February 11, 1998

APPLICANT: Westinghouse Electric Corporation

PROJECT: AP600

SUBJECT: SUMMARY OF AP600 MEETING TO DISCUSS AP600 STRUCTURAL MODULES

The subject meeting was held on January 20 and 21, 1998, at Brookhaven National Laboratory in Upton New York. Attachment 1 is a list of the participants. Attachment 2 contains the handouts provided by Westinghouse during the meeting. This attachment includes miscellaneous changes to the standard safety analysis report (SSAR) that were agreed to during the meeting to resolve issues. Westinghouse took an action to incorporate these changes in a future SSAR revision.

The purpose of the meeting was to review structural calculations in order to resolve final safety evaluation report (FSER) open items that were sent to Westinghouse in a December 9, 1997, letter. Attachment 3 provides the details of the results of the review for the selected FSER open items.

A draft of this meeting summary was provided to Westinghouse to allow them the opportunity to comment on the summary prior to issuance.

original signed by:

Joseph M. Sebrosky, Project Manager
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Office of Nuclear Reactor Regulation

Docket No. 52-003

Attachments: As stated

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AP600 MEETING TO DISCUSS AP600 STRUCTURAL MODULES
MEETING ATTENDEES
JANUARY 20 AND 21, 1998

NAME	ORGANIZATION
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RICHARD ORR	WESTINGHOUSE
TOM CHENG	NRR/DE/ECGB
JOSEPH BRAVERMAN	BNL(NRC CONSULTANT)
JOE SEBROSKY	NRR/DRPM/PDST

Attachment 1

NRC Structural Meeting - 1/20 - 21/98
FSER Open Items related to Structural Modules

FSER Open Items

220.121 Design of shear studs

220.122 Critical sections for CIS modules added to SSAR 3.8.3
Critical sections for fuel pool modules added to SSAR Appendix 3H in response to FSER Open Item 220.128
Summary reports are internal AP600 documents made available to NRC to assist during structural reviews

220.123 Implementation of design procedures in design calculations

220.126 Air baffle evaluation for air flow fluctuations

220.128 Auxiliary building roof slab
Shield building roof covered under 220.124
Critical sections for auxiliary and shield building provided in SSAR Appendix 3H

Documents

1100-SUC-101, Rev.6	Structural wall modules - Containment Internal structures
GW-SUP-003, Rev 2	Structural Analysis Methodology for Steel-Concrete Composite Panels with Welded Shear Studs.
1100-SUC-003, Rev. 1	Structural modules - Shear studs, General design
1200-SUC-101, Rev. ~~2~~ 4	Structural modules - areas 5 and 6 - auxiliary building
MT03-S3C-022, Rev. 1	IRWST Steel Wall Structural Design

Summary reports to assist review

1100-S3R-001	Design Summary Report Containment Internal Structures
1200-S3R-001	Design Summary Report Auxiliary and Shield Buildings

Status of FSER Open Items

Attachment 2

CHANGES TO SSAR SECTIONS 3.7 AND 3.8

Incorporate revision identified in issued responses to FSER Open Items

PLUS changes shown on the attached pages

3.7.2.14 Determination of Seismic Category I Structure Overturning Moments

Subsection 3.8.5.5.4 describes the effects of seismic overturning moments.

3.7.2.15 Analysis Procedure for Damping

Subsection 3.7.1.3 presents the damping values used in the seismic analyses. For structures comprised of different material types, the composite modal damping approach utilizing the strain energy method is used to determine the composite modal damping values. Subsection 3.7.2.4 presents the damping values used in the soil-structure interaction analysis.

3.7.3 Seismic Subsystem Analysis

This subsection describes the seismic analysis methodology for subsystems, which are those structures and components that do not have an interface with the soil-structure interaction analyses. Structures and components considered as subsystems include the following:

- Structures, such as floor slabs, and walls miscellaneous steel platforms and framing
- Equipment modules consisting of components, piping, supports, and structural frames
- Equipment including vessels, tanks, heat exchangers, valves, and instrumentation
- Distributive systems including piping and supports, electrical cable trays and supports, HVAC ductwork and supports, instrumentation tubing and supports, and conduits and supports

Subsection 3.9.2 describes dynamic analysis methods for the reactor internals. Subsection 3.9.3 describes dynamic analysis methods for the primary coolant loop support system. Subsection 3.7.2 describes the analysis methods for seismic systems, which are those structures and components that are considered with the foundation and supporting media. Section 3.2 includes the seismic classification of building structures, systems, and components.

3.7.3.1 Seismic Analysis Methods

The methods used for seismic analysis of subsystems include, modal response spectrum analysis, time-history analysis, and equivalent static analysis. The methods described in this subsection are acceptable for any subsystem. The particular method used is selected by the designer based on its appropriateness for the specific item. Items analyzed by each method are identified in the descriptions of each method in the following paragraphs.

3.7.3.2 Determination of Number of Earthquake Cycles

Seismic Category I structures, systems, and components are evaluated for one occurrence of the safe shutdown earthquake (SSE). In addition, subsystems sensitive to fatigue are evaluated for cyclic motion due to earthquakes smaller than the safe shutdown earthquake. Using analysis methods, these effects are considered by inclusion of seismic events with an amplitude not less than one-third of the safe shutdown earthquake amplitude. The number of cycles is calculated based on IEEE-344-1987 (Reference 21) to provide the equivalent fatigue damage of two safe shutdown earthquake events with 10 high-stress cycles per event. Typically, there are five seismic events with an amplitude equal to one-third of the safe shutdown earthquake response. Each event has 63 high-stress cycles. For ASME Class 1 piping, the fatigue evaluation is performed based on five seismic events with an amplitude equal to one-third of the safe shutdown earthquake response. Each event has 63 high-stress cycles.

When seismic qualification is based on dynamic testing for structures, systems, or components containing mechanisms that must change position in order to function, operability testing is performed for the safe shutdown earthquake preceded by one or more earthquakes. The number of preceding earthquakes is calculated based on IEEE-344-1987 (Reference 21) to provide the equivalent fatigue damage of one safe shutdown earthquake event. Typically, the preceding earthquake is one safe shutdown earthquake event or five one-half safe shutdown earthquake events.

3.7.3.3 Procedure Used for Modeling

The dynamic analysis of any complex system requires the discretization of its mass and elastic properties. This is accomplished by concentrating the mass of the system at distinct characteristic points or nodes, and interconnecting them by a network of elastic springs representing the stiffness properties of the systems. The stiffness properties are computed either by hand calculations for simple systems or by finite element methods for more complex systems.

Nodes are located at mass concentrations and at additional points within the system. They are selected in such a way as to provide an adequate representation of the mass distribution and high-stress concentration points of the system.

At each node, degrees of freedom corresponding to translations along three orthogonal axes, and rotations about these axes are assigned. The number of degrees of freedom is reduced by the number of constraints, where applicable. For equipment qualification, reduced degrees of freedom are acceptable provided that the analysis adequately and conservatively predicts the response of the equipment.

The size of the model is reviewed so that a sufficient number of masses or degrees of freedom are used to compute the response of the system. A model is considered adequate provided that additional degrees of freedom do not result in more than a 10 percent increase in response, or the number of degrees of freedom equals or exceeds twice the number of modes with frequencies less than 33 hertz.

Dynamic models of floor and roof slabs and miscellaneous steel platforms and framing include masses equal to 25 percent of the floor live load or 75 percent of the roof snow load, whichever is applicable.

Dynamic models are prepared for the following seismic Category I steel structures. Response spectrum or time history analyses are performed for structural design.

guide the operator on a timely basis to determine if the level of earthquake ground motion requiring shutdown has been exceeded. The procedures will follow the guidance of EPRI Reports NP-5930 (Reference 1), TR-100082 (Reference 17), and NP-6[illegible] (Reference 18), as modified by the NRC staff (Reference 32).

3.7.5.3 Seismic Interaction Review

The seismic interaction review will be updated by the Combined License applicant. This review is performed in parallel with the seismic margin evaluation. The review is based on as-procured data, as well as the as-constructed condition.

3.7.5.4 Reconciliation of Seismic Analyses of Nuclear Island Structures

The Combined License applicant will reconcile the seismic analyses described in subsection 3.7.2 for detail design changes such as those due to as-procured equipment information, including the effect due to these deviations. If it is necessary to update the soil structure interaction analyses, these analyses should be performed with site specific soil properties using seismic input defined by the response spectra given in Figures 3.7.1-1 and 3.7.1-2.

Deviations are acceptable based on an evaluation consistent with the methods and procedure of Section 3.7 provided the amplitude of the seismic floor response spectra do not exceed the design basis floor response spectra by more than ~~to~~ 10 percent.

3.7.6 References

1. EPRI Report NP-5930, "A Criterion for Determining Exceedance of the Operating Basis Earthquake," July 1988.

2. Uniform Building Code, 1991.

3. ASCE Standard 4-86, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," American Society of Civil Engineers, September 1986.

4. ASME B&PV Code, Code Case N-411.

5. H. B. Seed, and I. M. Idriss, "Soil Moduli and Damping Factors for D[illegible]ic Response Analysis," Report No. EERC-70-14, Earthquake Engineering Research Center, University of California, Berkeley, 1970.

6. H. B. Seed, R. T. Wong, I. M. Idriss, and K. Tokimatsu, "Moduli and Damping Factors for Dynamic Analysis of Cohesionless Soils," Report No. UCB/EERC-8914, Earthquake Engineering Research Center, University of California, Berkeley, 1984.

7. Bechtel Corporation, "User's and Theoretical Manual for Computer Program BSAP (CE800)," Revision 12, 1991.

8. Bechtel Corporation, "Theoretical, Validation and User's Manuals for Computer Program SASSI (CE994)," 198[illegible]

refueling cavity are also designed for the hydrostatic head due to the water in the refueling cavity and the hydrodynamic pressure effects of the water due to the safe shutdown earthquake.

Figure 3.8.3-8 shows the typical design details of the structural modules, typical configuration of the wall modules, typical anchorages of the wall modules to the reinforced base concrete, and connections between adjacent modules. Concrete-filled structural wall modules are designed as reinforced concrete structures in accordance with the requirements of ACI-349, as supplemented in the following paragraphs. The faceplates are considered as the reinforcing steel, bonded to the concrete by headed studs. The application of ACI-349 and the supplemental requirements are supported by the behavior studies described in subsection 3.8.3.4.1. The design of critical sections is described in the design summary report (see subsection 3.8.3.5.7).

3.8.3.5.3.1 Design for Axial Loads and Bending

Design for axial load (tension and compression), in-plane bending, and out-of-plane bending is in accordance with the requirements of ACI-349, Chapters 10 and 14.

3.8.3.5.3.2 Design for In-Plane Shear

Design for in-plane shear is in accordance with the requirements of ACI-349, Chapters 11 and 14. The steel faceplates are treated as reinforcing steel, contributing as provided in Section 11.5 of ACI-349.

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3.8.3.5.3.3 Design for Out-of-Plane Shear

Design for out-of-plane shear is in accordance with the requirements of ACI-349, Chapter 11.

3.8.3.5.3.4 Evaluation for Thermal Loads

The effect of thermal loads on the concrete-filled structural wall modules is evaluated by using the working stress design method for load combination 3 of Table 3.8.4-2. This evaluation is in addition to the evaluation using the strength design method of ACI-349 for the load combination without the thermal load. Acceptance for the load combination with normal thermal loads, which includes the thermal transients described in subsection 3.8.3.3.1, is that the stress in general areas of the steel plate be less than yield. In local areas where the stress may exceed yield the total stress intensity range is less than twice yield. This evaluation of thermal loads is based on the ASME Code philosophy for Service Level A loads given in ASME Code, Section III, Subsection NE, Paragraphs NE-3213.13 and 3221.4.

3.8.3.5.3.5 Design of Trusses

The trusses provide a structural framework for the modules, maintain the separation between the faceplates, support the modules during transportation and erection, and act as "form ties" between the faceplates when concrete is being placed. After the concrete has cured, the trusses

3.8.3.5.6 Steel Form Modules

The steel form modules consist of plate reinforced with angle stiffeners and tee sections as shown in Figure 3.8.3-16. The steel form modules are designed for concrete placement loads defined in subsection 3.8.3.3 2.

The steel form modules are designed as steel structures according to the requirements of AISC-N690. This code is applicable since the form modules are constructed entirely out of structural steel plates and shapes and the applied loads are resisted by the steel elements.

3.8.3.5.7 Design Summary Report

FSER OI 230.122

A design summary report is prepared for containment internal structures documenting that the structures meet the acceptance criteria specified in subsection 3.8.4.5. ~~Reference 49 provides the design summary report. Critical sections included in the report are:~~

- ~~South west wall of the refueling cavity~~
- ~~South wall of west steam generator cavity~~
- ~~North east wall of in-containment refueling water storage tank~~
- ~~In-containment refueling water storage tank steel wall~~
- ~~Column supporting operating floor~~

Deviations from the design due to as-procured or as-built conditions are acceptable based on an evaluation consistent with the methods and procedures of Section 3.7 and 3.8 provided the following acceptance criteria are met.

- the structural design meets the acceptance criteria specified in Section 3.8
- the ~~amplitude of the~~ seismic floor response spectra ~~do not exceed the design basis floor response spectra by more than 10 percent~~ meet the acceptance criteria specified in subsection 3.7.5.4

Depending on the extent of the deviations, the evaluation may range from documentation of an engineering judgement to performance of a revised analysis and design. The results of the evaluation will be documented in an as-built summary report by the Combined License applicant

3.8.3.5.8 Design Summary of Critical Sections

3.8.3.5.8.1 Structural Wall Modules

This subsection summarizes the design of the following critical sections:

- South west wall of the refueling cavity (4′ 0″ thick)
- South wall of west steam generator cavity (2′ 6″ thick)
- North east wall of in-containment refueling water storage tank (2′ 6″ thick)

The thicknesses and locations of these walls which are part of the boundary of the in-containment refueling water storage tank are shown in Table 3.8.3-3 and Figure 3.8.3-18.

elevation 66'-6" to elevation 135'-3". The minimum thickness of the faceplates is 0.5 inch.

MOVE FIGURE INTO APPENDIX 3H.

The ceiling of the main control room (floor at elevation 135'-3"), and the instrumentation and control rooms (floor at elevation 117'-6") are designed as finned floor modules (Figure 3.8.4-?). A finned floor consists of a 24-inch-thick concrete slab poured over a stiffened steel plate ceiling. The fins are rectangular plates welded perpendicular to the plate. Shear studs are welded on the other side of the steel plate, and the steel and concrete act as a composite section. The fins are exposed to the environment of the room, and enhance the heat-absorbing capacity of the ceiling (see Standard Safety Analysis Report (SSAR) subsection 6.4.2.2). Several shop-fabricated steel panels, placed side by side, are used to construct the stiffened plate ceiling in a modularized fashion. The stiffened plate is designed to withstand construction loads prior to concrete hardening.

The new fuel storage area is a separate reinforced concrete pit providing temporary dry storage for the new fuel assemblies.

A cask handling crane travels in the east-west direction. The location and travel of this crane prevents the crane from carrying loads over the spent fuel pool, thus precluding them from falling into the spent fuel pool.

3.8.4.1.3 Containment Air Baffle

The containment air baffle is located within the upper annulus of the shield building, providing an air flow path for the passive containment cooling system. The air baffle separates the downward air flow entering at the air inlets from the upward air flow that cools the containment vessel and flows out of the discharge stack. The upper portion is supported from the shield building roof and the remainder is supported from the containment vessel. The air baffle is a seismic Category I structure designed to withstand the wind and tornado loads defined in Section 3.3. The air baffle structural configuration is depicted in Figures 1.2-14 and 3.8.4-1. The baffle includes the following sections:

- A wall supported off the shield building roof (see Figure 1.2-14)
- A series of panels attached to the containment vessel cylindrical wall and the knuckle region of the dome
- A sliding plate closing the gap between the wall and the panels fixed to the containment vessel, designed to accommodate the differential movements between the containment vessel and shield building
- Flow guides attached at the bottom of the air baffle to minimize pressure drop.

The air baffle is designed to meet the following functional requirements:

- The baffle and its supports are configured to minimize pressure losses as air flows through the system

3.8.4.3.1.5 Dynamic Effects of Abnormal Loads

The dynamic effects from the impulsive and impactive loads caused by P_a, R_a, Y_r, Y_j, Y_m, and tornado missiles are considered by one of the following methods:

- Applying an appropriate dynamic load factor to the peak value of the transient load

- Using impulse, momentum, and energy balance techniques

- Performing a time-history dynamic analysis

Elastoplastic behavior may be assumed with appropriate ductility ratios, provided excessive deflections will not result in loss of function of any safety-related system.

Dynamic increase factors appropriate for the strain rates involved may be applied to static material strengths of steel and concrete for purposes of determining section strength.

3.8.4.3.2 Load Combinations

3.8.4.3.2.1 Steel Structures

The steel structures and components are designed according to the elastic working stress design methods of the AISC-N690 specification using the load combinations specified in Table 3.8.4-1.

3.8.4.3.2.2 Concrete Structures

The concrete structures and components are designed according to the strength design methods of ACI-349 Code, using the load combinations specified in Table 3.8.4-2.

3.8.4.3.2.3 Live Load for Seismic Design

the response due to   whichever is applicable.

Include this paragraph as 3H.4.1 in Appendix 3H

Floor live loads, based on requirements during plant construction and maintenance activities, are specified varying from 50 to 250 pounds per square foot (with the exception of the containment operating deck which is designed for 800 pounds per square foot specified for plant maintenance condition).

seismic loads include masses equal to 25 percent of the floor live loads or 75 percent of the roof snow load. These seismic loads are combined with

For the local design of members, such as the floors and beams, ~~live loads in combination with the safe shutdown earthquake are taken as~~ 100 percent of the~~se~~ specified live loads, or 75 percent of the roof snow load, whichever is applicable, except in the case of the containment operating deck. For the seismic load combination, the containment operating deck is designed for a live load of 200 pounds per square foot which is appropriate for plant operating condition. These live and snow loads are included as mass in calculating the vertical seismic forces on the floors and roof. The mass of equipment and distributed systems is included in both the dead and seismic loads.

Steel Construction, Load and Resistance Factor Design, First Edition. See subsection 6.1.2.1 for additional description of the protective coatings.

3.8.4.5.3 Design Summary Report

A design summary report is prepared for seismic Category I structures documenting that the structures meet the acceptance criteria specified in subsection 3.8.4.5. ~~Reference 50 provides the design summary report.~~ Critical sections included in the report are:

Move this section as modified by response to OI 220.128 into new subsection 3.8.4.5.4 Design summary of Critical Sections

- Passive containment cooling system water storage tank
- Shield building roof to cylinder connection
- Shield building to auxiliary building connection at elevation 180'
- South wall of auxiliary building (column line 1)
- Interior wall of auxiliary building (column line 7.3)
- West wall of main control room in auxiliary building (column line L), elevation 117'-6" to elevation 153'-0"
- North wall of auxiliary building (column line 11 between Q and P), elevation 117'-6" to elevation 153'-0"
- Floor slab in north end of auxiliary building at elevation 135'-3" including:
  - 9 inch concrete slab on metal deck
  - 24 inch reinforced concrete slab
  - 24 inch finned floor above the main control room
- Spent fuel pool divider wall and floor

Deviations from the design due to as-procured or as-built conditions are acceptable based on an evaluation consistent with the methods and procedures of Section 3.7 and 3.8 provided the following acceptance criteria are met.

- the structural design meets the acceptance criteria specified in Section 3.8
- the ~~amplitude of the~~ seismic floor response spectra ~~do not exceed the design basis floor response spectra by more than 10 percent~~ meet the acceptance criteria specified in subsection 3.7.5.4

Depending on the extent of the deviations, the evaluation may range from documentation of an engineering judgement to performance of a revised analysis and design. The results of the evaluation will be documented in an as-built summary report by the Combined License applicant

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

This subsection contains information relating to the materials, quality control program, and special construction techniques used in the construction of the other seismic Category I structures, as well as the containment internal structures.

results in the largest demand for the top reinforcement in the basemat. The analyses of the three construction sequences demonstrate the following:

- The design of the basemat and superstructure accommodates the construction-induced stresses considering the construction sequence and the effects of the settlement time history.

- The design of the basemat can accommodate delays in the shield building so long as the auxiliary building construction is suspended at elevation 117′ 0″. Resumption in construction of the auxiliary building can proceed once the shield building is advanced to elevation 100′ 0″.

- The design of the basemat can accommodate delays in the auxiliary building so long as the shield building construction is suspended at elevation 84′ 0″ feet. Resumption in construction of the shield building can proceed once the auxiliary building is advanced to elevation 100′ 0″.

- After the structure is in place and cured to elevation 100′ 0″, the basemat and structure act as an integral 40 foot deep structure and the loading due to construction above this elevation is not expected to cause significant additional flexural demand with respect to the basemat and the shield building concrete below the containment vessel. Accordingly, there is no need for placing constraints on the construction sequence above elevation 100′ 0″.

The site conditions considered in the evaluation provide reasonable bounds on construction induced stresses in the basemat. Accordingly, the AP600 basemat design is adequate for practically all soil sites and it can tolerate major variations in the construction sequence without causing excessive deformations, moments and shears due to settlement over the plant life.
The analyses of alternate construction scenarios show that member forces in the basemat are acceptable subject to the following limits imposed for soft soil sites on the relative level of construction of the buildings prior to completion of both buildings at elevation 82′6″:

- Concrete may not be placed above elevation 82′ 6″ for the shield building or containment internal structure.

- Concrete may not be placed above elevation 117′6″ in the auxiliary building.

INSERT 3.8.5.4.4 Design Summary Report
see next page

3.8.5.4.~~4~~5 Design Summary of Critical Sections

The basemat design meets the acceptance criteria specified in subsection 3.8.4.5. Two critical portions of the basemat are identified below together with a summary of their design. The boundaries are defined by the walls and column lines which are shown in Figure 3.7.2-12 (sheet 1 of 12). Table 3.8.5-3 shows the reinforcement required and the reinforcement provided for the critical sections.

3.8.3.5.6 Steel Form Modules

The steel form modules consist of plate reinforced with angle stiffeners and tee sections as shown in Figure 3.8.3-16. The steel form modules are designed for concrete placement loads defined in subsection 3.8.3.3.2.

The steel form modules are designed as steel structures according to the requirements of AISC-N690. This code is applicable since the form modules are constructed entirely out of structural steel plates and shapes and the applied loads are resisted by the steel elements.

5.4.4

3.8.~~3.5.7~~ Design Summary Report

FSER OI 230.122

A design summary report is prepared for ~~containment internal structures~~ the basemat documenting that the structures meet the acceptance criteria specified in subsection 3.8.4.5. ~~Reference 49 provides the design summary report. Critical sections included in the report are:~~

- ~~South west wall of the refueling cavity~~
- ~~South wall of west steam generator cavity~~
- ~~North east wall of in-containment refueling water storage tank~~
- ~~In-containment refueling water storage tank steel wall~~
- ~~Column supporting operating floor~~

Deviations from the design due to as-procured or as-built conditions are acceptable based on an evaluation consistent with the methods and procedures of Section 3.7 and 3.8 provided the following acceptance criteria are met.

- the structural design meets the acceptance criteria specified in Section 3.8
- the ~~amplitude of the~~ seismic floor response spectra ~~do not exceed the design basis floor response spectra by more than 10 percent~~ meet the acceptance criteria specified in subsection 3.7.5.4.

Depending on the extent of the deviations, the evaluation may range from documentation of an engineering judgement to performance of a revised analysis and design. The results of the evaluation will be documented in an as-built summary report by the Combined License applicant

3.8.3.5.8 Design Summary of Critical Sections

3.8.3.5.8.1 Structural Wall Modules

This subsection summarizes the design of the following critical sections:

- South west wall of the refueling cavity (4' 0" thick)
- South wall of west steam generator cavity (2' 6" thick)
- North east wall of in-containment refueling water storage tank (2' 6" thick)

The thicknesses and locations of these walls which are part of the boundary of the in-containment refueling water storage tank are shown in Table 3.8.3-5 and Figure 3.8.3-18.

Basemat between the shield building and exterior wall (line 11) and column lines K and L.

This portion of the basemat is designed as a one way slab spanning a distance of 23' 6" between the walls on column lines K and L. The slab is continuous with the adjacent slabs to the east and west. The critical loading is the bearing pressure on the underside of the slab due to dead and seismic loads. This establishes the demand for the top flexural reinforcement at mid span and for the bottom flexural and shear reinforcement at the walls. The basemat is designed for the bearing pressures and membrane forces from the analyses on uniform soil springs described in subsection 3.8.5.4.1. The design moments and shears are increased by 20 percent to accommodate the nonuniform sites defined in subsection 2.5.4.5. Negative moments are redistributed as permitted by ACI 349.

The top and bottom reinforcement in the east west direction of span are equal. The reinforcement provided is shown in sheets 1, 2 and 5 of Figure 3.8.5-3. Typical reinforcement details showing use of headed reinforcement for shear reinforcement is shown in Figure 3H.5-3

Basemat between column lines 1 and 2 and column lines K-2 and N

This portion of the basemat is designed as a one way slab spanning a distance of 22' 0" between the walls on column lines 1 and 2. The slab is continuous with the adjacent slabs to the north and with the exterior wall to the south. The critical loading is the bearing pressure on the underside of the slab due to dead and seismic loads. This establishes the demand for the top flexural reinforcement at mid span and for the bottom flexural and shear reinforcement at wall 2. The basemat is designed for the bearing pressures and membrane forces from the analyses on uniform soil springs described in subsection 3.8.5.4.1. The design moments and shears are increased by 20 percent to accommodate the nonuniform sites defined in subsection 2.5.4.5. The reinforcement provided is shown in sheets 1, 2 and 5 of Figure 3.8.5-3. Typical reinforcement details showing use of headed reinforcement for shear reinforcement are shown in Figure 3H.5-3.

~~Deviations from the design due to as-procured or as-built conditions are acceptable based on an evaluation consistent with the methods and procedures of Section 3.7 and 3.8 provided the following acceptance criteria are met.~~

- ~~The structural design meets the acceptance criteria specified in Section 3.8~~
- ~~The amplitude of the seismic floor response spectra do not exceed the design basis floor response spectra by more than 10 percent~~

~~Depending on the extent of the deviations, the evaluation may range from documentation of an engineering judgement to performance of a revised analysis and design.~~

3.8.5.5 Structural Criteria

The analysis and design of the foundation for the nuclear island structures are according to ACI-349 with margins of structural safety as specified within it. The limiting conditions for the foundation medium, together with a comparison of actual capacity and estimated structure

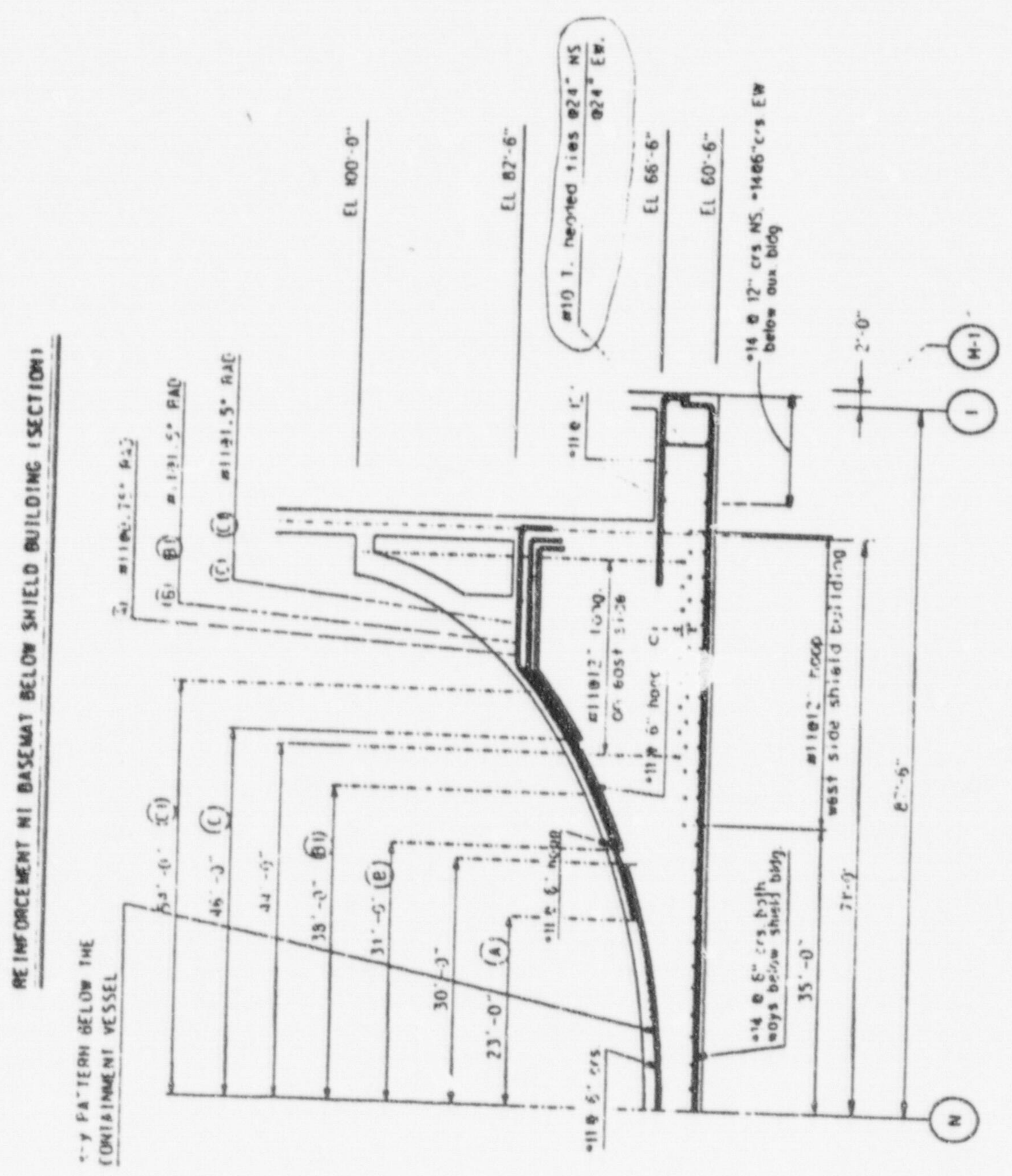


Figure 3.8.5-3 (Sheet 4 of 5)

Basemat Reinforcement - Cross Section

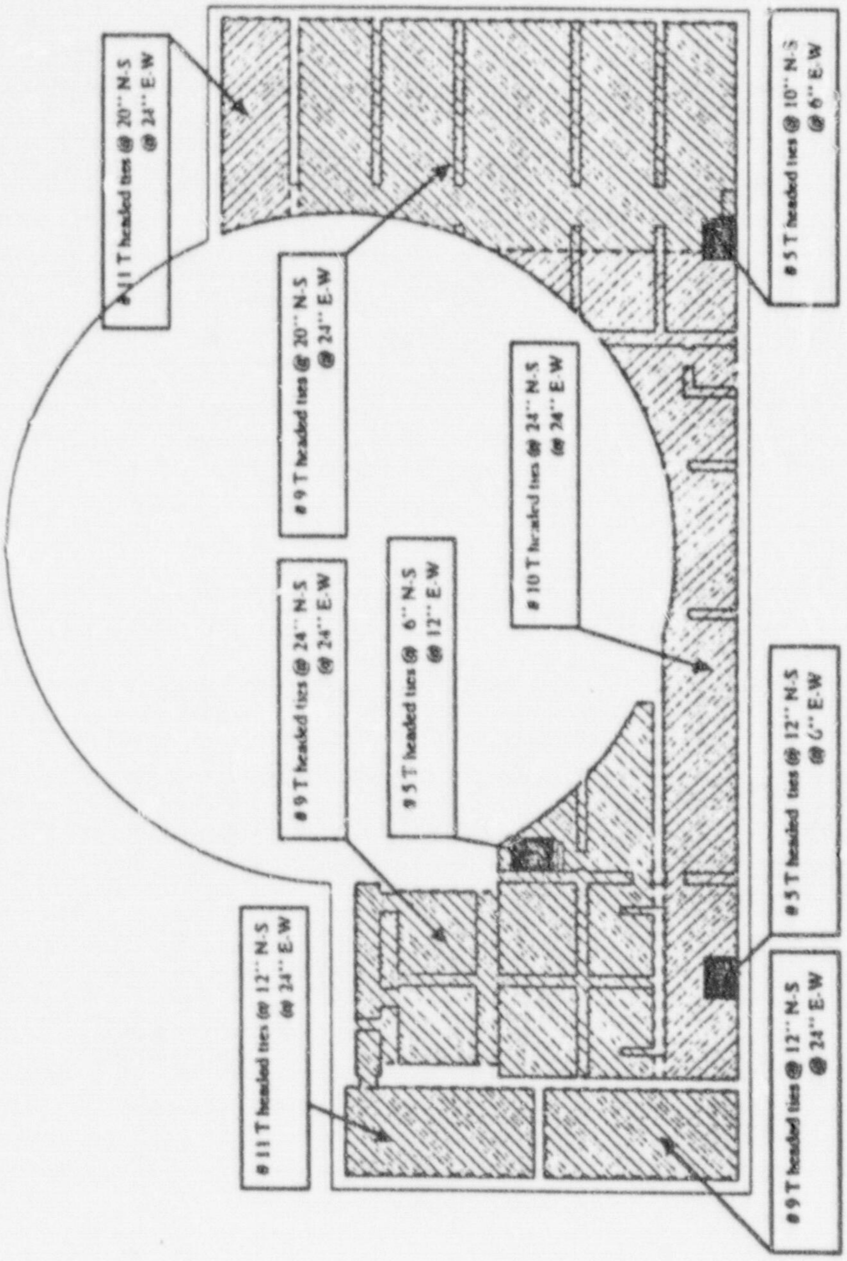


Figure 3.8.5-3 (Sheet 5 of 5)

Basemat Shear Reinforcement

SSAR Revision:

Revise subsection 3.8.4.7 as follows:

3.8.4.7 Testing and In-Service Inspection Requirements

Structures supporting the passive containment cooling water storage tank on the shield building roof will be examined before and after first filling of the tank.

- The boundaries of the passive containment cooling water storage tank and the tension ring of the shield building roof will be inspected visually for ~~any signs of leakage or distress~~ excessive concrete cracking before and after first filling of the tank. Any significant concrete cracking will be documented and evaluated in accordance with ACI 349.3R-96 (reference 50).
- The vertical elevation of the passive containment cooling water storage tank relative to the top of the shield building cylindrical wall at the tension ring will be measured before and after first filling. The change in relative elevation will be compared against the predicted deflection.
- A report will be prepared summarizing the test and evaluating the results.

There are no in-service testing or inspection requirements for other seismic Category I structures.

Revise subsection 3.8.6 as follows. This includes revision shown in response to Open Item 220.119.:

3.8.6 Combined License Information

~~This section has no requirement for additional information to be provided in support of the Combined License application.~~

The Combined License applicant will complete
~~T~~the final design of containment vessel elements (reinforcement) adjacent to concentrated masses (penetrations) ~~is completed by the Combined License applicant~~ and document~~ed~~ in the ASME Code design report.
the design

The Combined License applicant ~~should~~ will examine the structures supporting the passive containment cooling storage tank on the shield building roof during initial tank filling as described in subsection 3.8.4.7.

The Combined License applicant ~~should~~ will evaluate deviations from the design due to as-procured or as-built conditions and summarize the results of the evaluation in an as-built summary report as described in subsections 3.8.3.5.7, 3.8.4.5.3 and 3.8.5.4.4.

Add to references:

50 ACI 349.3R-96, "Evaluation of Existing Nuclear Safety-Related Concrete Structures"

FSER OPEN ITEM

220.112 Vertical sliding plate (Resolved)

The staff's concern raised under this open item is that the SSAR did not show the size of the sliding plate (which is a portion of the air baffle and is used to accommodate the differential movement between the containment vessel and the shield building) to ensure that the displacement due to seismic events will not affect the integrity of the air baffle. Westinghouse originally responded to this item in a letter dated January 7, 1998 (NSD-NRC-98-5512). The staff reviewed this response and had a concern that Westinghouse did not adequately explain why the SSAR only specified the limits of vertical movement for the sliding plate but not the limits of horizontal movement.

Due to this concern, Westinghouse revised the response to this item in a letter dated January 16, 1998 (NSD-NRC-985525). The staff reviewed this revised response during the meeting and found that Westinghouse adequately addressed their concern. Therefore, the staff considers this issue resolved.

220.113 Seismic design of the passive containment cooling water storage (PCCWS) tank (Resolved)

The staff considered this issue resolved prior to the meeting

220.114 Adequacy of seismic responses of structures due to post 72 hour changes (Resolved)

Westinghouse provided their original response to this item in a December 19, 1997, letter (NSD-NRC-97-5501). The staff's review of this submittal raised five concerns as described below:

1. SSAR Figure 3.7.2-4 should be revised to incorporate the elevations corresponding to the updated seismic model.

2. The phrase, "... and the design changes of tank structures due to the post 72 hour action requirements," should be added to the end of the last sentence of the first bullet of Section (revised) 3.7.2.2.1.

3. The SSAR should commit that if any new seismic analysis is to be performed for any site conditions, the revised model (Model B) should be used.

4. As indicated in Westinghouse's submittal (NSD-NRC-97-5251) dated July 28, 1997, the comparison of floor response spectra (FRS) from Models "A" and "B" showed that the vertical FRS at Elevations 272 ft, 284 ft, 297 ft and 307 ft from Model "B" significantly exceed (about 20 to 25 percent) those from Model "A." If the FRS at Elevations 272 ft, 284 ft and 297 ft are to be used for the design of safety-related subsystems and components (including seismic Category II piping and components), Westinghouse should either commit, in the SSAR, to use the FRS at Elevation 307 ft in the design or include the FRS at Elevations 272 ft, 284 ft and 297 ft in the SSAR.

5. In Sheet 2 of 2 of SSAR Table 3.7.2-23 (a new table), Westinghouse should include bending moments at Elevation 306.25 ft. These bending moments were shown in its submittal (NSD-NRC-97-5251) dated July 28, 1997.

Westinghouse revised the response to this item in a letter dated January 16, 1998 (NSD-NRC-98-5525). The staff reviewed the revised response during the meeting and found that Westinghouse adequately addressed their concern. Therefore, the staff considers this issue resolved.

220.115 Adequacy of floor response spectra (Resolved)

The staff's concern of this open item is related to the overall seismic analysis of the nuclear island structures. As stated in the NRC letter dated December 9, 1997, because technical issues were identified from the review of Westinghouse's seismic reanalysis (FSER Open Items 220.112F and 220.114F), this open item will not be closed until Westinghouse resolve these issues. Because the staff resolved those issues regarding the seismic reanalysis (220.112 and 220.114 above) during the meeting, the staff considers this issue resolved.

220.116 through 220.119 (Resolved)

The staff considered these issues resolved prior to the meeting.

220.120 Code case N-284 (Action N)

This issue was not discussed at the meeting. The staff had the action to review Westinghouse's response.

220.121 Design of shear studs (Action W)

Westinghouse responded to this item in a December 17, 1997, letter (NSD-NRC-97-5497). The staff reviewed this response and calculation package 1100-SUC-003, revision 1. The staff also reviewed 1100-SUC-101, revision 6, GW-SUP-003, revision 2, and 1200-SUC-101, revision 4.

The staff did develop a concern regarding concrete anchors based on the review of the response, the revision to the SSAR generated by the response, and the calculation packages.

Westinghouse standard safety analysis report (SSAR), revision 17, Section 3.8.4.5.1 provides requirements for design of concrete anchors. This section states that the design of fasteners to concrete is in accordance with ACI 349-90, Appendix B with supplementary criteria based on three other references. This section also states that anchors are designed wherever possible with sufficient depth of embedment and side cover such that the steel anchor yields prior to failure of the concrete.

The staff's concern is that the above criteria permits Westinghouse to design fasteners to concrete, including the embedded concrete anchors on the structural modules, such that the concrete fails prior to the steel yielding (i.e., non-ductile behavior). No criteria is presented in the SSAR to establish the strength for such non-ductile behavior. The staff position requires review of such criteria on a "case-by-case" basis. Westinghouse agreed to evaluate the commitments in this area made by the evolutionary plants to determine if they could make a similar commitment in their SSAR (Action W).

220.122 Critical sections for containment internal structures (Action W)

Westinghouse responded to this item in SSAR Section 3.8.3.5.7 and in a letter dated December 18, 1997 (NSD-NRC-97-5499). The staff reviewed this response, and design calculations (1100-SUC-101, revision 6, 1100-SUC-003, revision 1, and MT03-S3C-022, revision 1) during the meeting. The staff identified that the SSAR revision proposed in Westinghouse's letter dated December 18, 1997, contained several values in SSAR table 3.8.3-6 for a design load that were different than in calculation package 1100-SUC-101, revision 6. It was determined that the calculation package contained the correct information. Westinghouse took an action to check and correct the values in SSAR Table 3.8.3-6.

220.123 Implementation of design procedures in design calculations (Resolved)

Westinghouse responded to this item in a letter dated December 17, 1997 (NSD-NRC-97-5497). The staff reviewed this response and the selected design calculations (listed under documents in Attachment 2) during the meeting. Based on the review of these documents the staff considers this issue resolved.

220.124 Design calculation for the shield building and the PCCWS tank (Action N)

Westinghouse responded to this item in a letter dated December 17, 1997. This submittal is being reviewed by the staff. (Action NRC)

220.125 Vertical and radial deformation of PCCWS tank during filling. (Action W)

The staff was concerned that Westinghouse's response to this issue documented in a letter dated December 18, 1997 (NSD-NRC-97-5499) did not adequately address monitoring of the PCCWS tank during initial filling of the tank with water. The purpose of the monitoring would be to ensure that the tank responded to the addition of the water without experiencing structural problems.

Because of the staff's concern Westinghouse agreed to revise the response to 220.125 and provided a facsimile to the staff prior to the meeting (Attachment 4). The staff reviewed the response during the meeting and because of residual concerns Westinghouse agreed to add the handwritten words in section 3.8.4.7 (page 2 of Attachment 4). Westinghouse agreed to evaluate if a reference to the maintenance rule for the PCCWS tank is necessary (Action W).

220.126 Air baffle evaluation for air flow fluctuations (Resolved)

The staff reviewed Westinghouse's response to this item documented in a December 17, 1997 letter (NSD-NRC-97-5497) and the June 11, 1997, letter that is referenced in this response. The staff found Westinghouse's response to this item acceptable and therefore considers the issue resolved.

220.127 The staff considered this issue resolved prior to the meeting

The staff reviewed Westinghouse's response to this item documented in a letter dated December 17, 1997 (NSD-NRC-97-5497). The staff found Westinghouse's response to this item acceptable and therefore considers the issue resolved.

220.128 Auxiliary building roof slab (Action W/Action N)

The staff's concern of this item is that Westinghouse should provide a design summary of critical sections in the SSAR. Westinghouse responded to this item in SSAR Sections 3.8.3.5.7 and 3.8.5.4.4, and in the proposed Appendix 3H attached to the letter dated January 9, 1998 (NSD-NRC-98-5515). The staff also reviewed calculation package 1200-SUC-101, revision 4. The staff did not complete its review of this item during the meeting. Therefore, the staff took an action to complete its review of this response.

However, the staff did identify changes that needed to be made to the SSAR. Westinghouse provided Attachment 5 to address the staff's concerns. Westinghouse took an action to incorporate these changes into the SSAR.

220.129 Adequacy of foundation mat (Action N)

220.130 Consideration of loads due to construction sequence and settlements in the foundation mat design (Action W)

Westinghouse responded to this item in its letter dated January 7, 1998 (NSD-NRC-98-5512). In this letter, Westinghouse referred to a letter dated October 17, 1997, and restated its position for the resolution of the staff's concern regarding the design of the nuclear island foundation mat under construction loads. The staff reviewed Westinghouse's response and found them to be unacceptable. The staff's position is that in designing the foundation mat for construction loads, Westinghouse should follow the five-steps procedure agreed to during the meeting on August 4 through 8, 1997. A conference call was held to discuss the issue further. Participants in the call included: Don Lindgren, Richard Orr, Tom Cheng, Joe Sebrosky, and Carl Constantino.

As described in the submittal dated October 17, 1997, Westinghouse selected the five most critical locations (locations with the highest stresses under the combined load conditions) and demonstrated that by following the five-step procedure the original design has enough margin to cover the inclusion of construction loads. As a result of the conference call, the staff agreed that the five locations that Westinghouse chose were the appropriate areas for the demonstration and the calculations properly demonstrated that the design capacity of the foundation mat can cover the additional stresses induced by construction loads. However, the staff did not agree with the design procedure for the construction loads documented in the SSAR. To resolve this concern the staff stated that Westinghouse should incorporate the five-step procedure agreed to during the meeting on August 4 through 8, 1997 (documented in the meeting summary dated September 30, 1997) in the SSAR. Westinghouse took an action to evaluate providing the five-step design procedure for considering construction loads in the SSAR.

FSER Open Item

Open Item 220.125F (OITS #6312) **Response Revision 1**

Because a massive amount of water is to be contained in the PCCWS tank, the staff raised a concern that the COL applicant should monitor the vertical and radial deformation of the tank during initial filling, and compare the measured values with the tank deformation predicted by calculation. The staff identified this issue as Open Item 3.8.4.4-3 and COL Action Item 3.8.4.4-1.

At the meeting on June 12 through 16, 1995, Westinghouse stated that the water weight is small, in comparison with the total weight of the shield building roof structure (estimated to be about 10 percent). Westinghouse also showed that the deflection of the roof structure resulting from the first fill of water should be negligible. On that basis, Westinghouse contended that there is no need to monitor the tank deflections and compare the deflections against predictions.

During the meeting on December 9 through 13, 1996, Westinghouse repeated its justification concerning this issue. However, the staff did not agree with Westinghouse's basis for not monitoring the vertical and radial deformation of the tank during initial tank filling. Moreover, the staff asserted that post-construction testing is necessary to confirm the adequacy of the PCCWS tank. This is because the staff's review experience suggest that the excessive deformation resulting from the massive amount of water may cause cracking of the tank wall and base slab, as well as water leakage from reinforced concrete tanks with steel liners.

In Revision 17 of SSAR Section 3.8.4.1.1, Westinghouse added a statement that leak chase channels are provided over the liner welds to permit monitoring for leakage and to prevent degradation of the reinforced concrete wall which might result from the freezing and thawing of leakage. Also, Westinghouse indicated that the exterior face of the reinforced concrete boundary of the PCCWS tank is designed to control cracking, in accordance with Paragraph 10.6.4 of ACI-349, with reinforcement steel stress based on sustained loads (including thermal effects). However, Westinghouse still did not commit to monitor the vertical and radial deformation of the tank during initial filling and compare the measured values with the tank deformation predicted by analysis. On the basis of the above discussion, the staff concluded that Westinghouse's response to the staff's concern (as stated in Revision 17 of SSAR Section 3.8.4.1.1) is not acceptable. Therefore, Open Item 3.8.4.4-3 and COL Action Item 3.8.4.4-1 remain unsolved.

Response (Revision 1):

The SSAR is revised below to show monitoring of the tank during initial filling. Requirements for visual examination are given. The calculated deflections of the roof structure due to the first fill of water are less than one quarter of an inch. Monitoring of tank deflections and comparison against predictions is difficult because of the small magnitude of the deflections due to the water inventory. Vertical deflections could also be caused by thermal changes. The vertical deflection will be measured during tank fill and will be compared to the predicted magnitude. This will be used in combination with the visual examination to confirm acceptability.

DRAFT

Comments given Don Lindgren 1/20/98

FSER Open Item

SSAR Revision:

Revise subsection 3.8.4.7 as follows:

3.8.4.7 Testing and In-Service ~~Testing and~~ Inspection Requirements

Structures supporting the passive containment cooling water storage tank on the shield building roof will be examined before and after first filling of the tank.

- The boundaries of the passive containment cooling water storage tank and the tension ring of the shield building roof will be inspected visually for excessive concrete cracking before and after first filling of the tank. Any significant concrete cracking will be documented and evaluated in accordance with ACI 349.3R-96 (reference 50).

- The vertical elevation of the passive containment cooling water storage tank relative to the top of the shield building cylindrical wall at the tension ring will be measured before and after first filling. The change in relative elevation will be compared against the predicted deflection.

- A report will be prepared summarizing the test and evaluating the results.

There are no in-service testing or inspection requirements for the seismic Category I structures, but during the operation of the plant, it is necessary to monitor the performance or condition of these structures against license-established goals in a manner sufficient to provide reasonable assurance that such structures are capable of fulfilling their intended functions.

Revise subsection 3.8.6 as follows:

3.8.6 Combined License Information

~~This section has no requirement for additional information to be provided in support of the Combined License application.~~ The COL applicant should examine the structures supporting the passive containment cooling storage tank on the shield building roof during initial tank filling as described in subsection 3.8.4.7.

Add to references:

50 ACI 349.3R-96, "Evaluation of Existing Nuclear Safety-Related Concrete Structures"

DRAFT

- In Section Q1.5.8, for constrained members (rotation and/or displacement constraint such that a thermal load causes significant stresses) supporting safety-related structures, systems, or components, the stresses under load combinations 9, 10, and 11 are limited to those allowed in Table Q1.5.7.1 as modified above.

3H.4 GLOBAL SEISMIC ANALYSES

A global seismic analysis of the AP600 nuclear island structure is performed to obtain building seismic response spectra for the seismic design of nuclear safety-related structures. This analysis is described in subsection 3.7.2. For determining the out-of-plane seismic loads on slabs and wall segments, spectral accelerations are obtained from the relevant response spectra, using the 7 percent damping curve. Hand calculations are performed to estimate the out-of-plane seismic forces and the corresponding bending moment in each shear wall and floor slab element to supplement the loads obtained from the response spectra analyses.

The in-plane seismic loads for the design of the shear walls and the slabs in the auxiliary building are based on a response spectrum analysis of the auxiliary building and the shield building 3D finite element models. The response spectrum analyses are performed for two cases: one that considers the reinforced concrete elements to be uncracked with full elastic stiffness, and the other that models the elements with 70 percent of their full stiffness. The larger of the two values for each finite element, from these two cases, for the stress resultants is used in the design evaluation.

INSERT 3H.4.1 Live load for seismic design

see next page

3H.5 STRUCTURAL DESIGN OF CRITICAL SECTIONS

This subsection summarizes the structural design of representative seismic Category I structural elements in the auxiliary building and shield building. These structures are listed below and the corresponding location numbers are shown on Figure 3H.5-1. The basis for their selection to this list is also provided for each structure.

(1) South wall of auxiliary building (column line 1), elevation 66'-6" to elevation 180'-0". (This exterior wall illustrates typical loads such as soil pressure, surcharge, temperature gradients, seismic, and tornado.)

(2) Interior wall of auxiliary building (column line 7.3), elevation 66'-6" to elevation 160'-6" (This is one of the most highly stressed shear walls.)

(3) West wall of main control room in auxiliary building (column line L), elevation 117'-6" to elevation 153'-0". (This illustrates design of a wall for subcompartment pressurization.)

(4) North wall of MSIV east compartment (column line 11), elevation 117'-6" to 153'-0". (The main steam line is anchored to this wall segment.)

(5) Shield building cylinder at elevation 180'-0".

Attachment 5

3.8.4.3.1.5 Dynamic Effects of Abnormal Loads

The dynamic effects from the impulsive and impactive loads caused by P_a, R_a, Y_r, Y_j, Y_m, and tornado missiles are considered by one of the following methods:

- Applying an appropriate dynamic load factor to the peak value of the transient load
- Using impulse, momentum, and energy balance techniques
- Performing a time-history dynamic analysis

Elastoplastic behavior may be assumed with appropriate ductility ratios, provided excessive deflections will not result in loss of function of any safety-related system.

Dynamic increase factors appropriate for the strain rates involved may be applied to static material strengths of steel and concrete for purposes of determining section strength.

3.8.4.3.2 Load Combinations

3.8.4.3.2.1 Steel Structures

The steel structures and components are designed according to the elastic working stress design methods of the AISC-N690 specification using the load combinations specified in Table 3.8.4-1.

3.8.4.3.2.2 Concrete Structures

The concrete structures and components are designed according to the strength design methods of ACI-349 Code, using the load combinations specified in Table 3.8.4-2.

3H.4.1

~~3.8.4.3.2.3~~ Live Load for Seismic Design

the response due to

whichever is applicable.

Include this paragraph as 3H.4.1 in Appendix 3H

Floor live loads, based on requirements during plant construction and maintenance activities, are specified varying from 50 to 250 pounds per square foot ~~(with the exception of the containment operating deck which is designed for 800 pounds per square foot specified for plant maintenance condition)~~.

seismic loads include masses equal to 25 percent of the floor live loads or 75 percent of the roof snow load. These seismic loads are combined with

For the local design of members, such as the floors and beams, ~~live loads in combination with the safe shutdown earthquake are taken as~~ 100 percent of the~~se~~ specified live loads, or 75 percent of the roof snow load, whichever is applicable, except in the case of the containment operating deck. ~~For the seismic load combination, the containment operating deck is designed for a live load of 200 pounds per square foot which is appropriate for plant operating condition~~. These live and snow loads are included as mass in calculating the vertical seismic forces on the floors and roof. The mass of equipment and distributed systems is included in both the dead and seismic loads.

(6) Roof slab at elevation 180'-0" adjacent to shield building cylinder. (This is the connection between the two buildings at the highest elevation.)

(7) Floor slab on metal decking (elevation 135'-3")
(This is a typical slab on metal decking and structural steel framing.)

(8) 2'-0" slab in auxiliary building (tagging room ceiling at elevation 135'-3")
(This illustrates the design of a typical 2'-0" thick concrete slab.)

(9) Finned floor in the main control room at elevation 135'-3"
(This illustrates the design of the finned floors.)

(10) Shield building roof/PCCS water storage tank
(This is a unique area of the roof and water tank.)

(11) Shield building roof to cylinder location at columns
(This is the junction between the shield building roof and the cylindrical wall of the shield building.)

(12) Divider wall between the spent fuel pool and the fuel transfer canal. (This wall is subjected to thermal and seismic sloshing loads)

3H.5.1 Shear Walls

Structural Description

Shear walls in the auxiliary building vary in size, configuration, aspect ratio, and amount of reinforcement. The stress levels in shear walls depend on these parameters and the seismic acceleration level. The range of these parameters and the stress levels in various regions of the most severely stressed shear wall are described in the following paragraphs.

The height of the major structural shear walls in the auxiliary building ranges between 30 to 120 feet. The length ranges between 40 and 260 feet. The aspect ratio of these walls (full height/full length) is generally less than 1.0 and often less than 0.25. Therefore, these walls fall within the category of low rise shear walls. The walls are typically 2 to 5 feet thick, and are monolithically cast with the concrete floor slabs, which are 9 inches to 2 feet thick. Exterior shear walls are several stories high and do not have many large openings. Interior shear walls, however, are discontinuous in both vertical and horizontal directions. The in-plane behavior of these shear walls, including the large openings, is adequately represented in the analytical models for the global seismic response. Where the refinement of these finite element models is insufficient for design of the reinforcement, for example in walls with a large number of openings, detailed finite element models are used. The shear walls are used as the primary system for resisting the lateral loads, such as earthquakes. The auxiliary building shear walls are also evaluated for flexure due to the out-of-plane loads.

3H.5.1.1 Exterior Wall at Column Line 1

The wall at column line 1 is the exterior wall at the south end of the nuclear island. The reinforced concrete wall extends from the top of the basemat at elevation 66'-6" to the roof at elevation 180'-0". It is 3'-0" thick below the grade and 2'-3" thick above the grade.

(static and dynamic)

The wall is designed for the applicable loads including dead load, live load, hydrostatic load, lateral soil pressure loads, seismic loads, and thermal loads. As shown in Figure 3H.5-2, the wall is divided in 12 segments for design purpose. Table 3H.5-2 provides the listing and magnitude of the various design loads. Table 3H.5-3 presents the governing load combination for each wall segment and the details of the wall reinforcement. The actual reinforcement provided is compared to the required rebar area for each wall segment. Figure 3H.5-3 shows the typical reinforcement for the wall at column line 1.

3H.5.1.2 Wall at Column Line 7.3

The wall at column line 7.3 is a shear wall that connects the shield building and the nuclear island exterior wall at column line I. It extends from the top of the basemat at elevation 66'-6" to the top of the roof. The wall is 3 feet thick below the grade at elevation 100'-0" and 2 feet thick above the grade. Out-of-plane lateral support is provided to the wall by the floor slabs on either side of it and the roof at the top.

Wall 7.3 is designed for the applicable loads described in subsection 3H.3.3.

For various segments of this wall, the corresponding governing load combination and associated design loads are shown in Table 3H.5-4.

Table 3H.5-5 presents the details of the wall reinforcement. The actual reinforcement provided is compared to the required reinforcement area for each wall segment. Typical wall reinforcement is also shown on Figure 3H.5-4

3H.5.1.3 Wall at Column Line L

The wall at column line L is a shear wall on the west side of the Main Control Room. It extends from the top of the basemat at elevation 66'-6" to the top of the roof. The wall is 2 feet thick. Out-of-plane lateral support is provided to the wall by the floor slabs on either side of it and the roof at the top. The segment of the wall that is a part of the main control room boundary is from elevation 117'-6" to elevation 135'-3".

The auxiliary building design loads are described in subsection 3H.3.3, and the wall is designed for the applicable loads. In addition to the dead, live and seismic loads, the wall is designed to withstand a 5 pounds per square inch pressure load due to a pipe break in the MSIV room even though it is a break exclusion area. This wall segment is also designed to withstand a jet load due to the pipe break.

The governing load combination and associated design loads are shown in Table 3H.5-6.

Table 3H.5-7 presents the details of the wall reinforcement. The actual reinforcement provided is compared to the required reinforcement area for each wall segment.

3H.5.1.4 Wall at Column Line 11

The north wall of the MSIV east compartment, at column line 11 between elevation 117'-6" and elevation 153'-0", has been identified as a critical section.

The segment of the wall between elevation 117'-6" and elevation 135'-3" is 4 feet thick, and several pipes such as the main steam line, main feed water line, and the start-up feed water line are anchored to this wall at the interface with the turbine building.

The wall segment from elevation 135'-3" to elevation 153'-0" does not provide support to any high energy lines, and is 2 feet thick. This portion does not have to withstand reactions from high energy line breaks.

The wall is designed to withstand loads such as the dead load, live load, seismic load and the thermal load. The MSIV room is a break exclusion area, but the design also considered the loads associated with pipe rupture in the MSIV room, such as compartment pressurization, jet load, and the reactions at the pipe anchors. The loads on the pipe anchor include pipe rupture loads for breaks in the turbine building.

The wall structure is analyzed using three dimensional finite element analyses. Analyses are performed for individual loads, and design loads are determined for applicable load combinations from Table 3.8.4-2. ~~The design is performed for the enveloping cases for critical regions~~.

~~General features of the~~ Typical wall reinforcement ~~are~~ is shown in Figure 3H.5-5.

3H.5.1.5 Shield Building Cylinder at Elevation 180'-0"

The thickness of the cylindrical portion of the shield building wall is 3 feet.

The wall is designed for the applicable loads described in subsection 3H.3-3. A detailed finite element analysis is performed to determine the design forces. The amount of reinforcement in horizontal and vertical directions provided on each face is same. Typical reinforcement from elevation 200'-0" to 160'-6", above the auxiliary building roof, on each face, is as follows:

Elevation 200'-0" to 180'-6": Required horizontal reinforcement = 3.45 inch^2/ft.
Provided horizontal reinforcement = 3.81 inch^2/ft.

Required vertical reinforcement = 3.71 inch^2/ft.
Provided vertical reinforcement = 3.81 inch^2/ft.

or continuous. The seismic load is obtained using the applicable floor acceleration response spectrum (7 percent damping for the SSE loads).

The load combinations applicable to the design of these floors are shown in Tables 3.8.4-1 and 3.8.4-2. The design of the floor system is performed in two parts:

- Design of structural steel beams
  - The structural steel floor beams are evaluated to withstand the weight of wet concrete during the placement of concrete. The composite section is checked for the design loads during normal and extreme environment conditions. Shear connectors are also designed.
- Design of concrete slab
  - The concrete slab and the steel reinforcement of the composite section are evaluated for normal and extreme environmental conditions. The slab concrete and the reinforcement is designed to meet the requirements of American Concrete Institute standard ACI 349-90 "Code Requirements for Nuclear Safety-related Structures."
  - The slab design considers the in-plane and out-of-plane seismic forces. The global in-plane and out-of-plane forces are obtained from the response spectrum analysis of the 3D finite element model of the auxiliary and shield buildings. The out-of plane seismic forces due to floor self-excitation are determined by hand calculations using the applicable vertical seismic response spectrum and slab frequency.

3H.5.2.1 Roof at Elevation 180'-0", Area c (Critical Section is between Col. Lines N & K-2 and 3 & 4)

The layout of this segment of the roof is shown in Figure 3H.5-7 as Region "B." The concrete slab is 15 inches thick, plus 4.5-inch deep metal deck ribs. It is composite with 5 feet deep plate girders, spaced 14'-2" center to center, by using shear connectors. The girder flanges are 20" x 2" and the web is 56" x 7/16." The girders span approximately 64 feet in the north-south direction and are designed as simply supported. The concrete slab between the girders behaves as a one-way slab and is designed to span between the girders.

The roof girders are designed for dead and live loads, including construction loads (with wet concrete) with simple support end conditions. A one-third increase in allowable stress is permitted for the construction load combination.

The girders are also evaluated as part of the composite beam after drying of concrete. The composite roof structure is designed to withstand dead and live load / snow load, as well as the wind, tornado and seismic loads.

serve as the formwork and withstand the load of wet concrete slab. The main reinforcement is provided in the precast panels which are connected to the concrete placed above it by shear reinforcement. The precast panels and the cast-in-place concrete act together as a composite reinforced concrete slab. Examples of such floors are the Tagging Room ceiling slab at elevation 135 ft 3 inches in Area 2, and the Area 5/6 elevation 100'-0" slab between column lines 1 & 2.

3H.5.3.1 Tagging Room Ceiling

ADD FIGURE showing slab configuration Refer to SSAR 1.2 el. 117' for location

Design dimensions of the Tagging Room Ceiling are as follows:

Room Size:	16'-0" x 11'-10"
Boundary Conditions:	Fixed at Walls J and K
Clear Span:	16'-0"
Slab Thickness:	Total = 24 inches Precast Panel = 8 inches Cast-in-Place = 16 inches

The two precast concrete panels, each 5'-11" wide and spanning over 16'-0" clear span, are installed to serve as the formwork.

Design of the Precast Concrete Panels:

Governing Load Combination	= Construction
Design Bending Moment (Midspan)	= 14.53 ft-kip/ft.
Bottom Reinforcement (E/W Direction) Required	= 0.51 in^2/ft.
Bottom Reinforcement (E/W Direction) Provided	= 0.79 in^2/ft.
Top Reinforcement (E/W Direction) Required	= (Minimum required by Code)
Top Reinforcement (E/W Direction) Provided	= 0.20 in^2/ft.
Top and Bottom Reinforcement (N/S Direction) Required	= (Minimum required by Code)
Top and Bottom Reinforcement (N/S Direction) Provided	= 0.20 sq. in/ft.

the steel and concrete act as a composite section. The fins are exposed to the environment of the room and enhance the heat-absorbing capacity of the ceiling. Several shop-fabricated steel panels, cut to room width and placed side by side perpendicular to the room length, are used to construct the stiffened plate ceiling in a modularized fashion. The stiffened plate with fins is designed to withstand construction loads prior to concrete hardening.

The main control room ceiling fin floor is designed for the dead, live, and the seismic loads.

move Figure 3.8.4-6 into Appendix 3H

The finned floor structure is evaluated for the load combinations listed in Tables 3.8.4-1 and 3.8.4-2.

Design Methodology

Define fin size and reinforcement size

The finned floors are designed as reinforced concrete slabs in accordance with ACI Standard 349. For positive bending, the steel plate is in tension. The steel plate with fin stiffeners serves the function of bottom rebars. For negative bending, the potential for buckling due to compression in this element is checked by using the criteria of American National Standards Institute/American Institute of Steel Construction standards ANSI/AISC N690-84. Twisting, and therefore lateral buckling of the stiffener, is restrained by the concrete.

The finned floors resist vertical and in-plane forces for both normal and extreme loading conditions. For positive bending, the concrete above the neutral axis carries compressive stresses and the stiffened steel plate resists tension. Negative bending compression is resisted by the stiffened plate and tension by top rebars in the concrete. The neutral axis for negative bending is located in the stiffened plate section, and the concrete in tension is assumed inactive. Horizontal in-plane forces are resisted by the stiffened plate and longitudinal rebars.

Minimum top reinforcement is provided in the slab in each direction for shrinkage and temperature crack control. In addition, top reinforcement located parallel to the stiffeners is used as tension reinforcement in negative bending. The stiffened plate provides crack control capability for the bottom of the slab in the transverse direction.

Composite section properties, based on an all steel-transformed section, as detailed in Section Q1.11 of ANSI/AISC N690-84, are used to check the following:

- Weld strength between stiffener and the steel plate
- Spacing of the shear studs for the composite action

The stiffened plate alone is designed to resist all construction loads prior to the concrete hardening. The plate is checked against the criteria for bending and shear, specified in ANSI/AISC N690-84, Sections Q1.5.1.4 and Q1.5.1.2. In addition, the weld between the stiffener and the steel plate is checked to satisfy the code requirements.

3H.5.5 Structural Modules

as shown in Figure 3.8.3-2

Structural modules are used for ~~part of~~ some of the structural elements on the south side of the auxiliary building. These structural modules are ~~structural elements~~ built up with welded steel structural shapes and plates. The modules consist of steel faceplates connected by steel trusses. The primary purpose of the trusses is to stiffen and hold together the faceplates during handling, erection, and concrete placement. The ~~nominal~~ thickness of the steel faceplates is 0.5 inch except in a few local areas. The nominal spacing of the trusses is 30 inches. Shear studs are welded to the inside faces of the steel faceplates. Face plates are welded to adjacent plates with full penetration welds so that the weld is at least as strong as the plate. The structural wall modules are anchored to the concrete base by reinforcing steel dowels or other types of connections embedded in the reinforced concrete below. After erection, concrete is placed between the faceplates.

~~Provide reference to Figure 3.8.3~~

face

These modules include the spent fuel pool, fuel transfer canal, and cask loading and cask washdown pits. The structural modules are similar to the structural modules for the containment internal structures (see description in subsection 3.8.3 and Figures 3.8.3-8, 3.8.3-14, 3.8.3-15 and 3.8.3-17). Figure 3.8.4-5 shows the location of the structural modules in the auxiliary building. The structural modules extend from elevation 66'-6" to elevation 135'-3".

The loads and load combinations applicable to the structural modules in the auxiliary building are the same as for the containment internal structures (subsection 3.8.3.5.3) except that there are no ADS nor pressure loads due to pipe breaks.

The design methodology of these modules in the auxiliary building is similar to the design of the structural modules in the containment internal structures described in subsection 3.8.3.5.3.

3H.5.5.1 West Wall of Spent Fuel Pool

Figure 3H.5-8 shows an elevation of the west wall of the spent fuel pool (column line L-2), and element numbers in the finite element model. The wall is a 4 feet thick concrete filled structural wall module.

A finite element analysis of the spent fuel building module is performed for seismic, thermal and hydrostatic loads with the following assumptions:

- The analysis model includes the structure between Lines 2 and 4, Lines I and N, and between El. 66'-6" and 135'-3", and is fixed at the base. There is no support at elevation 135'-3"
- The seismic input consists of floor response spectra derived from the spectra for the floor at El. 135'-3", which are conservatively applied at the basemat level as ground response spectra.

- The thermal loads are applied as linearly varying temperatures between the inner and outer faces of the walls and floors.

- The hydrostatic loads are applied to the spent fuel pool walls and floors, which is considered full with water. This provides the loads for the design of the divider wall.

- The seismic sloshing is modeled in the spent fuel pool.

The concrete filled structural wall modules are designed as reinforced concrete structures in accordance with the requirements of ACI-349. The face plates are treated as reinforcing steel.

Methods of analysis are based on accepted principles of structural mechanics and are consistent with the geometry and boundary conditions of the structures. Both computer codes and hand calculations are used.

Table 3H.5-8 shows the magnitude of typical design loads, load combinations, and the required and provided plate thickness for certain critical locations. The steel plates are generally half inch thick. The plate thickness is increased close to the bottom of the gate through the wall where the opening results in high local member forces. The first part of the table shows the member forces due to individual loading. The lower part of the table shows the governing load combinations. The steel plate thickness required to resist mechanical loads is shown at the bottom of the table as well as the thickness provided. The maximum principal stress for the load combination including thermal is also tabulated. If this value exceeds the yield stress at temperature, a supplemental evaluation is performed. For these cases, the maximum stress intensity range is shown together with the allowable stress intensity range which is twice the yield stress at the temperature.

3H.5.6 Shield building roof

The shield building roof is a reinforced concrete shell supporting the passive containment cooling system tank and air diffuser. Air intakes are located at the top of the cylindrical portion of the shield building. The conical roof supports the passive containment cooling system tank as shown in Figure 3.8.4-7. The design of critical areas is discussed below. These areas include the tension ring at the connection of the conical roof to the cylindrical wall, the columns between the air inlets just below the air inlets, and the exterior wall of the passive containment cooling system tank.

3H.5.6.1 Tension ring

air inlet columns

The connection between the conical roof and the ~~shield building cylindrical wall~~ is designated as the tension ring. It spans as a beam across the air inlets. The governing load for the tension ring is axial tension. The maximum tension is about 1100 kips under normal operating loads. SSE seismic loads result in maximum axial loads of about 1800 kips. The combined load ranges from 2900 kips tension to 800 kips compression. The maximum axial tension results in a reinforcement stress of 34 ksi. The reinforcement will also see tensile stresses due to other member force components, primarily torsion and bending about the horizontal axis. The maximum axial compression results in a concrete compressive stress of 380 psi. This is less than 10 percent of the concrete compressive strength. The ring is designed as a tension member; shear stirrups are provided to carry the shear and torsion without taking credit for concrete shear strength. The reinforcement is shown in Figure 3H.5-9. The reinforcement required and provided is summarized in sheet 1 of Table 3H.5-9.

3H.5.6.2 Column (shear wall) between air inlets

The column between the air inlets has plan dimensions of 36" x 138" and is 60" high. Its primary loading is vertical load due to dead and seismic loads and horizontal seismic shear. It is designed as a low rise shear wall. The axial compression is about 1200 kips under normal operating loads. SSE seismic loads result in maximum axial loads of about 1700 kips. The combined load ranges from 2900 kips compression to 500 kips tension. The maximum horizontal shear is 2200 kips in-plane and 800 kips out-of-plane (D.L. = 300, SSE = 500). The 2900 kips compression corresponds to an axial compressive stress of about 600 psi. These loads and the associated bending moments result in a maximum concrete compressive stress of 1400 psi and a maximum concrete tensile stress of 800 psi at the base of the column assuming gross concrete section properties. The reinforcement is shown in the Figure 3H.5-9. The reinforcement required and provided is summarized in sheet 2 of Table 3H.5-9.

3H.5.6.3 Exterior wall of the passive containment cooling system tank

The exterior wall of the passive containment cooling system tank is two feet thick. There is a stainless steel liner on the inside surface of the tank. The wall liner consists of a plate with stiffeners and welded studs on the concrete side of the plate. Leak chase channels are provided over the liner welds. The reinforcement in the concrete wall is designed without taking credit for the strength provided by the liner. The governing loads for design of the exterior wall are the hydrostatic pressure of the water, the in-plane and out-of-plane seismic response, and the temperature gradient across the wall. The reinforcement required and provided is summarized in sheet 3 of Table 3H.5-9.

Expand this section and describe connection to conical roof.

Table 3H.5-7

Interior Wall on Column Line L
Details of Wall Reinforcement

Wall Segment	Type ~~Location~~	Reinforcement on Each Face, sq.in./ft. Required	Provided
Elevation 117'-6" to 135'-3"	Horizontal	3.54	3.72
	Vertical	4.74	5.12
Elevation 135'-3" to Roof	Horizontal	1.81	2.00
	Vertical	2.19	2.56

Shear Reinforcement:

Wall Segment	Type ~~Location~~	Reinforcement, sq.in./ft. Required	Provided
Elevation 117'-6" to 135'-3"	~~Through wall thickness~~ ~~In east-west direction~~ T-Headed stirrups	0.88	1.2

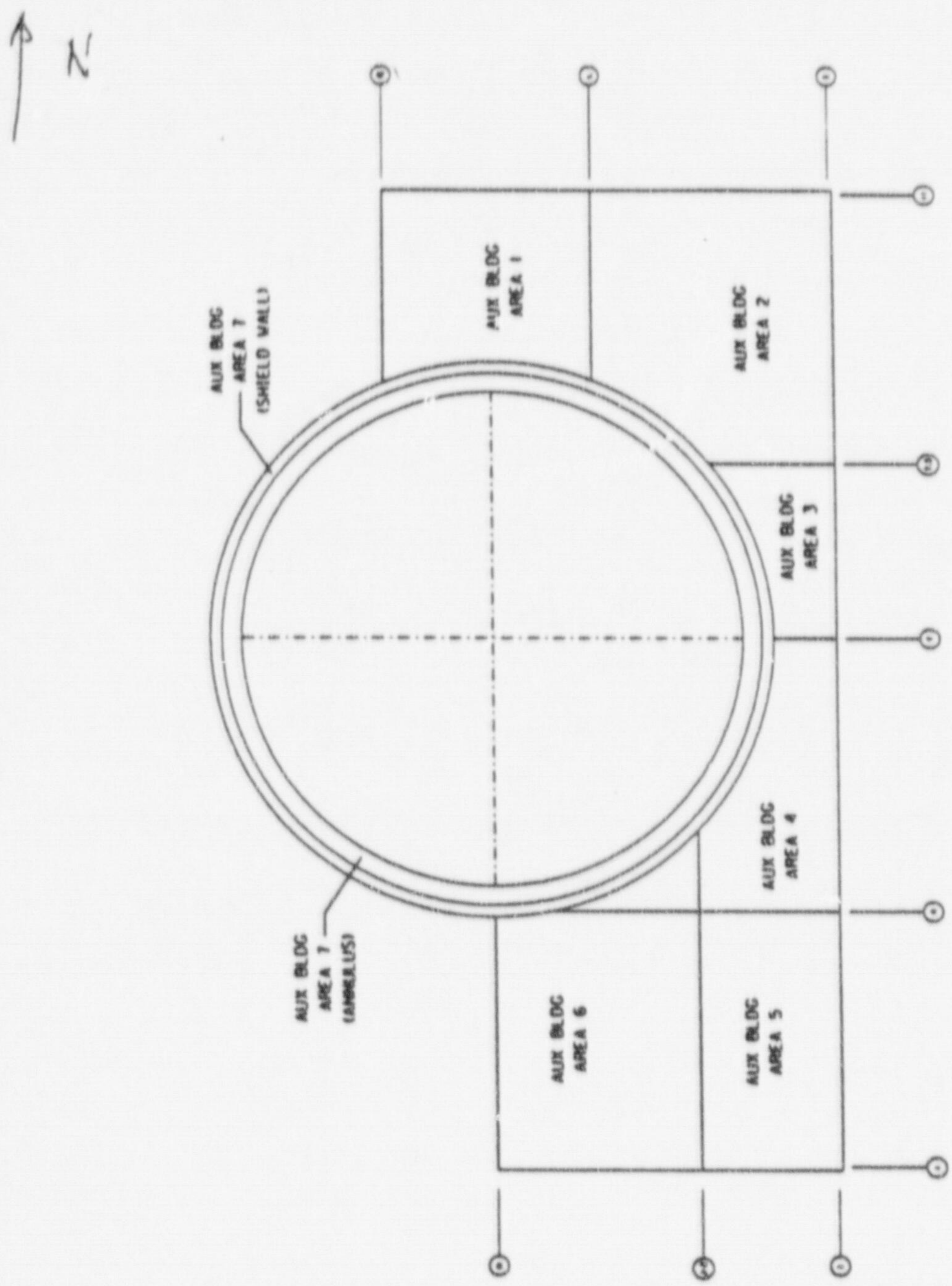


Figure 3H.2-1

General Layout of Auxiliary Building

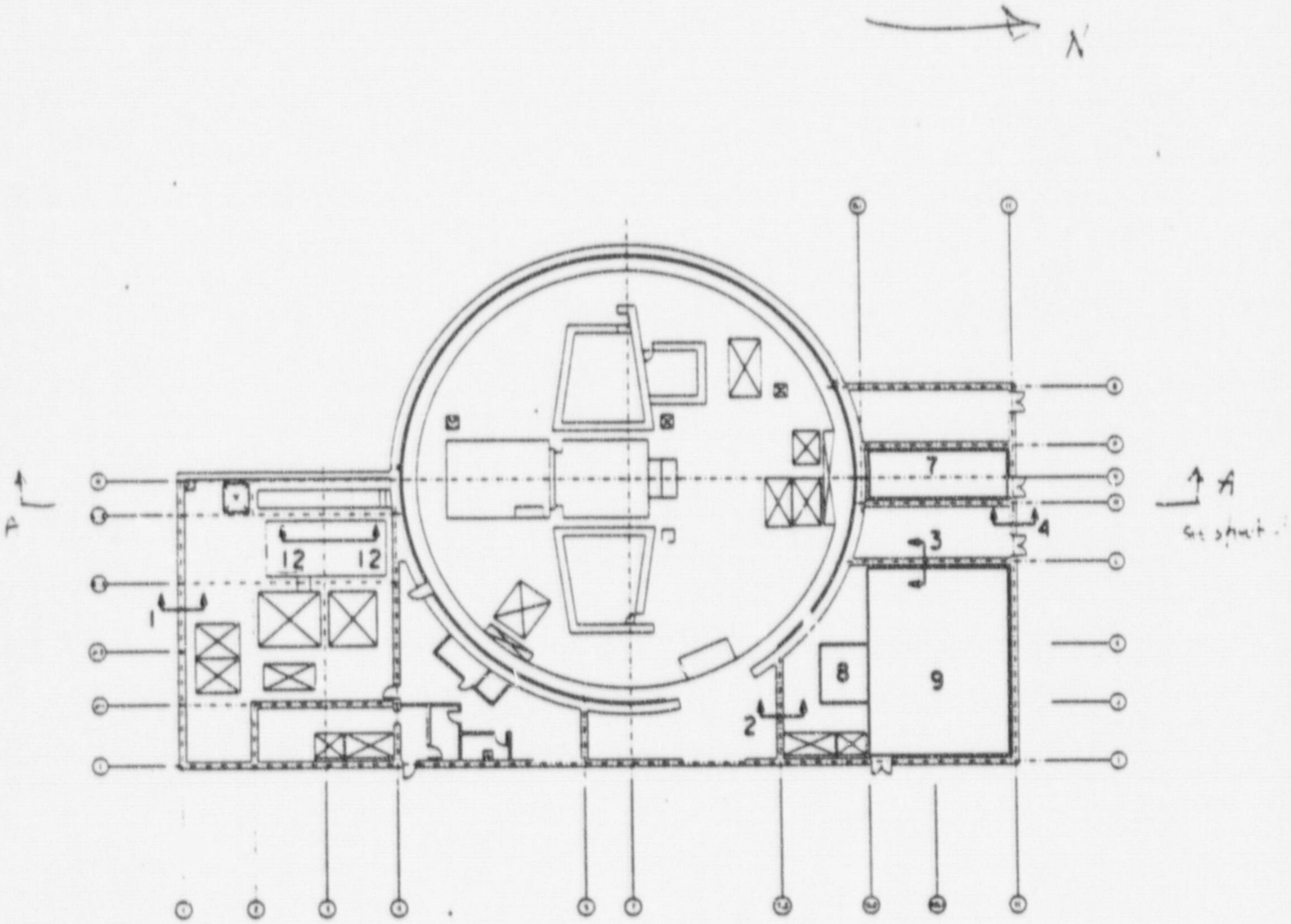


Figure 3H.5-1 (Sheet 1 of 3)

Nuclear Island Critical Sections
Plan at El. 135'-3"

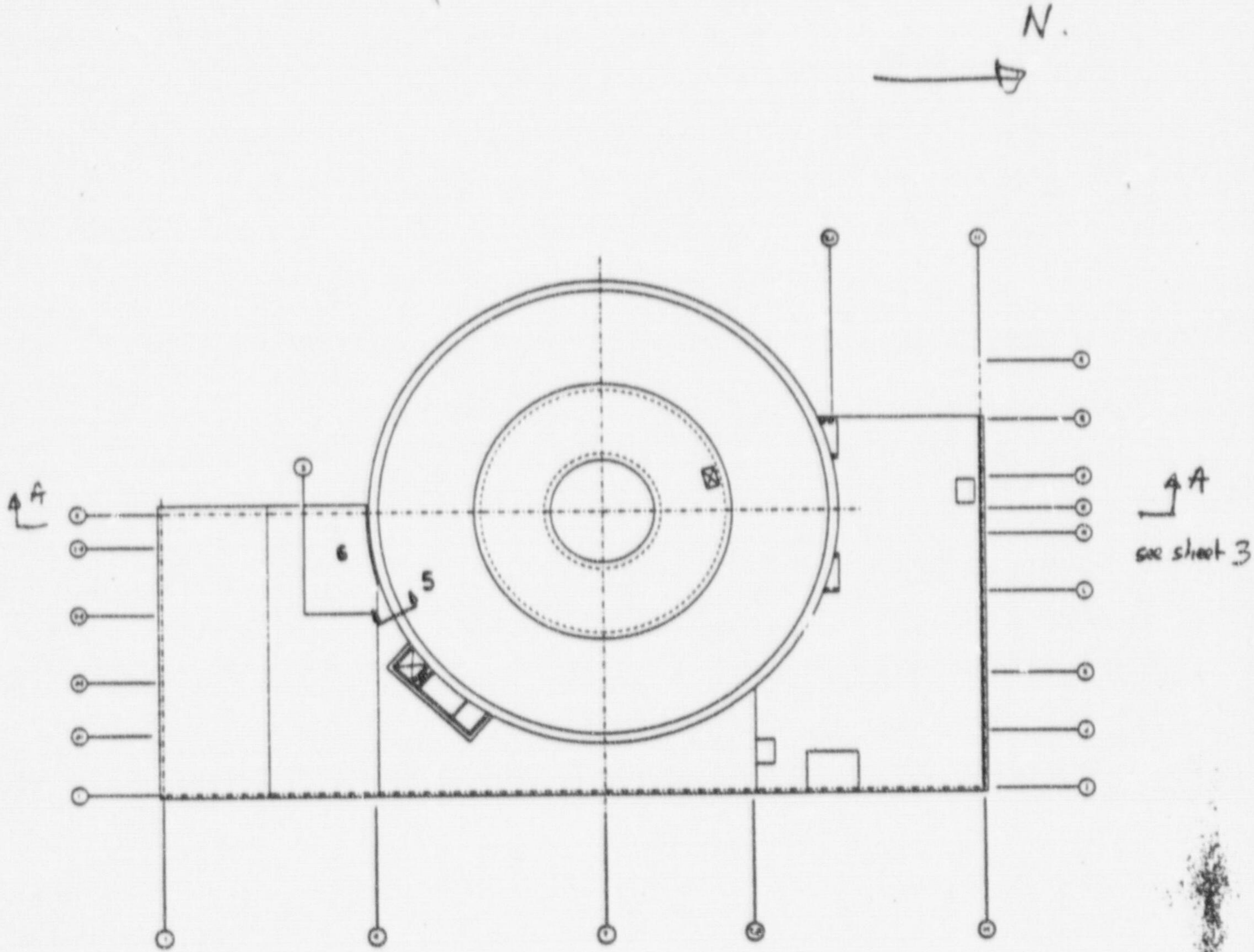


Figure 3H.5-1 (Sheet 2 of 3)

Nuclear Island Critical Sections
Plan at El. 180'-0"

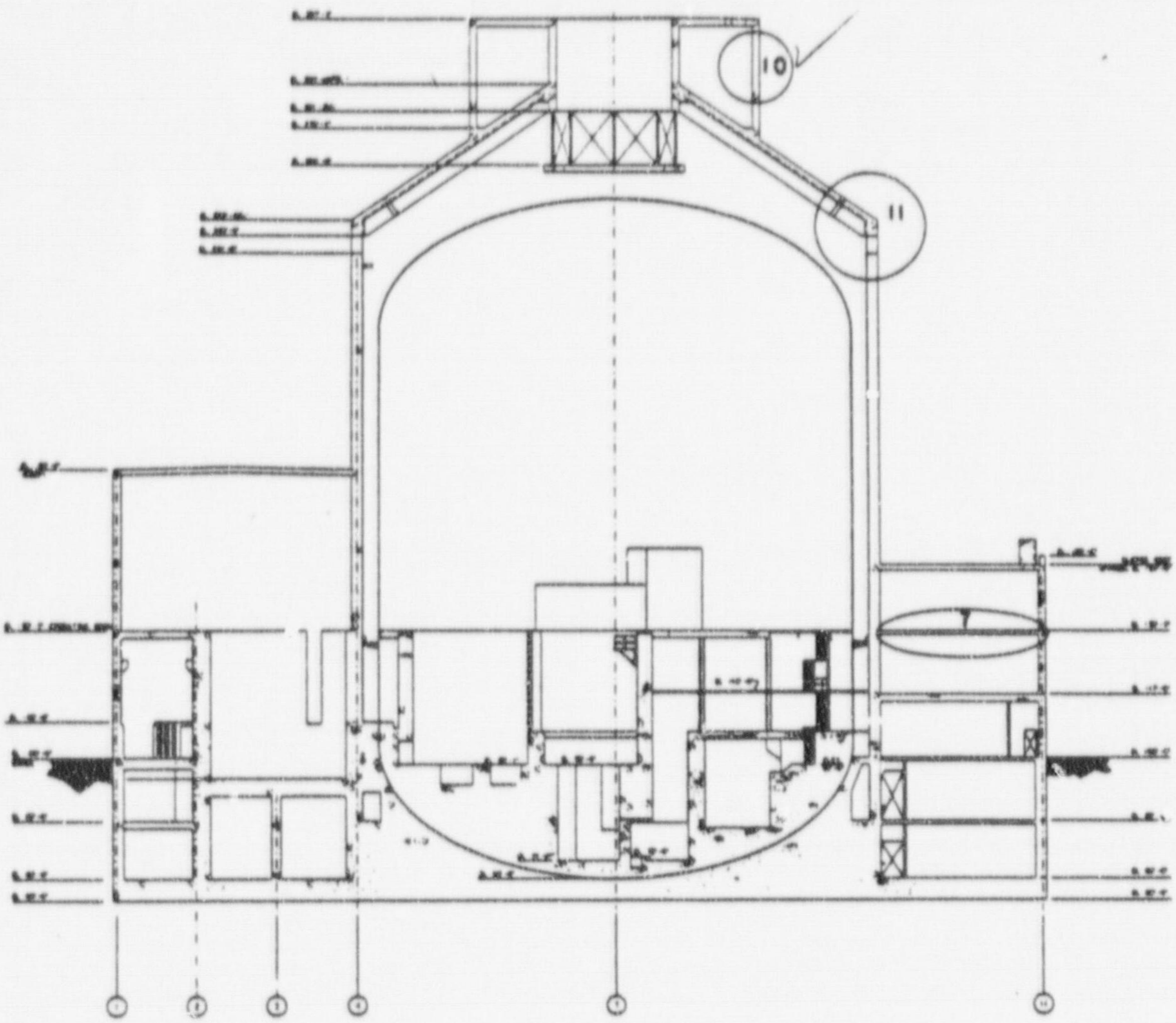


Figure 3H.5-1 (Sheet 3 of 3)

Nuclear Island Critical Sections
Section A-A

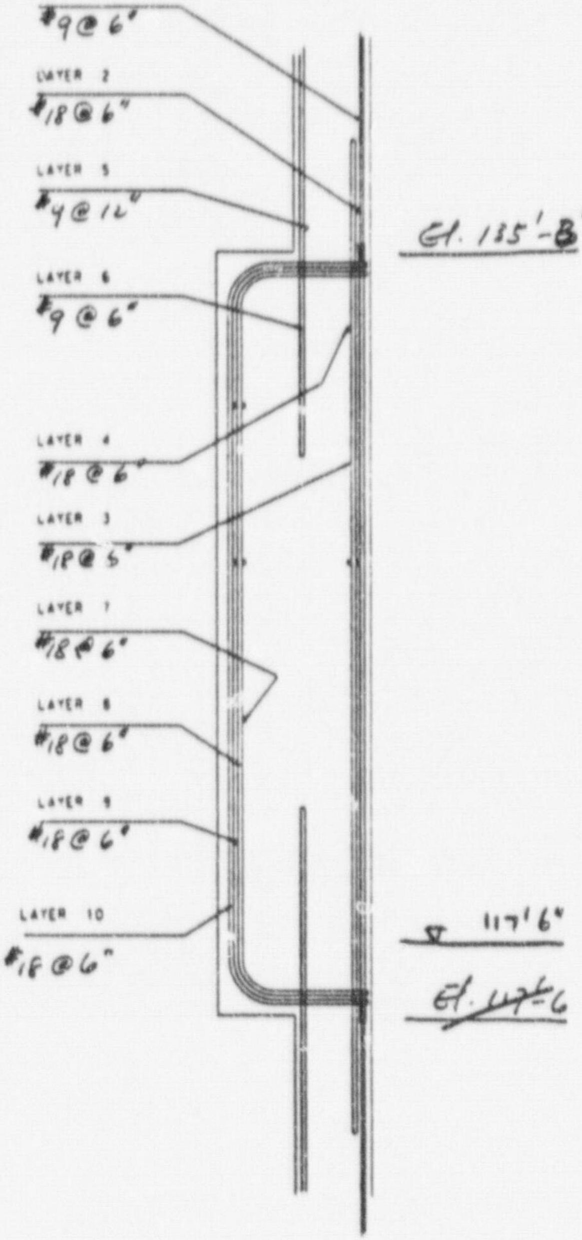


Figure 3H.5-5 (Sheet 2 of 3)

Concrete Reinforcement in Wall 11

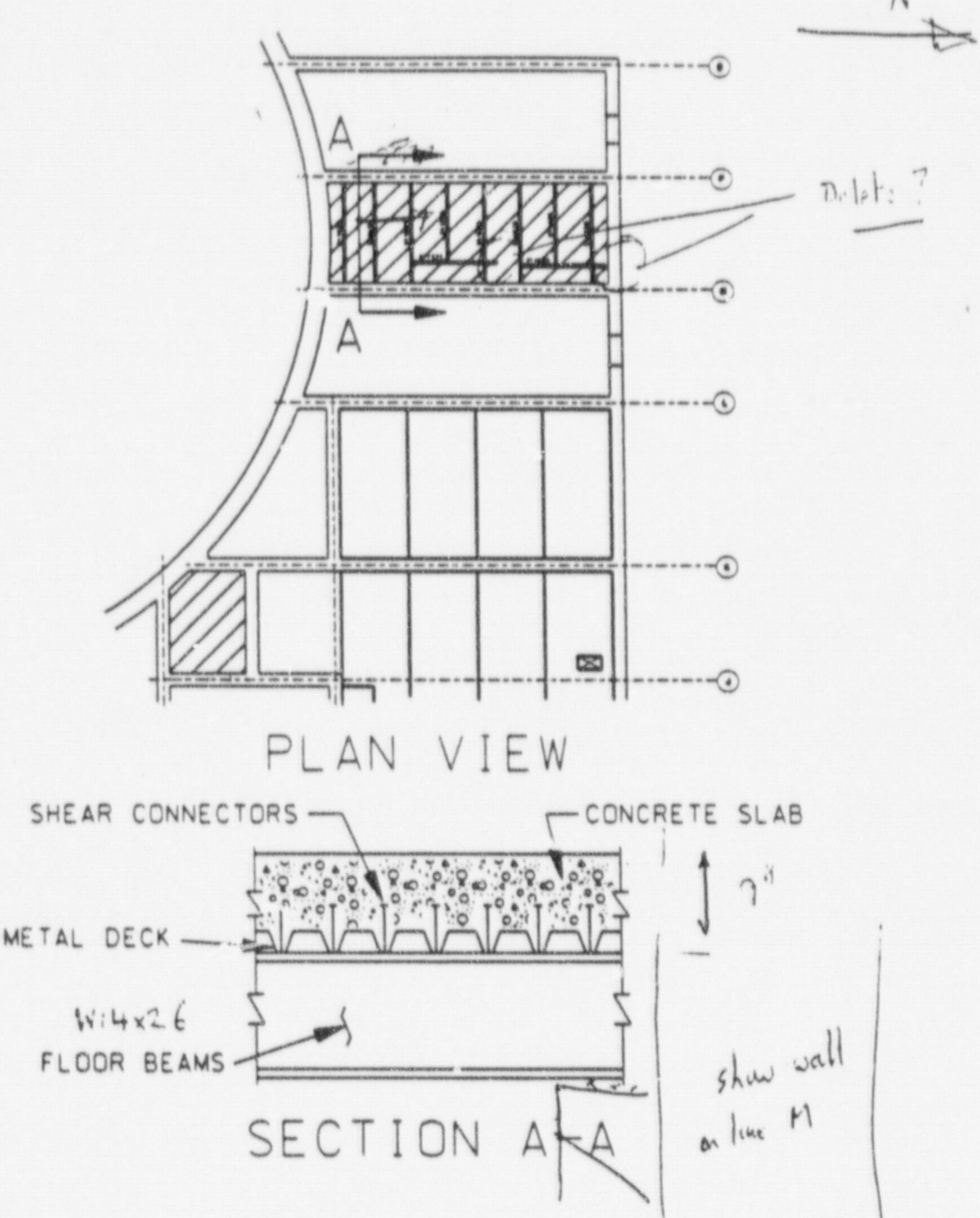
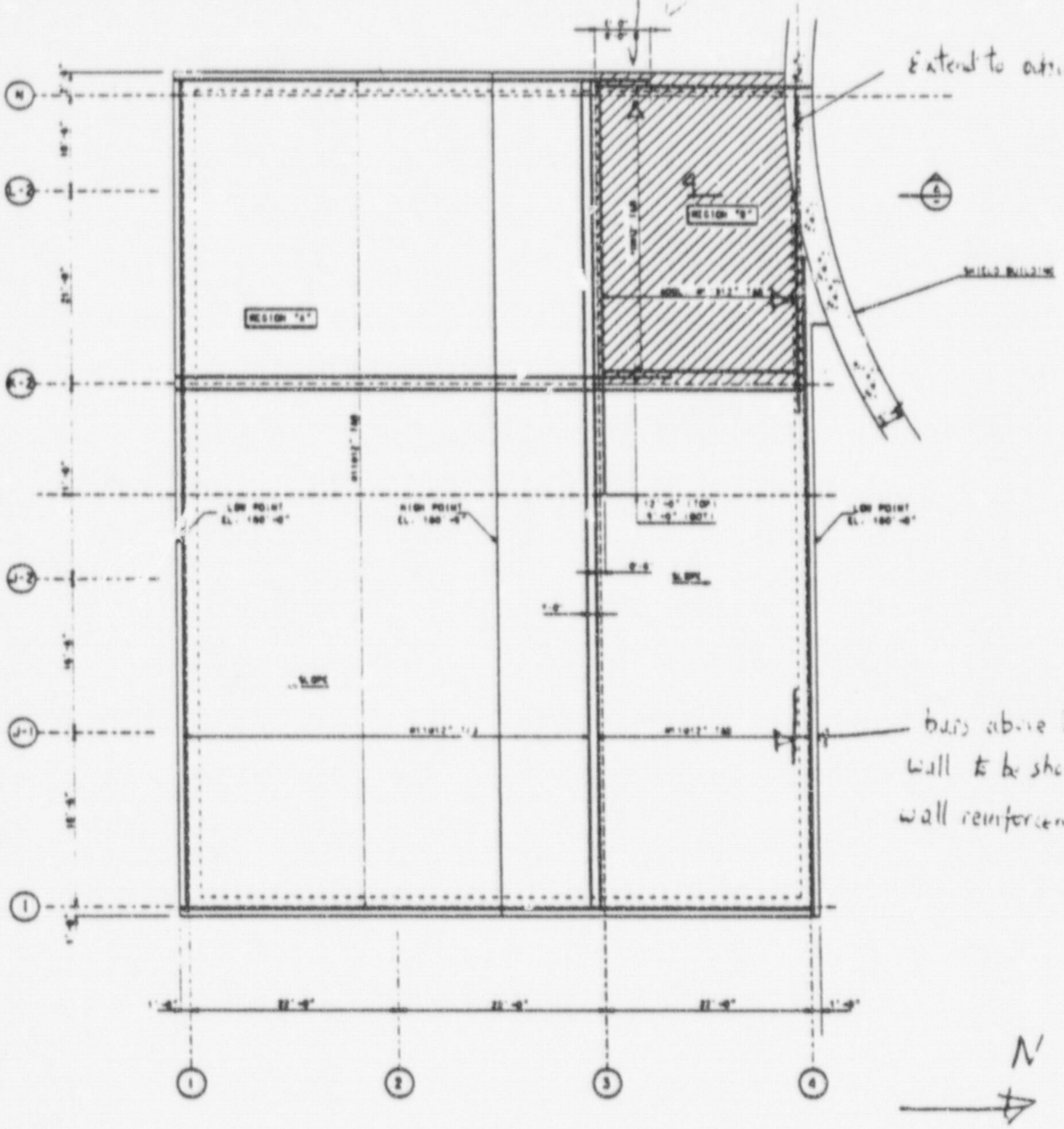


Figure 3H.5-6

Auxiliary Building
Typical Composite Floor

bars above thickness of wall to be shown on wall reinforcement drawing



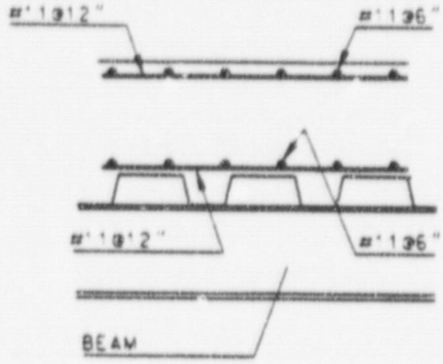
Extend to outside face of wal

bars above thickness of wall to be shown on wall reinforcement drawing

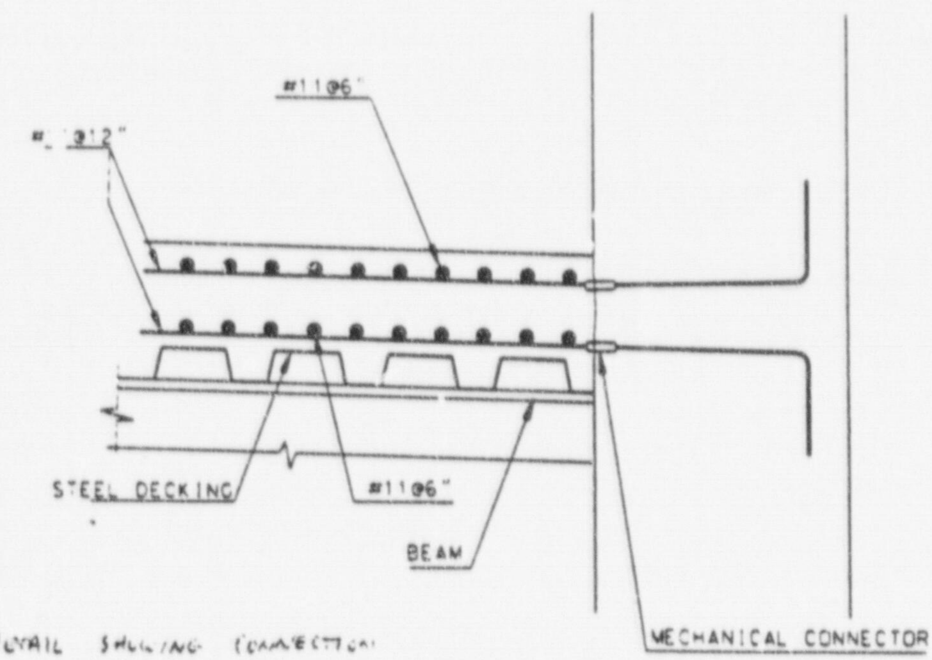
Add new sheet showing reinforcement in walls just below the roof.

Figure 3H.5-7 (Sheet 1 of 2)

Auxiliary Building Roof
Typical Reinforcement and Connection to Shield Building



REINFORCEMENT FLOOR DETAIL IN REGION "B"



ADD DETAIL SHOWING CONNECTION
OF STEEL GIRDER

Figure 3H.5-7 (Sheet 2 of 2)

Auxiliary Building Roof
Typical Reinforcement and Connection to Shield Building

Table 3H.5-7

Interior Wall on Column Line L
Details of Wall Reinforcement

Wall Segment	Location	Reinforcement on Each Face, sq.in./ft.	
		Required	Provided
Elevation 117'-6" to 135'-3"	Horizontal	3.54	3.72
	Vertical	4.74	5.12
Elevation 135'-3" to Roof	Horizontal	1.81	2.00
	Vertical	2.19	2.56

Shear Reinforcement:

Wall Segment	Location	Reinforcement, sq.in./ft.	
		Required	Provided
Elevation 117'-6" to 135'-3"	stet ~~In east-west direction~~ T-Headed stirrups	0.88	1.2