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Energy Systems

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DCP/NRC1154
NSD-NRC-97-5456
Docket No.: 52-003

December 1, 1997

Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555

ATTENTION: T. R. Quay

SUBJECT: SHIELD BUILDING ROOF DSER OPEN ITEM 3.8.4.4-2

Reference: Letter from NRC to Westinghouse, dated September 30, 1997, "Summary of AP600 Meeting to Review the Structural Design of Nuclear Island Structures."

Dear Mr. Quay:

The structural design of the shield building roof was reviewed by the staff during the week of August 11-15, 1997. The NRC summary of the meeting is provided in reference 1. DSER open item 3.8.4.4-2 (OITS# 750) was identified as Action W with the action being to provide additional information on the shear reinforcement of the air inlet columns, tension ring beam, and compression ring beam.

The response for this item is provided in the Attachment. The basis for the design has been described. The shear reinforcement has been changed to respond to the staff comments. The revised design is shown in proposed revisions to the SSAR included with the response. These revisions provide a description of the shield building roof critical sections. SSAR markups in the Attachment include a portion of SSAR Appendix 3H which will be added to show critical sections in the containment, shield building, and the auxiliary building. The inclusion of critical sections in Appendix 3H represents a change to the previous commitment for this information to be included in a separate design summary report. This change is in response to the expressed preference of the NRC staff for the critical section information to be included in the SSAR.

Attachment 8 of the Reference also identified a series of issues identified during the staff review which were not fully addressed by Westinghouse during the meeting. Responses to these issues are provided in the Attachment.

The status of OITS #750 will be changed to Confirm W pending transmittal of the identified information in Appendix 3H.

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Please contact Donald A. Lindgren at (412) 374-4856 if you have any questions.

Brian A. McIntyre, Manager
Advanced Plant Safety and Licensing

jml

Attachment

cc: J. M. Sebrosky, NRC (w/Attachment)
N. J. Liparulo (w/o Attachment)

DSER Open Item 3.8.4.4-2 (OITS # 750)

In a letter dated September 30, 1997 additional questions about this open item were identified. The staff comments and the Westinghouse response are provided in the following.

The staff's review of design calculations of the shield building roof structures identified the following findings:

d) Westinghouse used double-U bars for the hoop reinforcement (or stirrups) at air inlet columns, tension ring beam and compression ring beam to resist shear and torsion (#8 rebar at 6 inches on center for the column, #6 rebars at 4 inches on center for the tension ring beam, and #6 rebars at 12 inches on center for the compression ring beam). In the air inlet columns, Westinghouse did not extend the shear hoop reinforcement (stirrups) and cross-ties above and below the air inlet openings. In the SSAR Westinghouse commits to ACI-318 Chapter 21, which states that stirrups should be provided with 135 degree hooks at both ends of the rebars. The use of double-U bars for the shear reinforcement by Westinghouse does not meet this commitment with their current design."

Response:

Position 3 of Regulatory Guide 1.142 states ACI 349-76 lacks specific requirements to ensure the ductility of concrete moment frames and that adherence to the requirements of Appendix A to ANSI / ACI 318-77 is acceptable. ACI 318 has been revised a number of times since the regulatory guide was issued and Appendix A is now incorporated in ACI 318 as Chapter 21. The Westinghouse position on Regulatory Guide 1.142 in SSAR Appendix 1A states that ACI 318-89 Chapter 21 is used in the design of moment resisting frames. This will be updated to the 1995 edition to be consistent with SSAR subsection 3.8.4.4.1 which states that the special requirements of Chapter 21 of ACI 318 are considered in detailing the reinforcement.

The intent of the commitment in the SSAR is to comply with the guidance in Regulatory Guide 1.142 for design of moment resisting frames. Appendix A of ACI 318-77 only included requirements for moment resisting frames. Subsequent revisions of ACI 318 have included additional requirements and Westinghouse agreed in the response to DSER open item 3.8.4.2-3 to consider the ductility criteria of ACI 318-95 in the design of the AP600 reinforced concrete structures. The review during the August, 1997 meeting identified disagreement between the NRC staff and Westinghouse in the implementation of these considerations. The following sections of this response describe:

- ACI 318-95, Chapter 21 - Special Provisions for Seismic Design
- AP600 application of ACI 318-95
- Design of shield building roof critical sections

Subsection 3.8.4.4.1 will be revised to clarify how Chapter 21 is considered in the design of critical sections of the AP600.

NRC staff also identified concerns on the Westinghouse approach to the special requirements of Chapter 21 in their review of the basemat. The concerns related to the basemat are addressed in Westinghouse letter NSD-NRC-97-388, dated October 17, 1997.

1.0 ACI 318-95, Chapter 21 "Special Provisions for Seismic Design"

"Building Code Requirements for Structural Concrete" (ACI 318-95) covers the design and construction of structural concrete and is written in a form that it may be adopted by reference in a general building code. Chapter 21 is provided for those jurisdictions that require seismic design.

Chapter 21 of ACI 318-95 contains special requirements for design and construction of reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response.

Seismic design of commercial structures is typified by the provisions of the Uniform Building Code. The latest edition published in 1997 is based on recommendations by the Structural Engineers Association of California (SEAOC) for the seismic loads. It is patterned after, and in general conformity with, ACI 318-95 for the design of concrete structures. The seismic loads in UBC-97 are those that would be calculated in an elastic analysis of the structure reduced by a reduction factor (R_w) which takes credit for the non-linear behavior of the structure. For a reinforced concrete frame structure the reduction factor is 5.6 or 8.5 depending on the detailing; for a shear wall structure it is 4.5. These reduced seismic loads are combined with the dead and live loads using a load factor of 1.0 for seismic loads. Thus, the factored design loads used as input to the structural evaluation of Chapter 21 are lower by a factor of about 5 than the loads that would be calculated in an elastic analysis. Significant non-linear response cycles will occur.

The commentary to chapter 21 discusses the level of toughness required for a structure. For a given design earthquake intensity, it states that it would be plausible to soften or relinquish some of the detail requirements if the design strength is increased with respect to the minimum code requirements. It identifies the two practical choices as (a) a system with sufficient strength to respond to the ground motion within the linear or nearly linear range of response and (b) a system with adequate details to permit nonlinear response without critical loss of strength. Chapter 21 provides requirements in relation to the second option. ACI 349 is close to the first option since it provides sufficient strength to respond to the ground motion within the linear or nearly linear range of response requirements.

2.0 AP600 application of ACI 318-95

The AP600 is designed to ACI 349 using linear elastic analyses for the safe shutdown earthquake. Since the seismic design does not rely on energy dissipation in the non-linear range of response, the provisions of ACI 318 are not directly applicable. However, ductility is desirable and some non-linear response is considered in the seismic margins analysis.

Chapter 21 is intended for structures for which the design forces are determined on the basis of energy dissipation in the nonlinear range of response. For such structures the elastic demand may be a factor of five greater than the nominal strength. For the AP600, Chapter 21 is not directly required by the code since the structure is designed to give basically elastic response under the SSE loads. The demand under

the review level seismic margin earthquake is 1.67 times the SSE. Details in accordance with the other portions of the code will provide adequate ductility for this 67 percent increase in demand. However, consideration is given to the detailing requirements of Chapter 21 as additional margin to assure ductile behavior. The special reinforcing details of Chapter 21 are applied to critical structural elements providing major seismic load resistance. The provisions are applied to elements that experience reinforcement tensile stresses above yield or concrete compressive stresses above the concrete strength when the safe shutdown earthquake loads are increased by a factor of 1.67.

The special provisions of Chapter 21 are primarily applicable to structures where lateral loads are taken by beam and column moment resisting frames. There are a few provisions for shear walls, particularly for high walls with large compressive forces due to overturning. The AP600 concrete structures are primarily shear wall structures with shear walls in each direction. Each shear wall frames into other shear walls. The shield building cylinder and roof are shell structures with lateral resistance provided by in-plane shear behavior. Floor and roof slabs act as structural diaphragms which transmit inertial forces to the shear walls.

The reinforcement details of Chapter 21 are considered for the following structural elements that provide the major seismic lateral load resistance:

- out-of-plane behavior of nuclear island basemat (see Westinghouse letter NSD-NRC-97-388, dated October 17, 1997)
- in-plane behavior of cylindrical wall of shield building including the columns between the air inlets (see section 3 below)
- in-plane behavior of interior and exterior walls of the nuclear island (similar to in-plane evaluation for shield building columns described in subsection 3.2.1 below)

3.0 Design of shield building roof critical sections

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. Two critical areas are discussed below, namely the tension ring at the connection of the conical roof to the cylindrical wall and the columns between the air inlets just below the air inlets. For each area the critical design loads are identified and the consideration for ductility is described. Minor changes have been made to the reinforcement subsequent to the NRC staff review in August. The revised configuration of reinforcement is shown in Figure 3H.5.6-1 (attached). Appendix 3H is being added to the SSAR to document the design of critical sections. Information on the shield building roof critical sections will be included in subsection 3H.5.6.

3.1 Tension ring

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 tension ring has been reviewed against paragraph 21.3 of ACI 318-95. This is not directly applicable since it applies to frame members proportioned primarily to resist flexure whereas the ring is proportioned primarily to resist axial tension. The reinforcement details shown in sheet 2 of Figure 3H.5.6-1 have been selected as follows:

- Hoop reinforcement meets the requirements of paragraph 21.3.3.6

- Ties have either seismic hooks with a bend not less than 135 degrees or T heads at each end. Use of T heads was described in the Westinghouse letter on basemat issues (NSD-NRC-97-388, dated October 17, 1997).

3.2 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.

3.2.1 In-plane behavior

Paragraph 21.6.2.3 requires special transverse reinforcement when the compressive stress exceeds 0.2 f_c'. This requirement applies for structures for which the design forces are determined on the basis of energy dissipation in the nonlinear range of response. For such structures the elastic demand may be a factor of five greater than the nominal strength. Thus, the requirement for special transverse reinforcement is intended for those cases where the elastic demand exceeds f_c'. The maximum compressive stress in the column of 1400 psi is 0.35 f_c'. This magnitude, even when the seismic contribution is increased by 67 percent, is well below the strength of the concrete and hence does not require special transverse reinforcement.

The lap splices in the double U horizontal reinforcement (#8 bars at 6 inches on center) have been eliminated. The horizontal reinforcement in the shear walls is now provided by T headed bars as shown in sheet 3 of Figure 3H.5.6-1. These T heads act as mechanical anchorage as recommended in the commentary to paragraph 21.6.6.

3.2.2 Out-of-plane behavior

The column is 36" thick and 60" high. The primary loading is vertical load plus out of plane shear. The out-of-plane shear is smaller than the in-plane shear since the horizontal seismic loads are resisted primarily by in-plane shear in the cylindrical wall. The out-of-plane loads are caused by the deformation of the tension ring and do not contribute significantly to the lateral load resistance. The column was evaluated against the provisions of paragraph 21.6. Out-of-plane shear is resisted by through wall T headed ties as shown in sheet 3 of Figure 3H.5.6-1.

The area of out-of-plane shear reinforcement meets the requirements of 21.4.4.1. The through wall transverse reinforcement is extended 30 inches into the shield building cylinder below. The through wall ties provided on the tension ring above the air inlet are extended above the column. This meets paragraph 21.4.4.4 (note that the extension above and below the openings was already shown on one of the drawings at the time of the meeting and the staff comment was incorrect).

Paragraph 21.6 does not define spacing limits applicable to these ties. The 6 inch spacing vertically meets the spacing limits given for flexural members in 21.3.3.2. It meets 21.4.4.2 (a) but exceeds the 4 inch limit of 21.4.4.2 (b). The spacing is supported by tests on beams with a span to depth ratio of 5 using T-headed stirrups (Reference 750-1). This reference recommends that the stirrup spacing be limited to $d/(2 \tan \theta)$, where θ is the inclination of the shear plane, typically in the range of 30 to 45 degrees from the member axis. The 0.9 degree horizontal spacing meets the requirements of 21.3.3.3 and 21.4.4.3.

Ties at the end of each column adjacent to the air inlets are at the same elevations as the T headed horizontal bars for in-plane shear and result in a hoop configuration capable of resisting the small torsion on the column.

Reference 750-1: L. A. Zheng, "Shear tests to investigate stirrup spacing limits", Master's thesis, University of Toronto, 1989.

SSAR Revisions:

Appendix 1A of the SSAR:

In Regulatory Guide 1.142, position C.3, Appendix A, revise fourth column to read:

> "Concrete moment resist~~ant~~ing frames are designed to ACI 318-~~89~~95."

Revise first paragraph of SSAR 3.8.4.4.1 as follows:

> The design and analysis procedures for the seismic Category I structures (other than the containment vessel and containment internal structures), including assumptions on boundary conditions and expected behavior under loads, are in accordance with ACI-349 for concrete structures, with AISC-N690 for steel structures, and AISI for cold formed steel structures. The structural modules in the auxiliary building are designed using the same procedures as the structural modules in the containment internal structures described in subsection 3.8.3.

The ductility criteria of ACI-318, Chapters 12 and 21, are considered in detailing, placing, anchoring, and splicing of the reinforcing steel. Chapter 21 of ACI 318 contains special requirements for design and construction for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the non-linear range of response. The special requirements are intended to provide a structural system with adequate details to accommodate non-linear response and displacement reversals without critical loss of strength. The nuclear island structures are designed for the safe shutdown earthquake without credit for energy dissipation. To provide additional margin, the special reinforcing details of Chapter 21 are applied to critical structural elements providing major seismic load resistance. The provisions are applied to elements that experience reinforcement tensile stresses above yield or concrete compressive stresses above the concrete strength when the safe shutdown earthquake loads are increased by a factor of 1.67 (ratio of the seismic margin review level earthquake to the safe shutdown earthquake). The structural elements include:

- in-plane behavior of interior and exterior walls of the nuclear island
- in-plane behavior of cylindrical wall of shield building including the columns between the air inlets
- out-of-plane behavior of nuclear island basemat

The bases of design for the tornado, pipe breaks, and seismic effects are discussed in Sections 3.3, 3.6, and 3.7, respectively. The foundation design is described in subsection 3.8.5.

Add Appendix 3H Critical Sections

Appendix 3H is being added to document the design of the critical sections. The following material will be included for the shield building roof.

3H.5.6 Connection of shield building roof to cylindrical wall

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. Two critical areas are discussed below, namely the tension ring at the connection of the conical roof to the cylindrical wall and the columns between the air inlets just below the air inlets. The reinforcement in these areas is shown in the Figure 3H.5.6-1 and is summarized in Table 3H.5.6-1.

3H.5.6.1 Tension ring

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.

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.

Add table 3H5.6-1 as follows:

Table 3H.5.6-1

Shield building roof reinforcement summaries

Tension ring

Member force	Reinforcement required sq.in/in. length	Reinforcement provided	Reinforcement provided sq.in/in. length	Ratio required / provided
Axial + bending		38 # 14 bars		0.77(1)
Torsion	0.055	#9 hoop @ 0.45 °	0.15	0.37
Torsion + vertical shear	2 x 0.055 + 0.19 = 0.30	2 legs # 9 hoop @ 0.45° 2 # 8 ties @ 0.9 °	0.42	0.71
Torsion + horizontal shear	2 x 0.055 + 0.081 = 0.19	2 legs # 9 hoop stirrup @ 0.45° 3 # 5 ties @ 1.8 °	0.33	0.58

Air inlet column

Member force	Reinforcement required sq.in/in. height	Reinforcement provided	Reinforcement provided sq.in/in. height	Ratio required / provided
Axial + bending		48 # 11 bars		0.3 (1,2)
Torsion	0.015	#5 hoop at 6"	0.05	0.30
Torsion + in-plane shear	2 x 0.015 + 0.20 = 0.23	3 # 7 ties @ 6"	0.30	0.77
Torsion + out-of-plane shear	0.37	# 5 hoop @ 6" 9 # 5 ties @ 6"	0.56	0.66

Notes

1) This ratio is calculated from the interaction diagram for axial load and moments for the section and does not include the effect of torsion loading. It is the ratio of the loads on the interaction surface divided by the design loads for the same ratio of axial loads and moments.

2) The vertical reinforcement in the column is provided to meet minimum vertical reinforcement requirements for shear walls.

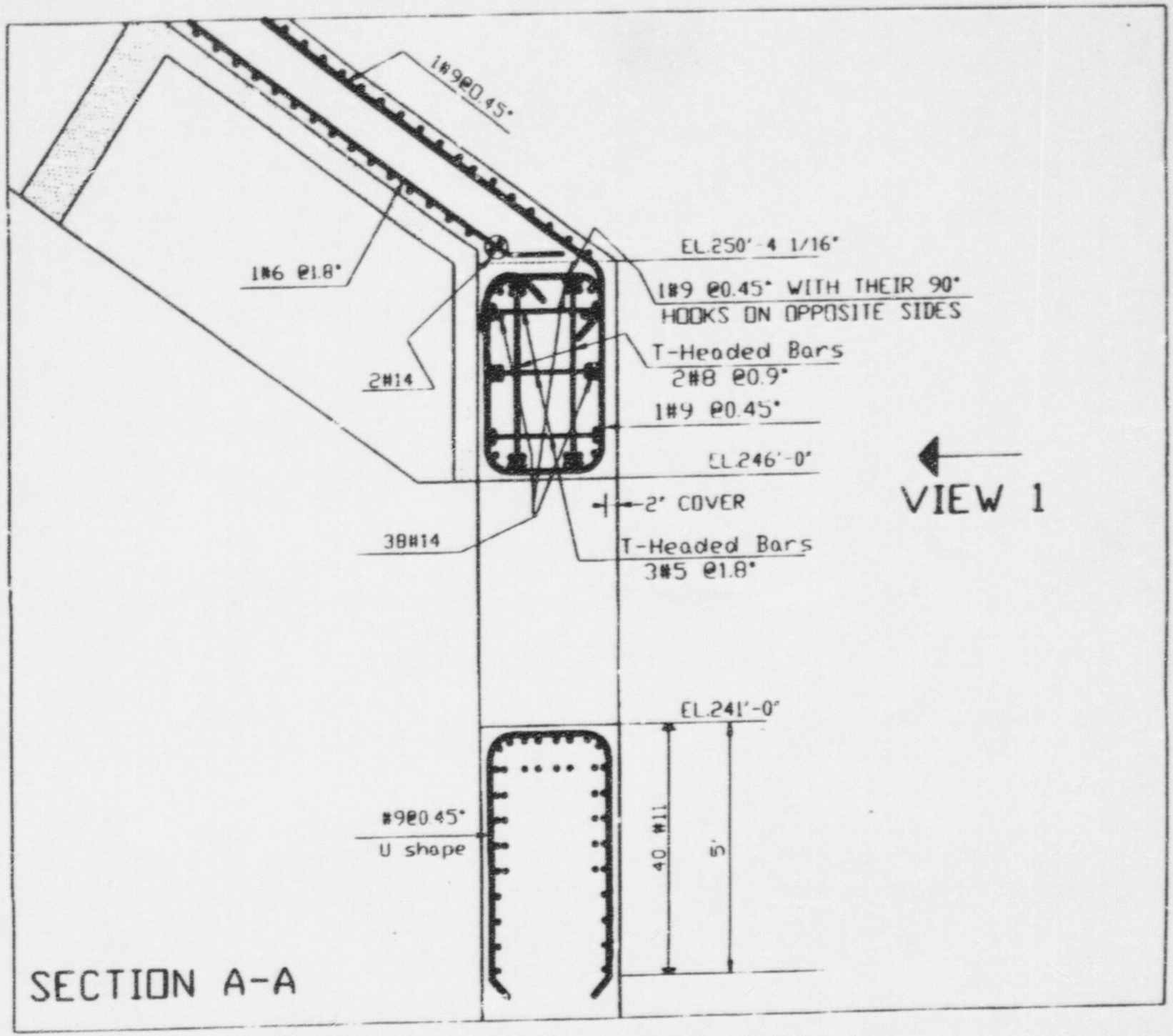

Figure 3H.5.6-1(sheet 1 of 5)

Shield building roof
Reinforcement at roof to wall connection

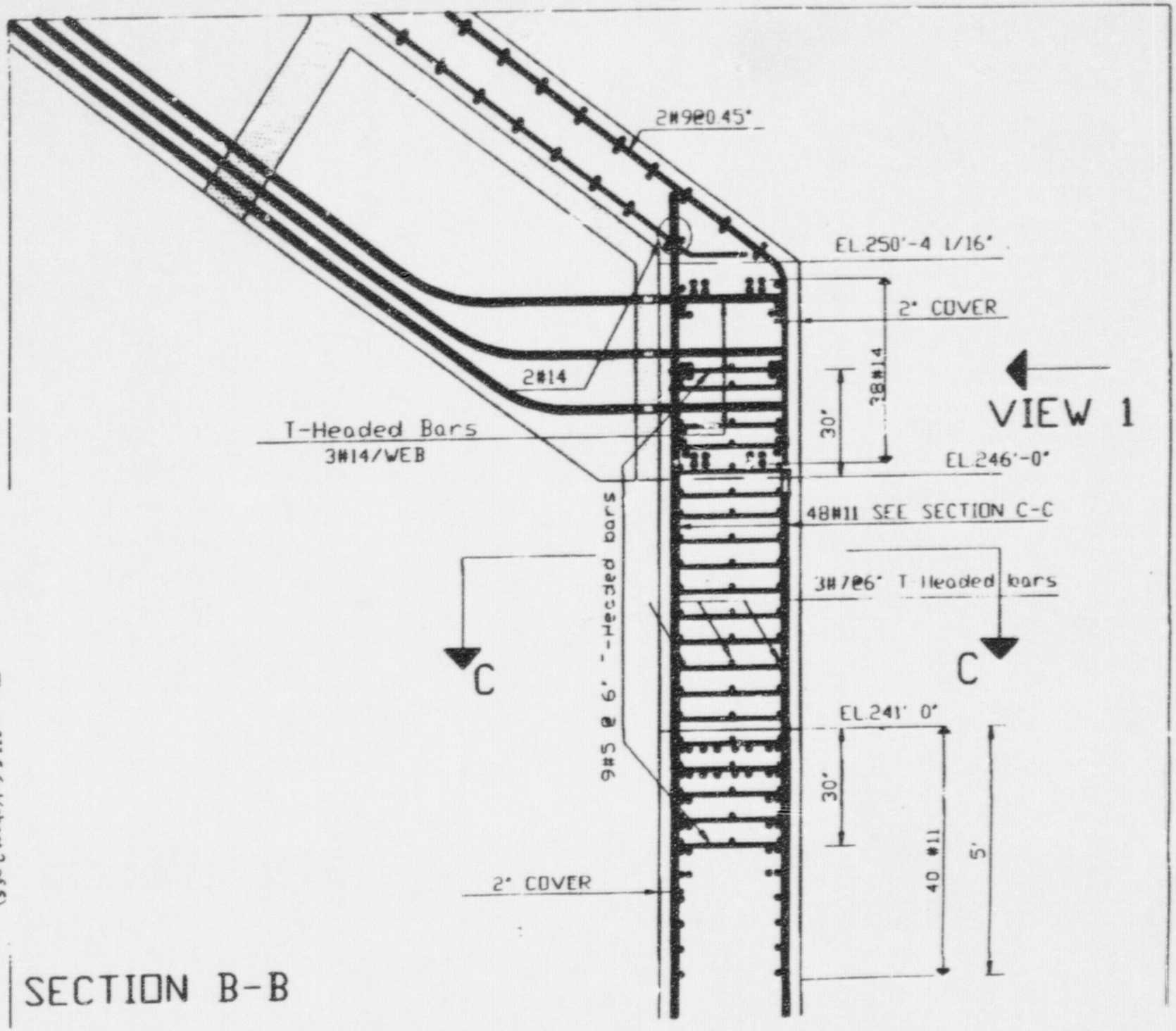

Figure 3H.5.6-1(sheet 2 of 5)

Shield building roof
Reinforcement at roof to wall connection

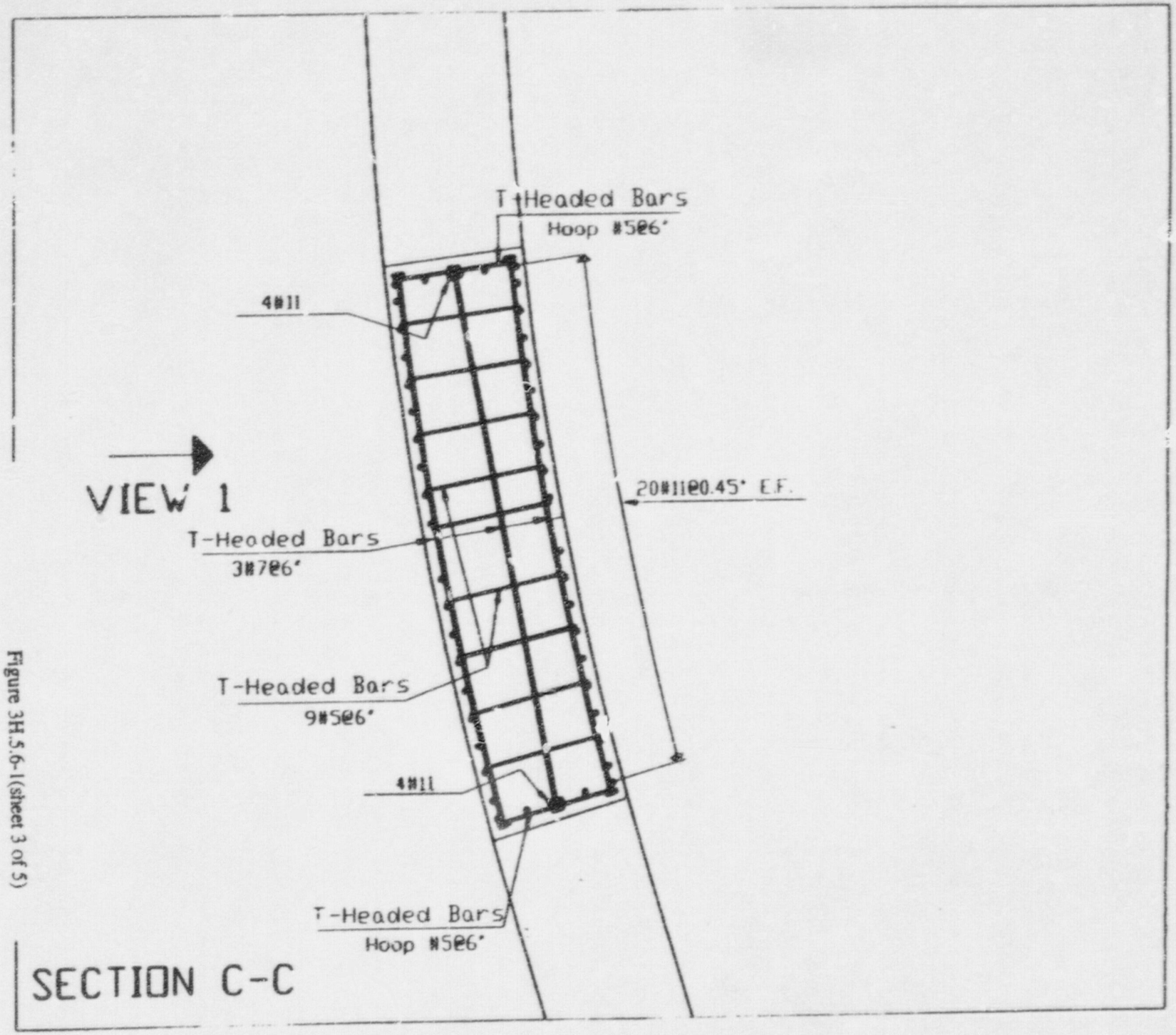

Figure 3H.5.6-1(sheet 3 of 5)

Shield building roof
Reinforcement at roof to wall connection

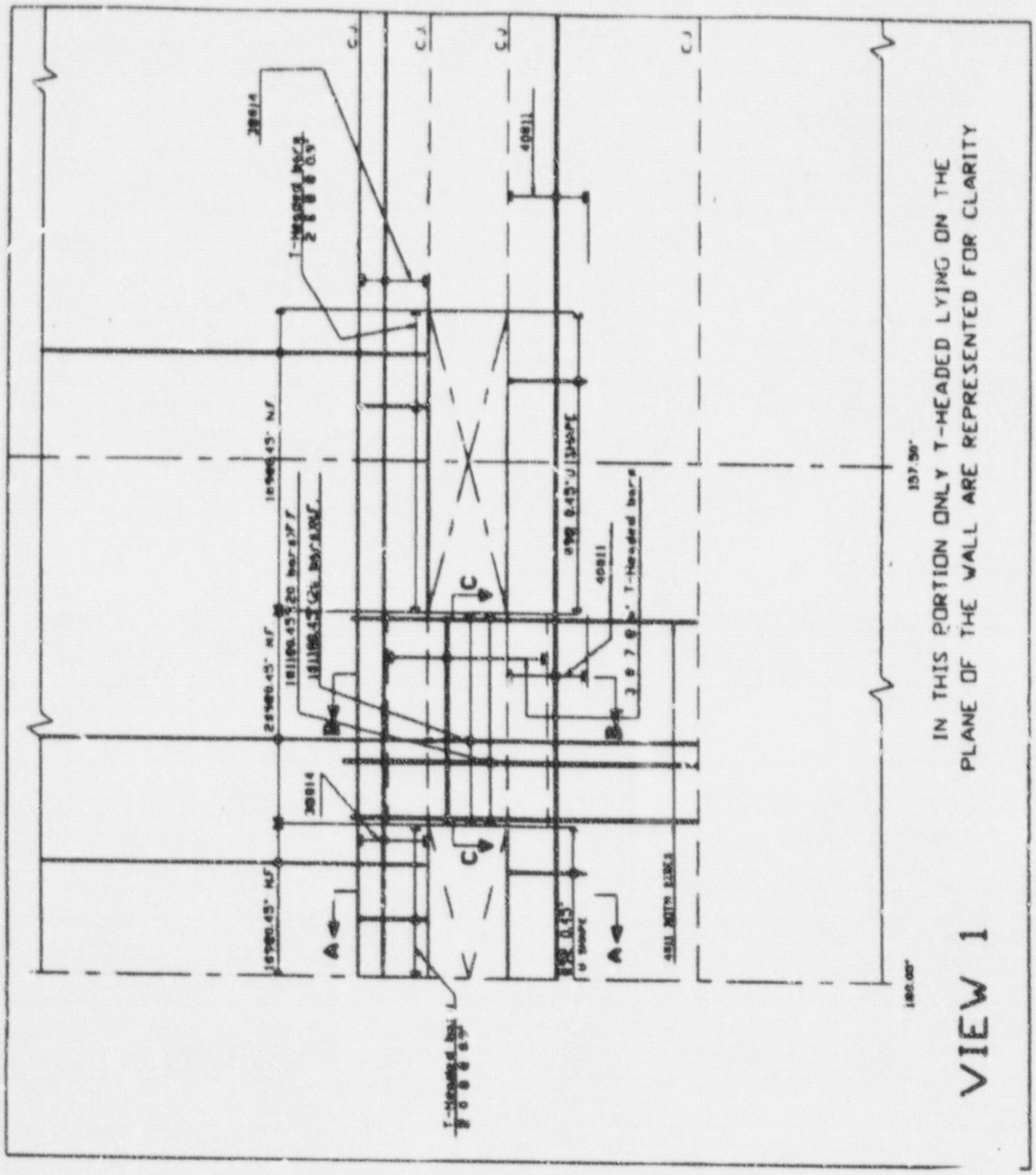

Figure 3H.5.6-1(sheet 4 of 5)

Shield building roof
Reinforcement at roof to wall connection

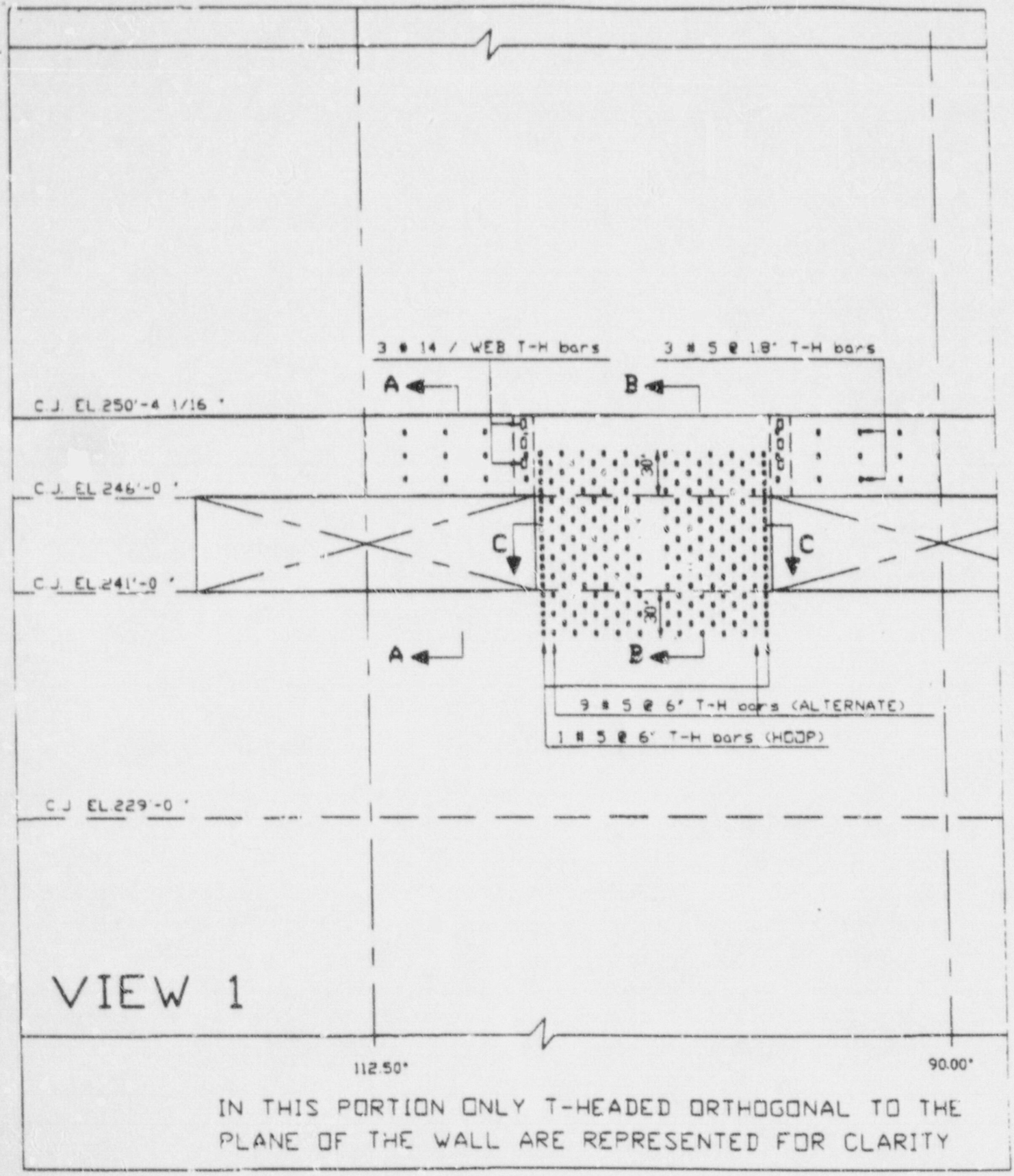

Figure 3H.5.6-1(sheet 5 of 5)

Shield building roof
Reinforcement at roof to wall connection

Meeting Open Items

The NRC staff comments *(shown in italics)* are taken from the letter dated September 30, 1997 transmitting notes from the structural meetings between August 11-15, 1997

NRC Staff comment

5.(b) *In the design of the PCWS tank roof, Westinghouse assumed the roof slab is rigid and did not consider the amplification due to the out-of-plane flexibility. The staff's review of Calculation No. 1070-S3R-010 found that the frequency of the roof slab in the out-of-plane direction is in the 14 Hz range and a 30 to 40 percent amplification of seismic load should be included in the design.*

Westinghouse response

Calculation No. 1070-S3R-010 was reviewed in detail following the meeting. It shows a 14 hertz mode and describes it as "local roof vert." Further investigation identified that this mode corresponded to a very local mode of the shield building roof and not of the PCS tank roof. This mode has negligible participation in the shield building roof response.

Additional analyses were performed to determine the frequency of the PCS tank roof. The fundamental out-of-plane frequency of the PCS tank roof, including the fire protection tank, is 24.5 Hertz. Review of the vertical floor response spectra at this elevation (SSAR Figure 3.7.2.15, sheet 3) shows that the amplification is negligible at this frequency.

This issue was incorrectly identified during the meeting and further investigation has demonstrated that the original design methodology was appropriate.

Note that the fire protection tank has subsequently been eliminated. Analyses of the revised configuration without the fire protection tank show similar results for the fundamental out-of-plane frequency.

NRC Staff comment

6 *The staff's review of design calculations of the shield building roof structures identified the following findings:*

For checking the design adequacy, Westinghouse should provide comparisons of required reinforcement versus provided reinforcement for the tension ring and column. This action is for documentation purpose.

Westinghouse response

Comparisons of the required reinforcement versus the provided reinforcement for the tension ring and column are included in Table 3H.5.6 (see draft table above)

NRC Staff comment

6(b) The reinforcement drawings do not show the vertical reinforcement in sides of the air inlet columns. The length of these bars should also be provided.

Westinghouse response

This vertical reinforcement is the reinforcement at the end of the columns adjacent to the air inlet openings. It was shown on the reinforcement drawings. The drawings have been revised to clarify this reinforcement and to show the length. The length of the bars in the middle of the wall is the height of the opening plus development length above and below the opening.

NRC Staff comment

6(c) Westinghouse should provide additional explanation on Table 11.6 of Appendix 11 to Calculation No. 1277-S3C-006 for the calculation of the T-sections. Explain how the strains are calculated and provide the definition of the safety ratio. Also describe how the shear peak value becomes the shear flow described in Page 18 of Appendix 11. In addition, provide the definition of the "shear triangle area."

Westinghouse response

Additional explanation has been provided in the calculation for Table 11.6. Strains are calculated using the T section properties. The safety ratio is the ratio of the actual loads to the loads corresponding to maximum permitted steel or concrete strain. The "shear flow" and "shear triangle area" are used to determine the average shear in a portion of the web.

6(d) Westinghouse used double-U bars for the hoop reinforcement (or stirrups) at air inlet columns, tension ring beam and compression ring beam to resist shear and torsion (#8 rebar at 6 inches on center for the column, #6 rebars at 4 inches on center for the tension ring beam, and #6 rebars at 12 inches on center for the compression ring beam). In the air inlet columns, Westinghouse did not extend the shear hoop reinforcement (stirrups) and cross-ties above and below the air inlet openings. In the SSAR Westinghouse commits to ACI-318 Chapter 21, which states that stirrups should be provided with 135 degree hooks at both ends of the rebars. The use of double-U bars for the shear reinforcement by Westinghouse does not meet this commitment with their current design.

Westinghouse response

The response above for staff comments in the September 30, 1997 letter on DSER Open Item 3.8.4.4-2 addresses the design of the tension ring beam and the air inlet columns. The compression ring beam is only a ring beam during early stages of construction. In the final configuration it is a thickened portion of the roof slab. The shear stirrups are provided for construction loads only. Hence, it is not necessary to consider the special details of Chapter 21 for the compression ring.

NRC Staff comment

6(e) Some inconsistencies between the summary table in Appendix 25 to Calculation No. 1277-S3C-006 and the design drawing were identified. Westinghouse should provide explanation for the concerns discussed below:

The conical shell roof at the tension ring beam shows that an amount of 2.14 square inches per foot bottom radial reinforcement is needed at the columns and an amount of 1.78 square inches bottom radial reinforcement over the air inlet. However, Table 11.6 of Appendix 11 to Calculation No. 1277-S3C-006 shows that one # 9 rebar is provided above the air inlet and none at the column. Also, the drawing shows the bottom reinforcement discontinued at the end of conical roof.

Westinghouse response

The bottom radial reinforcement is provided in the web of the precast panel. The summary table has been revised.

NRC Staff comment

The conical roof at the compression ring shows that an amount of 2.04 square inches bottom radial reinforcement is needed. The same table shows no reinforcement provided. The drawing also shows the bottom reinforcement stopped at the compression ring.

Westinghouse response

Reinforcement for the T section is provided in the web of the precast panel. Bottom radial reinforcement is not required in the compression ring. A supplemental calculation has been performed to show that the ties between the precast panel and the compression ring are adequate to transfer the loads across this interface.

NRC Staff comment

The conical shell roof at the internal PCCWS tank wall shows that nine #9 rebar were provided for the hoop reinforcement at the top and bottom faces. However, the drawing shows that 18 #9 rebars are provided but they are not properly distributed at the top and bottom faces.

Westinghouse response

The drawing has been revised showing the reinforcement equally distributed between the two faces.