNECTION DETAIL BETWEEN CONCRETE INTERNAL STRUCTURES AND CONTAINMENT STRUCTURE FOUNDATION SLAB

FIGURE 2-10. FAILURE OF SOIL AND PIPING RESULTED FROM
CONTAINMENT BUILDING ROCKING RESPONSE

FIGURE 2-11. SECTION OF CONTAINMENT WALL BUTTRESS

FIGURE 2-12. CONTAINMENT WALL SHEAR FAILURE ALONG THE BUTTRESS PLATE

FIGURE 2-13. SHEAR AND DIAGONAL TENSION CRACK AT THE BASE OF THE CONTAINMENT WALL

FIGURE 2-14. BUCKLING OF CONTAINMENT WALL REINFORCING FROM REVERSED CYCLIC LOADING

NOTE: HORIZONTAL WALL REINFORCING AND PRESTRESSING
TENDONS ARE NOT SHOWN HERE

FIGURE 2-15. CONCRETE SPLITTING ALONG VERTICAL WALL REINFORCING
(DOWEL) AXIS UNDER TANGENTIAL SHEAR

NOTE: HORIZONTAL WALL REINFORCING AND PRESTRESSING
TENDONS ARE NOT SHOWN HERE

FIGURE 2-16. DAMAGE TO CONTAINMENT WALL AND LINER PLATE DUE
TO SLIDING ACROSS HORIZONTAL CRACK

FIGURE 2-17. FAILURE MODE OF TENDON GALLERY WALL

FIGURE 2-18. LAYOUT OF CONTAINMENT
COOLANT SYSTEM AT
OPERATING FLOOR

FIGURE 2-19. N-S SECTION OF CONTAINMENT BUILDING

PRESSURIZER ENCLOSURE ROOF

SECTION A-A

SECTION B-B

FIGURE 2-20. PLAN AND SECTIONS OF PRESSURIZER ENCLOSURE

FIGURE 2-21. CONTAINMENT BUILDING IMPACT FAILURE MODES

3. AUXILIARY/TURBINE BUILDING

The turbine/auxiliary building complex of the Zion station consists of the following buildings: the turbine building, the auxiliary building, the fuel handling building and the diesel generator rooms. All four buildings are structurally interconnected at different levels through roofs and floor slab. The general layout of the complex is given in Figure 3-1. The roof elevations of all four buildings are shown in Figure 3-2. It is observed that the complex is nearly symmetric about the east-west axis but is highly unsymmetric about the north-south axis. The reactor containment buildings of Unit 1 and Unit 2 are located symmetrically on each side of the fuel handling building but are not structurally coupled to the adjacent structure.

Among these four buildings, the auxiliary building, the diesel generator rooms, and the fuel handling building are Class I structures. The turbine building is non-Class I. All buildings are founded on either reinforced concrete foundation slabs or spread footings. The lateral force resisting systems used are structural steel braced frames and reinforced concrete shear wall systems. All Class I buildings in the complex have the latter (shear wall) lateral force resisting system. Detail descriptions of the two major buildings in the complex, i.e., the turbine building and the auxiliary building are given in the following sections.

3.1 TURBINE BUILDING DESCRIPTION

The turbine building, a 678 ft by 130 ft structure, is symmetrical about an approximate east-west centerline. The south part of the building houses the turbine of Unit 1 and the north part of the building houses the turbine of Unit 2. Most of the turbine building (i.e., turbine and condenser supporting structures) is founded on a reinforced concrete foundation mat with varying thickness (Figure 3-3).

The top of the foundation mat elevation is at approximately elevation 560 ft. The remainder of the turbine building which is not located over the foundation mat is supported by concrete columns which extend downward to the spread footings. Centrally located in the turbine building below the turbines are the condenser well slabs at elevation 560 ft. A 14-ft thick continuous slab is located between the condenser well slabs of Unit 1 and Unit 2 (Figure 3-3). At the west edge of the condenser well slabs, a 3-ft 4-in thick continuous slab connects the foundation mat to the floor slab at elevation 560 ft in the auxiliary building (Figure 3-4).

Rectangular reinforced concrete piers rise from the top of the condenser well slabs to elevation 592 ft where continuous haunched girders connect the piers along each side of each condenser well. The turbine foundations are massive reinforced concrete space frames which are continuous with the piers of the condenser wells and rise from elevation 592 ft to the main floor of the turbine building at elevation 642 ft. The turbine foundations are isolated from the major turbine building floors at elevations 617 ft and 642 ft by a one-inch gap (Figure 3-5).

The ground floor, a 3-ft thick reinforced concrete slab, is continuous with the floor slab at the same elevation (elevation 592 ft) in the auxiliary building (Figure 3-4). At elevations 617 ft and 642 ft, the floors were constructed of poured-in-place concrete slabs supported by vertical and horizontal braced steel framing. A typical floor slab support detail is shown in Figure 3-10. The slabs are continuous, through the steel floor framings and concrete slabs, with the floor slabs at the same elevations in the auxiliary building (Figure 3-6). Two 110 ton capacity cranes are located at elevation 689 ft 10 in (one for each turbine). The cranes run in the north-south direction. A typical transverse section of the turbine building is shown in Figure 3-7.

The roof was constructed of 3-1/2 inch thick precast concrete channel slabs covered with 1-inch rigid insulation and is supported by braced steel roof framing. The elevation of the main roof over the turbines is 712 ft and the roof elevations of the heater bay are 666 ft and 642 ft as shown in Figure 3-2. The roof framing consists of steel roof girders, wide flange roof beams, and double angle diagonal bracings. A minimum of three 7/8-inch diameter bolts and 3/8-inch thick gusset plates were used for the connections of the diagonal bracings. No detail information was available for the connections between the individual precast roof slabs and between the roof slabs and steel framing. It is assumed that only nominal metal clips were used to restrain the roof slabs against uplift due to wind.

The vertical lateral force resisting systems of the turbine building are the steel braced frames along all four sides of the building. Schedule 40 pipes were used as diagonal bracing elements for the braced frames. The braced frames at the north and south end of the turbine building are shown in Figures 3-8 and 3-9. Fluted metal sidings were attached to the girt system of each vertical braced frame to enclose the turbine building. No detail information was available concerning the connection details and resulting rigidity of this siding. A portion of the west side vertical braced frame along column Line G, located between the turbine building and the auxiliary building and diesel generator rooms, is encased in reinforced concrete walls (Figure 3-10) from ground level up to the auxiliary building roof level at elevation 668 ft. The wall above the auxiliary building roof level is again constructed of fluted metal sidings (Figure 3-11).

3.2 AUXILIARY BUILDING DESCRIPTION

The tee-shaped auxiliary and fuel handling building is located west of and is structurally continuous with the turbine building. A 24-inch thick reinforced concrete common wall is located between the two buildings from ground floor up to the auxiliary building roof level (Figures 3-10 and 3-11). Structural connectivity between the two

buildings is provided by continuous floor slabs at various levels. The diesel generator rooms are an integral part of the structural complex. Both the auxiliary-fuel handling building and the diesel generator rooms are Class I structures.

The auxiliary building is founded on a 5-ft thick soil-supported reinforced concrete mat at elevation 542 ft. The grade elevation is 591 ft. A reinforced concrete mat foundation was also used for the fuel handling building. The elevation of the mat at the spent fuel pit is 576 ft and the rest of the fuel handling building foundation mat is at the grade level (Figure 3-12). The diesel generator rooms at the north and south ends of the auxiliary building are founded on walls extending to a strip footing at elevation 557 ft 4 in. Reinforced concrete foundation walls of the auxiliary building are laterally supported by concrete floor slabs at elevations 560 ft, 579 ft and 592 ft. The floors are reinforced concrete slabs supported by concrete beams, columns, and foundation walls.

Above grade, the lateral force resisting system is a combination of braced structural steel frames and concrete slabs and walls. Vertical braced steel frames were erected on foundation walls around the periphery of the auxiliary-fuel handling building and diesel generator rooms. Various diameter steel pipe was used for the diagonal bracing. A typical connection detail is shown in Figure 3-13. The entire vertical braced frames were then encased in reinforced concrete walls which form the shear wall system. The floors at elevations 617 ft, 630 ft, and 642 ft are reinforced concrete slabs supported by horizontal braced steel framing. At places where heavy floor loads were expected, shear studs were used at the top flange of the steel floor beams to achieve a composite action. Figure 3-14 indicates the location and thickness of the exterior shear walls of the auxiliary building. The roofs of the auxiliary building and diesel generator rooms were constructed of a poured concrete slab supported by braced steel roof framing at elevations 668 ft and 658 ft.

The fuel handling building houses the spent fuel pit which is a rectangular concrete tank which extends from elevation 576'-0" to elevation 617'-0". Reinforced concrete walls rise on the three exterior sides from elevations 576 ft and 592 ft to the roof at elevation 663'-11-1/2". There are partial floor slabs at elevation 602'-0" and 617'-0" in the Fuel Handling Building (Figure 3-12). A discontinuity between the Auxiliary Building and the Fuel Handling Building is formed by the fuel transfer channel (Figure 3-15). The only structural connection is at the base slab and the roof level (Figure 3-16). The roof of the Fuel Handling Building was constructed of 4-1/2" deep corrugated metal decking with 12-in concrete slab supported by steel roof framing (Figure 3-17).

At several locations there is only a 1-inch gap between the auxiliary-fuel handling building and the reactor containment structure. Three typical places are: a) roof of the purge system room at elevation 642 ft and roof of the cable penetration vault at elevation 643 ft (Figure 3-18), b) the floor slab of the pipe chase at elevation 568 ft (Figure 3-19), and c) between the containment shell and the fuel transfer channel wall of the fuel handling building. The potential impact between the two structures was discussed in the containment building section.

Locations of some of the safety-related equipment and piping in the turbine/auxiliary complex are given from Figures 3-20 to 3-23. These important equipment and piping are identified as: safety injection pumps, containment spray pumps, diesel generator oil storage tanks, feedwater pump seal water collection tank, motor driven and turbine driven steam generator feedwater pumps, auxiliary motor driven and turbine driven steam generator feedwater pumps, diesel generators, and condensate storage tanks. These locations are noted in relation to their proximity to structural walls.

3.3 FAILURE MODES OF THE TURBINE BUILDING

The turbine building, auxiliary building, fuel handling building and diesel generator buildings form a single combined structure. Thus, failure of one part of the structure, while not necessarily resulting in failure of the entire complex, will at least influence the dynamic response characteristics of the overall building. However, the individual parts of the overall structure such as the turbine building are constructed to different codes and standards and different construction methods and hence, may be expected to have different modes of failure.

Some of the soil failure modes discussed previously for the containment structure also apply to the turbine/auxiliary building. However, since the soil loadings developed for the containment building are much greater than those for the turbine/auxiliary building, soil failure under the containment structure would be expected to occur considerably before similar failure under the turbine/auxiliary building.

Below is listed a summary of the more likely potential failure modes for the combined turbine/auxiliary building. Table 3-1 provides a somewhat more detailed listing together with causes and effects. The following two subsections contain the discussion for the turbine and auxiliary/fuel handling building.

1. Failure of turbine building roof bracing system (element, gusset plate, or bolt failure) - transfer of inertia loads to out-of-plane wall.
2. Yielding or buckling of columns between the turbine and auxiliary building - collapse of roof (Fig. 3-26).
3. Loss of the turbine building vertical braced frame systems (element, gusset plate, or bolt failure) - column buckling and collapse.

4. Column anchor bolt failure under combined shear and tension - loss of lateral load transfer capacity.
5. Auxiliary building concrete slab roof diaphragm failure (shear failure of the slab or failure of shear transfer to collector beams).
6. Auxiliary building roof truss failure (shear failure of bolts, member, or gusset plate failure).
7. Failure of composite wall between turbine and auxiliary building (failure of shear studs or crushing of concrete) - transfer of load to braced frame.
8. Auxiliary building shear wall failures (shear failure across a construction joint, shear failure across a plastic hinge joint, flexural failure) - transfer of load to braced frame (Fig. 3-29).
9. Auxiliary building vertical braced frame failure (shear failure of bolts, gusset plate failure, element failure) - loss of lateral support and eventual collapse.
10. Plastic hinge of roof girder - partial collapse of roof.
11. Out-of-plane bending and collapse of one-foot-thick walls around control room and other critical equipment.

There are two horizontal lateral force resisting systems in the turbine building roof which are effective in collecting and transmitting lateral inertia forces to the vertical shear resisting systems. The first system consists of the precast concrete channel slabs. The second system is the braced steel roof truss. No specific connection details were called out in the structural drawings between each individual slab

and between slabs and vertical lateral force resisting systems. Thus, no positive connection of the roof channel slabs is assumed. This means that the roof inertia force can be collected and transferred to the vertical resisting systems by the roof channel slabs only through the friction forces developed between the channel slabs and supporting steel members. The channel slabs span in the east-west direction. Thus, under the east-west direction ground excitations, only half of a channel slab weight is effective in producing friction forces and resulting couples to transfer the roof inertia force to the end vertical braced frames (Fig.3-24). It is therefore expected that the diaphragm capacity of the first horizontal force resisting system is very low and sliding between adjacent concrete channel slabs and between the slabs and roof beams will occur at a very low ground acceleration.

The second system, i.e., the roof braced frame, will resist the roof inertia force as soon as sliding occurs in the first system. The steel roof framing system consists of roof girders, roof beams, and double angle diagonal bracing members. Due to the high aspect ratio (approximately 5) of the turbine building, the roof frame is expected to be quite flexible. There are several possible load paths for the turbine building roof inertia forces to be transferred down to the foundation as a result of the east-west direction ground excitations. First of all, part of the roof inertia force is resisted by the vertical steel braced frames at the north and south ends of the turbine building. Because there are a large number of east-west direction reinforced concrete shear walls and braced steel frames located between the two end steel braced frames at lower elevations, the lower portion of the structure is expected to be quite stiff. Consequently, part of the turbine roof inertia force under the east-west ground excitation will be resisted by the turbine building columns (30WF172). The columns will resist the lateral force in bending and behave as cantilever beams with bases fixed at the top of the various shear resisting systems (i.e., shear walls and braced frames) in the east-west direction. Under the north-south ground

motions, the turbine building roof inertia force is resisted by the steel braced frame and concrete shear wall at the west side and by the steel braced frame alone at the east side.

Connection failure or buckling of the diagonal bracing members of the roof frame are judged to be possible failure modes. The connection can fail due to tearing of the gusset plate or shear failure of the connection bolts or welds. The loss of the roof diaphragm action will then lead to the failure of the building columns and partial collapse of the roof slab of the turbine building due to the $P-\Delta$ effect of the columns (Figures 3-25 and 3-26). However, damage will probably be limited to equipment located above the operating floor or the next lower floor under any open hatches.

Due to the eccentricity between the center of mass and center of rigidity in the N-S direction of the turbine/auxiliary complex, a significant amount of torsion response is expected to result from the N-S ground motions. As a result of the concurrent ground motions in the E-W and N-S direction, the braced frames at the ends of the turbine building will be subject to high shear forces. The frames could fail in buckling of the braces, connection failure of the bracing, buckling of the columns, or column anchor bolt failure. The $P-\Delta$ effect on those columns near the ends of the building is worsened by the torsional response of the complex. Partial to total collapse of the roof system is the likely consequence of this mode of failure.

The lower elevations of the turbine building are expected to fail as a result of shear wall failures. Failures of this type are discussed in more detail in the following section on the auxiliary building. The relative capacities between the auxiliary building and the (non-Class I) turbine building or between lower (concrete shear wall construction) elevations and the upper (braced frame and metal siding) elevation of the turbine building are not currently known. It is possible that the upper elevation of the turbine building could collapse

at response levels lower than those which would cause failure of the remainder of the structure. Although this would drastically alter the dynamic response characteristics of the structure, it would not necessarily lead to failure of critical equipment. Similarly, although impact between the turbine pedestal and the rest of the structure may occur with resulting local damage, safety related equipment will not necessarily be affected.

3.4 FAILURE MODES OF THE AUXILIARY BUILDING

The roof of the tee-shaped building is a 21-inch thick reinforced concrete slab supported by braced steel roof framing. Two horizontal lateral force resisting systems are thus available at the auxiliary building roof. The more rigid concrete roof slab acts as a diaphragm in transmitting the lateral inertial forces to the shear walls. The possible failure mode of this diaphragm is the shear failure at the junction of the roof slab and the shear wall. Substantial reduction in the structure system stiffness and loss of strength will result. However, the braced roof frame will then resist the inertia force after the rigid roof slab fails in shear although considerably greater deformation and storey drift will occur. Buckling of the double angle diagonal braces and connection failure of the braces are the possible failure modes of this braced roof frame. Loss of the roof diaphragm results in the requirement that the concrete walls resist the lateral inertia force in transverse bending. This capacity is expected to be relatively low. Failure of the reinforced concrete walls in bending about the weak axis then leads to the collapse of the roof. The control room at the floor immediately below (elevation 642 ft) will be severely damaged by the collapsed roof.

Because of the irregular shape of the auxiliary building and the layout of the shear walls, two collector beams were used in the roof framing to collect and transfer the roof inertia loads to the shear walls at the north and south sides of the auxiliary-fuel handling building (Figure 3-27). The in-plane inertia forces in the roof diaphragm are

transferred to the collector beams through shear. However, no shear connections are observed between the concrete roof slab and the collector beams. Thus, friction appears to be the only means of transferring the shear. No connection details at the ends of the collector beams are available. Failure of the end connections or slippage between the concrete roof slab and the collector beams would result in local tearing of the roof diaphragm near the end of the beams. The inertia forces of the roof over the control room area will then be resisted by the 12-inch thick concrete shear walls enclosing the control room after the loss of the load path described above. The control room shear walls are supported on wide flange beams at elevation 642 ft. The dimensions of the wall are 26 ft high by 50 ft long. Possible failure modes of the shear walls are discussed below.

The auxiliary building walls were constructed of structural steel braced frames encased in reinforced concrete walls. Typical wall details are shown in Figure 3-28. A section of the wall between the auxiliary and turbine buildings is shown in Figure 3-10. The shear studs in the figure were used to transfer VQ/I type shear between the concrete walls on each side of the wide flange columns. Upon failure of this type shear transfer due to either studs failing in shear, welding failure at the studs, or crushing of the concrete surrounding the studs, the shear wall between the turbine and auxiliary buildings will then behave as several panels of shear walls separated by steel columns. This will change the dynamic behavior of the structure due to some reduction in stiffness and an increase in damping.

For the structural steel braced frames encased in the concrete walls, no shear connectors nor reliable bond between the steel members and concrete is observed. Thus, instead of having a composite shear resisting system, a redundant vertical shear resisting system is assumed. The lateral inertia forces are initially resisted by the stiff concrete shear walls. Several failure modes of the reinforced concrete shear wall are discussed below.

- a. Flexural failure mode: The vertical wall reinforcing at the extreme fiber of the shear wall may be subjected to large tensile forces above the yield. This implies that after load reversal compression yielding must occur before a previously formed large crack can close and the concrete can again carry compression. Due to lack of horizontal stirrups at the extreme faces of the wall, the extreme fiber rebars could buckle under compression (Figure 3-29).
- b. Shear failure across a horizontal construction joint: The shear force across a horizontal construction joint can be resisted by aggregate interlocking (interface shear transfer) and dowel action of the vertical wall reinforcing bars. The aggregate interlocking depends heavily on the initial crack width and the degree of restraint that prevents further opening of the crack. Failure of this shear transfer mechanism occurs when the wall reinforcement across the crack starts to yield. Dowel action of the rebars resists the shear force by direct shear on the rebars accompanied by bending. Failure of the dowel action particularly from large diameter rebars without sufficient lateral restraint, occurs when the concrete splits along the reinforcement axis.
- c. Shear failure across plastic hinge zone: As a result of reversed cyclic loading and the residual plastic strain in the flexural reinforcement of the wall, large cracks may form across the entire width of the wall (Figure 3-29). The interlock shear transfer mechanism is greatly diminished due to the large cracks. A sliding shear failure may occur across the precracked compression zone. Excessive shear stress may occur across the relatively small compression zone. Dowel action of the vertical wall reinforcement will be mobilized only after a substantial slip occurs.

The encased vertical steel braced frames will resist the lateral inertia force after the gross concrete shear wall failure occurs. The ductility of the braced frame system will be considerably greater than the concrete if non-ductile failure modes such as bolt failure do not control. The system damping will possibly be higher due to energy dissipation of the cracked concrete. Because of the concrete and wall reinforcing encasing the steel frames, buckling of the diagonal bracing members may also be restrained. Possible failure modes would then likely be limited to connection failure and column anchor bolts failure or tensile failure of the brace element itself. Failure of the vertical braced frame system will then lead to the total failure of the auxiliary building due to essentially in-place vertical collapse due to gravity and vertical seismic loads. In addition to causing failure through lack of lateral support, failure of the shear walls which make up the 400,000 gallon refueling water storage tanks can cause loss of the water with possible flooding of adjacent electrical equipment on the tank level (Elevation 579') or below.

ROOF GIRDER IN VERTICAL BENDING

The collector beams of the auxiliary building roof system discussed above are built-up roof girders spanning 50 ft. Under concurrent horizontal and vertical ground motions, the beam is subjected to a significant amount of axial compression or tension and amplified vertical bending. Failure of the roof girders may lead to the partial collapse of the roof system and damage to the control room equipment. The likely failure mode is the forming of a plastic hinge at the mid-span.

OUT-OF-PLANE WALL BENDING

The control room in the auxiliary building is located at the operating floor and is enclosed by 1-ft thick reinforced concrete walls. The walls span from the floor at elevation 652 ft to the roof slab at elevation 668 ft. The transverse bending of the walls is of concern as the failure of these walls may damage the control room equipment. Out-of-plane bending and collapse may also occur at other locations as shown in Figures 3-20 through 3-22 although these locations are judged less likely than the area of the control room.

Structural Element Description	Structural Response Description	Damage
Reactor containment building wall	Response to horizontal ground motion	Shear and diagonal tension cracks near the wall base - potential liner damage
Containment wall - concrete	Response to horizontal ground motion and vertical seismic plus dead weight loads	Crushing and spalling of concrete in flexural response; reduction in dowel action capacity of the rebars; loss of confinement of the rebars; loss of prestress; - liner damage, structural collapse
Containment wall - bonded reinforcing steels	Response to horizontal ground motion	Tension yielding and compression buckling at the reversed cyclic loading; loss of prestress in the vertical prestressing tendons - liner damage, structural collapse
Containment wall in shear after loss of prestress	Response to horizontal ground motion	Loss of aggregate interlocking of concrete and dowel action of rebars in resisting tangential shear - potential failure of liner and pipes may result and partial or total collapse is possible
Axial (VQ/I) shear transfer along buttress plate	Response to horizontal ground motion	Vertical shear failure along the entire length of the buttress plate - liner damage
Concrete foundation slab	Rocking response to horizontal ground motion	Shear key failure at the construction joint - loss of liner integrity and lack of support for reactor vessel and internal structures with eventual structural collapse
Prestressing tendon gallery	Response to horizontal ground motion	Shear key failure at the construction joint or formation of a plastic hinge due to lateral soil pressure; soil degradation increases the potential for pipe failure; possible failure of vertical prestressing tendon anchorages - structural collapse
Concrete internal structure shear anchors	Response to horizontal ground motion	Sliding of internals at liner interface - lack of support for equipment, piping failure, piping and equipment failure
Ring and shield walls	Response to horizontal ground motion	Loss of aggregate interlocking of concrete and dowel action of rebars in resisting tangential shear; partial or total collapse of internal structure is possible
Concrete enclosure of the pressurizer	Response to horizontal ground motion	Plastic hinge forms at the base of the wall; collapse of wall may damage the pressurizer or piping
Reactor containment building wall	Response to horizontal ground motion induced impact	Damage to containment liner, wall reinforcing and prestressing tendons due to impacting from concrete internal structures, polar crane, or the auxiliary building

TABLE 3-7. POTENTIAL TURBINE/AUXILIARY BUILDING STRUCTURAL FAILURE MODES

FIGURE 3-1. ZION STATION PLAN VIEW

FIGURE 3-2. ROOF PLAN OF TURBINE-AUXILIARY BUILDING COMPLEX

FIGURE 3-3. LONGITUDINAL SECTION OF THE TURBINE BUILDING FOUNDATION MAT

FIGURE 3-4. STRUCTURAL CONNECTIVITY BETWEEN THE TURBINE AND AUXILIARY BUILDING AT EL. 560'-0" AND 592'-0"

FIGURE 3-5. ISOLATION OF THE TURBINE FOUNDATION (CONCRETE FRAME)

FIGURE 3-6. STRUCTURAL CONNECTIVITY BETWEEN THE TURBINE AND AUXILIARY BUILDING AT ELEVATIONS 617'-0" AND 630'-0"

FIGURE 3-7. TRANSVERSE SECTION OF THE TURBINE BUILDING

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FIGURE 3-9. STRUCTURAL STEEL BRACED FRAME AT THE SOUTH END OF THE TURBINE BUILDING

STEEL BRACED FRAME AND SHEAR WALL ALONG LINE ①

TYPICAL CONCRETE FLOOR CONSTRUCTION

FIGURE 3-10. STRUCTURAL DETAIL OF THE TURBINE BUILDING WEST WALL AND TYPICAL CONCRETE FLOOR SLAB

FIGURE 3-11. DETAIL OF TURBINE BUILDING WALL ABOVE THE AUXILIARY BUILDING ROOF LEVEL

FIGURE 3-12. LONGITUDINAL SECTION OF THE FUEL HANDLING BUILDING

TYP. CONN. FOR
BRACING TO
COL. WEB

TYP. CONN. FOR
BRACING TO
COL. FLANGE

TYP. CONN. FOR
BEAM

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FIGURE 3-13. CONNECTION DETAIL OF THE DIAGONAL BRACINGS OF THE STEEL BRACED FRAMES

* STEEL BRACED FRAME FROM ELEV. 668'-0" UP TO TURBINE BUILDING ROOF

FIGURE 3-14. SHEAR WALLS AND BRACED FRAMES ALONG THE PERIMETER OF THE TURBINE-AUXILIARY BUILDING COMPLEX

FIGURE 3-15. AUXILIARY-FUEL HANDLING BUILDING AT ELEVATION 592'-0"

FIGURE 3-16. AUXILIARY-FUEL HANDLING BUILDING AT THE ROOF LEVEL

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FIGURE 3-17. DETAIL OF THE FUEL HANDLING BUILDING ROOF SLAB

FIGURE 3-18. SEPARATION BETWEEN REACTOR CONTAINMENT WALL AND ROOF
AT ELEVATION 643'-0"

FIGURE 3-19. SEPARATION BETWEEN PIPE CHASE FLOOR AT ELEVATION 568 FT. AND CONTAINMENT WALL

FIGURE 3-20. LOCATION OF SAFETY RELATED EQUIPMENTS AT FLOOR ELEVATION 542 FT. OF THE AUXILIARY BUILDING

FIGURE 3-21. LOCATION OF SAFETY RELATED EQUIPMENTS AT FLOOR ELEVATION 560 FT.
OF THE AUXILIARY BUILDING

FIGURE 3-22. LOCATION OF SAFETY RELATED EQUIPMENTS AT FLOOR ELEVATION 579 FT.
OF THE AUXILIARY BUILDING

FIGURE 3-23. LOCATION OF SAFETY RELATED EQUIPMENTS AT FLOOR ELEVATION 617'-0" OF THE TURBINE BUILDING

FIGURE 3-24. TURBINE BUILDING ROOF DETAIL AT THE END BRACED STEEL FRAMES

LOW GROUND ACCELERATIONS

HIGH GROUND ACCELERATION, ROOF TRUSS OR END BRACED FRAMES FAIL

FIGURE 3-25. RESPONSE OF TURBINE BUILDING ROOF TRUSS TO THE E-W GROUND MOTION

FIGURE 3-26. RESPONSE OF TURBINE BUILDING COLUMNS TO THE E-W GROUND MOTION

FIGURE 3-27. COLLECTOR BEAMS IN THE AUXILIARY BUILDING ROOF FRAMING.

DETAIL A

DETAIL B

FIGURE 3-28. TYPICAL WALL REINFORCING NEAR THE STEEL COLUMNS OF THE AUXILIARY BUILDING

FLEXURAL FAILURE MODE

SLIDING ACROSS CONSTRUCTION JOINT

SLIDING ACROSS A PLASTIC HINGE ZONE

FIGURE 3-29. FAILURE MODES OF A TALL SHEAR WALL

4. CRIB HOUSE (INTAKE STRUCTURE)

4.1 CRIB HOUSE DESCRIPTION

The crib house of the Zion Station is a partially open box-like reinforced concrete structure which acts as a reservoir for the circulating water pumps and also houses the circulating water pumps, the service water pumps, and the fire pumps. The structure is founded on a rectangular reinforced concrete slab 6 ft thick, 170 ft long in the E-W direction, and 179 ft wide in the N-S direction. The foundation slab is horizontal at elevation 545 ft on the intake end of the structure and slopes gently downward to another horizontal slab at elevation 537 ft under the pump section area. Grade elevation is 591 ft. Figure 4-1 shows a vertical section through the structure.

The circulating water supply flows into the crib house through three 16 ft diameter circular intake pipes which extend approximately 2600 ft out into Lake Michigan. At the intake end of the crib house, reinforced concrete box structures anchor the intake pipe and channel the water flow into the structure (Figure 4-2). Above these box structures, two vertical walls (3 ft and 4 ft thick), spaced 14 ft apart, extend across the crib house. Between these two walls, a warming pipe runs from the side of the crib house to the center intake pipe.

At the back or west end of the crib house, longitudinal walls (Figure 4-3) form six cells which channel the flow of water into the pump section areas. The longitudinal walls span from the foundation slab to the operating floor at elevation 594 ft. Except for one 7 ft. thick wall at the center of the crib house, all the longitudinal walls are 3 ft. thick. The lateral support of these walls is provided mainly by the operating floor and two deep beams (Figure 4-1). At the intake end of

the six cells stop log guides are provided. Water flowing into the cells passes through moving screens whose driving mechanisms are supported on the operating floor.

The operating floor is a 2 ft. thick reinforced concrete slab which covers the total width and approximately one-half the length of the crib house. The foundation walls at the open area are supported by two vertical walls and concrete beams which run approximately parallel to the foundation slab at about one-third the way down from the top of the foundation wall (Figure 4-5). The operating floor supports six vertical service water pumps spaced equally across its width, the two fire pumps, and the reinforced concrete pump enclosure. Figure 4-7 indicates the location of the service pumps and fire pumps inside the enclosure. The enclosure was constructed of 18-inch thick reinforced concrete roof slab and walls. The roof plan of the pump enclosure at elevation 616'-6" is shown in Figure 4-8. Several large openings in the roof slab are shown in the figure.

The circulating water pump room, located under the operating floor and behind the service water pumps, houses six vertical circulating water pumps. The room is enclosed by three foundation walls (4 ft. thick), one 4-ft. thick vertical wall, the operating floor, and the floor slab at elevation 552 ft. 3 in. The pump floor slab (2 ft. 9 in. thick) is supported by short vertical walls below which is located the pump suction area (Figure 4-9). The circulating water pump drives are located on the operating floor directly over the circulating water pumps.

The elevations of the crib house aboveground structure are shown in Figures 4-10 and 4-11. It encloses the entire area from the center line of the stop log guides (see Figure 4-7) to the west end of the crib house. The exterior walls are 12-inch thick horizontally reinforced concrete block walls. Figure 4-7 indicates the location of the missile

walls which also have vertical reinforcing. The roof plan of the above ground structure is shown in Figure 4-12. The roof slabs at elevation 613'-0" and 624'-0" are 3-1/2-inch thick precast concrete channel slabs supported by steel braced roof framing. There is no indication from available drawings whether the roof channel slabs are connected to each other or tied down to the steel framing. The 12-inch concrete block walls are attached to the steel columns and roof beams. Details of the block wall construction are shown in Figure 4-13.

The structural steel framing at elevation 649'-0" supports the 16 ton trolley which runs in the north-south direction along the entire length of the service pump enclosure. The frame has bracing along the N-S direction only (Figure 4-10). In the E-W direction, the steel columns adjacent to the high roof (elevation 624 ft.) are connected to the high roof braced steel framing (Figure 4-11).

4.2 CRIB HOUSE FAILURE MODES

As described earlier, the function of the crib house is to act as a reservoir and to house the circulating water pumps, service water pumps and fire pumps. The identification of the failure modes of the crib house is thus concentrated primarily on those structural failures which will interrupt intake and flow of the water. Blockage of the flow channel by the collapsed longitudinal guide walls or foundation walls, loss of stop log and moving screens and possible subsequent damage to the pump by debris, and damage of the pumps by collapsed roof or walls could all cause loss of function of the crib house.

The most likely soil foundation mode of failure for the crib house is expected to involve sliding in the east direction. The crib house is embedded 60 ft. on the west side but essentially none on the east (inlet) side. There is some change in base slab elevation from west to east and some shear capacity provided by the local thickening of the slab under the stop log guides and at the intake. However, the dynamic

soil pressures developed on the west wall during a severe earthquake could conceivably result in some sliding of the structure with resulting pipe fracture.

A summary of the crib house structural failure modes is shown below. These potential failure modes are described in the following subsection, and Table 4-1 includes a somewhat more detailed tabulation.

1. Longitudinal guide wall failure from shear failure, flexural failure, or concrete crushing from out-of-plane response.
2. Operating floor diaphragm failure from shear failure of slab initiating from cut-outs or failure of shear transfer to walls.
3. Service water pump enclosure failure from loss of roof diaphragm due to shear failure at roof-shear wall junction or initiating from cut-outs - flexural failure and collapse of out-of-plane walls.
4. Failure of north and south shear walls - loss of lateral support - flexural failure and collapse of out-of-plane walls.
5. Failure of concrete walls at the intake end of the structure due to out-of-plane bending - partial flow blockage.
6. Failure of concrete strut in open area from combined axial compression and biaxial bending - loss of north and south foundation walls from excess lateral soil pressure:

7. Failure of masonry block walls due to rigid body rocking and collapse.
8. Collapse of roof top trolley frame due to lack of E-W lateral bracing.

MAJOR LATERAL FORCE RESISTING SYSTEM

The box-like crib house is symmetric about its E-W direction axis but is not symmetric in the N-S direction due to lack of north-south direction shear walls near the center of the crib house. The major vertical shear resisting structural elements for N-S ground motions are the west side foundation wall, circulating water pump room east wall, and the short walls below the pump room. For the E-W ground motions, the vertical shear resisting elements are the north and south foundation walls and the E-W direction longitudinal guide walls (Figure 4-3).

For the N-S direction ground motion, torsional response is expected due to nonsymmetry of the crib house structure. The longitudinal guide walls may be subject to tension yielding of reinforcing steel or crushing of concrete near the base due to excessive torsion-induced out-of-plane displacement at the operating floor level. Similarly, the short reinforced concrete walls supporting the pump room floor slab (Figure 4-9) will have the same mode of failure. The out-of-plane failure mode of these walls results in the reduction of in-plane shear resisting capacity and lower shear stiffness. The consequence is to lower the response frequencies of the structure in both principal directions and to force the foundation walls to resist most of the inertia force due to the E-W ground motions.

The possible failure mode of the concrete floor slabs which act as rigid diaphragms in distributing inertia forces to vertical resisting elements (i.e., walls) will probably be shear failure at the junctions of floor slab and walls.

PUMP ENCLOSURE

The pump enclosure at the operating floor is a 165 ft long by 28 ft wide reinforced concrete box-type structure (Figure 4-7). The enclosure structure is essentially symmetric about the two orthogonal directions. Thus, no torsion is expected except that resulting from the response of the remaining part of the structure. Because of the unusually high aspect ratio of the roof slab, some horizontal response amplification of the roof slab is expected. The failure modes in the lateral inertia force resisting system of the pump enclosure are: the flanges of the roof diaphragm, the shear capacity of the diaphragm, and the shear capacity of the north and south end shear walls. Failure of these critical structural elements would result in the loss of the lateral support of the east and west side walls. Failure of the service water pumps is probable if the east and west walls collapse.

VERTICAL WALLS AT INTAKE END

At the intake end of the crib house, two vertical walls (3 ft and 4 ft thick) extend the length of the crib house (Figure 4-1). No effective structural element is available above elevation 565'-0" to provide out-of-plane (west direction) lateral support for these two walls. The possible consequence of failure of these walls is the collapse of the north and south foundation walls and the consequent blockage of the flow channel.

FOUNDATION WALLS AT OPEN AREA

The sloped beams which support the north and south foundation walls at the open area of the crib house are supported in turn by the 4 ft thick wall and a reinforced concrete strut at each end of the structure (Figure 4-5). The possible failure mode of the 4 ft wall was discussed above. The reinforced concrete strut has dimensions of 3 ft wide by 5 ft deep and continuously spans between the longitudinal guide walls. The span between two adjacent longitudinal guide walls is 25'-4". The horizontal or vertical response amplification of this strut under ground motion needs to be considered. The strut can fail in buckling due to

interaction of axial compression and biaxial bending moments. The consequence of the strut failure is the possible collapse of foundation wall under lateral soil pressure (static plus dynamic effects).

SERVICE WATER PIPES

The 48-inch diameter service water pipes have their centerlines at elevation at 579'-0" (Figure 4-1) and are buried underground between the crib house and auxiliary building. The detail layout of the pipes is not available at this time. The possible failure modes are shear failure, tension failure, or buckling of the pipes due to seismic waves and/or relative displacements at the anchor points.

ABOVEGROUND STRUCTURE

The aboveground structure of the crib house includes the pump enclosure which is in turn enclosed by the concrete block walls and precast concrete channel roof slabs. The roof slabs are supported by a braced roof frame which encloses the entire operating floor. On top of the service water pump enclosure a steel braced frame supports the trolley. The steel braced frame is laterally supported by the high roof as discussed previously. The masonry block walls are expected to crack at the base from out-of-plane response at very low ground acceleration due to lack of vertical wall reinforcing. This will be locally modified in the areas designated as missile walls by the presence of vertical reinforcing steel. For the unreinforced masonry walls, the failure will be essentially vertical, in-place collapse once the $p-\Delta$ effects become critical. However, failure of either rooftop steel frame or enclosing masonry block walls will not necessarily damage any of the safety related equipment.

Structural Element Description	Structural Response Description	Structural Damage
Turbine building roof truss connection	Roof truss response to the E-W ground motion	Shear failure of the bolts; stress relief; lateral failure of the gusset plate or the member; loss of roof diaphragm
Turbine building roof truss remote angle diagonal braces	Roof truss response to the E-W ground motion	Buckling of the diagonal bracing; tension failure of braces; loss of roof diaphragm
AS3072 building columns between the turbine and auxiliary building	Orthogonal roof inertia force due to the E-W ground motion after the roof truss fails	Yielding or buckling of the column under the P- Δ ; effects collapse of the roof
Vertical braced frames at the north and south ends of the turbine building	Building response to concurrent E-W and N-S direction ground motions	Loss of lateral force load path; vertical collapse of roof system
a. Pipe diagonal braces		a. Buckling failure of the braces; tension failure
b. Connections		b. Welding failure between the pipe brace and gusset plate; shear failure of the bolts
c. Corner columns		c. Column buckling under axial compression and P- Δ effects due to combined lateral and torsional response
d. Column anchor bolts		d. Anchor bolts failure under combined tension and shear
The auxiliary building roof concrete slab	Roof response to concurrent E-W and N-S direction ground motions	Loss of concrete slab roof diaphragm
a. Junction of the concrete roof slab and end shear walls		a. Shear failure of the concrete slab
b. Roof girder collector beams		b. Failure of steel joist/girder between the beam and concrete slab; beam and connection failure in tension or shear
The auxiliary building roof truss	Roof response to concurrent E-W and N-S direction ground motions	Loss of the roof diaphragm
a. Connection		a. Shear failure of the bolts; stress relief; tension failure of the gusset plate or the member
b. Remote angle diagonal braces		b. Buckling and tension failure of the diagonal braces
Steel stud welded to the web of the AS3072 building columns between the turbine and auxiliary building	Roof and floor slabs response to the concurrent N-S and E-W ground motion	Shear failure of the studs; failure of the stud walls; crush of the concrete surrounding the studs; change of system response due to reduction of shear wall stiffness
Auxiliary building shear walls	Lateral inertia force of the roof and floor slabs parallel to the plane of the wall	Diagonal failure; shear failure across a corner; P- Δ joint; shear failure across a plastic hinge zone; loss of shear wall retaining capacity
Auxiliary building structural steel braced frames	Lateral inertia force of the roof and floor slabs parallel to the frame	Loss of lateral force load path; vertical collapse of roof and floor slabs
a. Connections		a. Welding between the structural steel brace and gusset plate; shear failure of the bolts
b. Pipe brace		b. Tension failure of braces
c. Corner columns		c. Column buckling under axial compression and P- Δ effects due to E-W roof lateral and torsional response
d. Column anchor bolts		d. Anchor bolts failure under combined tension and shear
Roof girder	Response of the girder to concurrent seismic response and dead weight	Plastic hinge at the mid-span; partial collapse of the roof system
The first three reinforced concrete walls supporting the control room	Response resulting due to horizontal ground motion	Yielding of the vertical wall; compression; out-of-plane failure; vertical collapse of the roof system

TABLE 4-1. POTENTIAL CRIB HOUSE STRUCTURAL FAILURE MODES

FIGURE 4-1. EAST-WEST SECTION OF THE CRIB HOUSE

FIGURE 4-2. SECTION AT THE INTAKE END OF THE CRIB HOUSE

FIGURE 4-3. SECTION AT THE PUMP SUCTION AREA OF THE CRIB HOUSE

FIGURE 4-4. SHORT CONCRETE WALLS BELOW THE PUMP ROOM

FIGURE 4-6. FOUNDATION WALL AT THE OPEN AREA

- — INDICATES LOCATION OF SERVICE WATER PUMPS
— INDICATES LOCATION OF FIRE PUMPS
— INDICATES LOCATION OF CIRCULATING WATER PUMPS

NOTE: OPERATING FLOOR SLAB STOPS AT LINE A-A

FIGURE 4-7 OPERATING FLOOR PLAN

FIGURE 4-8. PUMP ENCLOSURE ROOF PLAN (EL. 616'-6")

FIGURE 4-9. PUMP ROOM FLOOR SLAB

EAST ELEVATION

WEST ELEVATION

FIGURE 4-10. ELEVATION OF CRIB HOUSE

FIGURE 4-11. ELEVATION OF CRIB HOUSE

FIGURE 4-12. CRIB HOUSE ROOF PLAN

TIE BETWEEN BLOCK WALL AND STEEL COLUMN

TIE BETWEEN BLOCK WALL AND STEEL BEAM

FIGURE 4-13. CONSTRUCTION DETAIL OF MASONRY BLOCK WALL

REFERENCES

1. Oesterle, R. G., A. E. Fiorato, L. S. Johal, J. E. Carpenter, H. G. Russel and W. G. Corley, "Earthquake Resistant Structural Walls - Tests of Isolated Walls", prepared for National Science Foundation, Portland Cement Association, Skokie, Illinois, November, 1976.
2. Jimenez, R., P. Perdikaris, P. Gergely, and R. White, "Interface Shear Transfer and Dowel Action in Cracked Reinforced Concrete Subject to Cyclic Shear", Proceedings of the National Structural Engineering Conference, Madison, Michigan, August, 1976.
3. Paulay, T., "Some Design Considerations of Earthquake Resistant Reinforced Concrete Shear Walls", International Symposium on Earthquake Structural Engineering, St. Louis, Missouri, August, 1976.
4. Bresler, B., "Behavior of Structural Elements - A Review", Building Practices for Disaster Mitigation, National Bureau of Standards (Washington, D.C.) Building Science Series 46, February, 1973.
5. Banerjee, A. K., K. A. Condon, R. A. Rettig, and C. F. Reeves, "Design of Reinforced Concrete Containment", Transactions Fourth International Conference on Structural Mechanics in Reactor Technology, San Francisco, California, paper J3/1, 1977.
6. Chen Pang Tan, "A Study of the Design and Construction Practices of Prestressed Concrete and Reinforced Concrete Containment Vessels", prepared for Oak Ridge National Laboratory, The Frankling Institute Research Laboratories, Philadelphia, Pennsylvania, August, 1969.
7. White, R. N., and P. Gergely, "Design Considerations for Seismic Tangential Shear in Reinforced Concrete Containment Structures", Transactions Fourth International Conference on Structural Mechanics in Reactor Technology, San Francisco, California, paper J3/5, 1977.
8. Green, D. F., and T. E. Johnson, "Design of Concrete Containments for Tangential Shear Loads", Transactions Fourth International Conference on Structural Mechanics in Reactor Technology, San Francisco, paper J3/4, 1977.

9. Sargent and Lundy Engineers, "Seismic Analysis of the Reactor Building - Zion Station", Chicago, Illinois, April, 1971.
10. Sargent and Lundy Engineers, "Seismic Analysis of Zion Crib House", Chicago, Illinois, April, 1968.
11. Sargent and Lundy Engineers, "Seismic Analysis of the Auxiliary-Turbine Building Complex - Zion Station", Chicago, Illinois, February, 1969.
12. "An Assessment of Accident Risk in U.S. Commercial Nuclear Power Plants", WASH-1400 (NUREG-75/014), USNRC, Washington, October, 1975.
13. Kennedy, R. P., Short, S. A., Wesley, D. A., and Lee, T. H., "Effect of Nonlinear Soil-Structure Interaction Due to Base Slab Uplift on the Seismic Response of a High Temperature Gas Cooled Reactor Plant", Transactions of the 3rd International Conference on Structural Mechanics in Reactor Technology, London, U.K., paper K3/5, 1975.
14. Final Safety Analysis Report (FSAR), Zion Station, Commonwealth Edison Company.
15. Cardenas, A. E., J. M. Hanson, W. G. Corley, and E. Hognested, "Design Provisions for Shear Walls", ACI Journal Proceedings, Vol. 70, March, 1973, pp. 221-230.
16. Benjamin, J. R., and H. A. William, "Investigation of Shear Walls", Parts 3, 5, 8, 9, and 12, Department of Civil Engineering, Stanford University, 1953-1956.
17. Barda, F., J. M. Hanson, and W. G. Corley, "Shear Strength of Low-Rise Walls and Boundary Elements", Reinforced Concrete Structures in Seismic Forces, SP-53, American Concrete Institute, 1977.
18. Benjamin, J. R., "Variability Analysis of Shear Wall Structures", Proceedings, Fourth World Conference on Earthquake Engineering, Santiago, Chile, 1969.

19. ATC-3-66, "Tentative Provisions for the Development of Seismic Regulation for Buildings", Applied Technology Council, Palo Alto, California. Also referenced as National Bureau of Standard Special Publication 510 and National Science Foundation Publication 78-8, June, 1978.
20. "Structural Analysis and Design of Nuclear Plant Facilities", Draft for Trial Use and Comment, ASCE, 1976.

NOTICE

This work was supported by the U.S. Nuclear Regulatory Commission under Interagency Agreement DOE 40-450-75 with the U.S. Department of Energy.

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