DISTRIBUTION SHEET

To G. A. Glover	From Structural Assessments	Page 1 of 1 Date 9/30/94
Project Title/Work Order 232Z Building Seismic Assessment		EDT No. 608824 609356 ECN No. N/A

Name	MSIN	Text With All Attach.	Text Only	Attach./ Appendix Only	EDT/ECN Only
G. A. Glover	T4-20	X			X
R. J. Thomas	T4-20				
T. J. Conrads	H5-55	X			X
L. W. Bartholf	H5-53				X
L. E. Ebbeson	T5-04				X
C. M. Kronvall	T5-53				X
R. D. Redekopp	T5-30				X
L. H. Rodgers	H5-53				X
R. A. Russell	T4-20				X
R. M. Suyama	H5-60				X
G. R. Wagenblast	H5-55	X			
Central Files (2)	L8-04	X			
OSTI (2)	L8-07	X			

OCT 27 1994

ENGINEERING DATA TRANSMITTAL

10-27-94

Original

35 Station 21

20

Page 1 of 1

1. EDT

609356

2. To: (Receiving Organization) G. A. Glover PFP Engineering		3. From: (Originating Organization) Structural Assessments		4. Related EDT No.: N/A	
5. Proj./Prog./Dept./Div.: PFP Start-up 232Z		6. Cog. Engr.: G. R. Wagenblast		7. Purchase Order No.: N/A	
8. Originator Remarks: Release calculations for 232Z Building Seismic Assessment				9. Equip./Component No.: N/A	
				10. System/Bldg./Facility: N/A	
11. Receiver Remarks:				12. Major Assm. Dwg. No.: N/A	
				13. Permit/Permit Application No.: N/A	
				14. Required Response Date: 10/19/94	

15. DATA TRANSMITTED					(F)	(G)	(H)	(I)
(A) Item No.	(B) Document/Drawing No.	(C) Sheet No.	(D) Rev. No.	(E) Title or Description of Data Transmitted	Impact Level	Reason for Trans- mittal	Orig- inator Dispo- sition	Receiv- er Dispo- sition
1	WHC-SD-WM-DCR-002		0	232Z Building Seismic Assessment	SQ	1	1	2

16. KEY					
Impact Level (F)		Reason for Transmittal (G)		Disposition (H) & (I)	
1, 2, 3, or 4 (see MRP 5.43)		1. Approval 2. Release 3. Information 4. Review 5. Post-Review 6. Dist. (Receipt Acknow. Required)		1. Approved 2. Approved w/comment 3. Disapproved w/comment 4. Reviewed no/comment 5. Reviewed w/comment 6. Receipt acknowledged	

17. SIGNATURE/DISTRIBUTION (See Impact Level for required signatures)										(G)	(H)
Reason	Disp.	(J) Name	(K) Signature	(L) Date	(M) MSIN	(J) Name	(K) Signature	(L) Date	(M) MSIN	Reason	Disp.
	1	Cog. Eng. G. R. Wagenblast	<i>[Signature]</i>	10/20/94	H5-55						
	1	Cog. Mgr. T. J. Conrads	<i>[Signature]</i>	10/20/94	H5-60						
	1	QA A. K. Sharma	<i>[Signature]</i>	10/20/94	H5-60						
	2	Safety S. E. Nunn	<i>[Signature]</i>	10/20/94	EGH-38						
		Env.									
	1	L. E. Ebbeson	<i>[Signature]</i>	10/25/94	T5-04						
	1	G. A. Glover	<i>[Signature]</i>	10/25/94	T6-20						

18. G. R. Wagenblast <i>[Signature]</i> 10/25/94 Signature of EDT Originator		19. G. A. Glover <i>[Signature]</i> 10/25/94 Authorized Representative for Receiving Organization		20. L. P. Shroeder <i>[Signature]</i> 10/25/94 T. J. Conrads Cognizant/Project Engineer's Manager		21. DOE APPROVAL (if required) Ltr. No. N/A ☐ Approved ☐ Approved w/comments ☐ Disapproved w/comments	
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BD-7400-172-2 (07/91) GEF097

A-6000-135 (01/93) WEF067

RELEASE AUTHORIZATION

Document Number: WHC-SD-WM-DCR-002, REV.0

Document Title: 232Z Building Seismic Assessment

Release Date: October 27, 1994

**This document was reviewed following the
procedures described in WHC-CM-3-4 and is:**

APPROVED FOR PUBLIC RELEASE

WHC Information Release Administration Specialist:

Angela Burchard For -
M.N. Boston

10/27/94

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<p><i>A. Laughery For M. Boston</i> 10-27-94 APPROVED FOR PUBLIC RELEASE</p>		
7. Abstract <p>The 232Z Building in the 200 West Area has no documented evidence of meeting the structural design requirements for withstanding seismic loadings.</p> <p>The objective of this evaluation is to qualify the 232Z Building in it's current configuration for the appropriate seismic loads in accordance with the requirements of SDC 4.1.</p> <p>The structural capacities of the masonry walls and the reinforced concrete slabs and beams of the 232Z Building exceed the dead, live and seismic demand loadings. The building complies with the UBC seismic loading requirements for the 0.20 g maximum ground acceleration earthquake.</p>		
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9. Impact Level SQ		

232Z BUILDING SEISMIC ASSESSMENTS

September 1994

Prepared by: J. A. Ryan Date: 09/30/94
J. A. Ryan, Professional Support
Structural Assessments

Reviewed by: G. R. Wagenblast Date: 9/30/94
G. R. Wagenblast, Principal Engineer
Structural Assessments

Approved by: T. J. Conrads Date: 10/20/94
T. J. Conrads, Manager
Structural Assessments

WESTINGHOUSE HANFORD COMPANY
For the U.S. Department of Energy
Richland Operations Office (RL)

MASTER

TABLE OF CONTENTS

	<u>PAGE</u>
1.0 INTRODUCTION	1
2.0 SUMMARY OF RESULTS AND CONCLUSIONS	1
3.0 DISCUSSION	2
3.1 BUILDING DESCRIPTION	2
3.2 SEISMIC DISCUSSION	2
3.3 VISUAL INSPECTION	3
4.0 RESULTS	3
5.0 DRAWINGS	4
6.0 REFERENCES	5
TYPICAL CHECKLIST FOR INDEPENDENT REVIEW	6
DESIGN VERIFICATION METHOD	7
CALCULATIONS	A1
RCR COMMENTS	B1

232Z BUILDING SEISMIC ASSESSMENTS

1.0 INTRODUCTION

The 232Z Building in the 200 West Area has no documented evidence of meeting the structural design requirements for withstanding seismic loadings. The 232Z Building was originally built in 1959. This concrete block structure is south of the 234-5Z Building. The 232Z Building (Incinerator and Leach Facility) start-up occurred in 1961, and the facility has been used for the extraction and recovery of plutonium from contaminated materials. A glovebox incinerator is located in the first floor process area, and was used to incinerate dry wastes from which radioactive materials were then recovered. The design calculations for the original building are not retrievable.

The objective of this evaluation is to qualify the 232Z Building in it's current configuration for the appropriate seismic loads in accordance with the requirements of SDC 4.1.

2.0 SUMMARY OF RESULTS AND CONCLUSIONS

This report presents the structural evaluation of the 232Z Building to assess the adequacy of the concrete masonry walls and the roof structure to withstand the Hanford Site Design Basis Earthquake (DBE) for a safety class 3 facility which houses the safety class 1 incinerator glovebox.

The walls and the roof slab are adequate for resisting the dead, live, and seismic demand loads.

The material properties used in this structural evaluation were based upon the drawings and the construction material specification. No signs of material degradation were evident during the visual inspection of the facility. UCRL-15910 recommends the use of specified material properties unless insitu test values provide justifiably "higher" values for analysis. The use of insitu testing was not warranted.

3.0 DISCUSSION

3.1 BUILDING DESCRIPTION

The 232Z Building consists of lightly reinforced load bearing concrete masonry walls supporting a concrete-on-steel interior floor and roof. The roof and the interior floor construction consists of a corrugated metal deck with concrete fill (see reference drawings).

Lateral force resistance is provided by the masonry shear capability in the exterior masonry walls.

3.2 SEISMIC DISCUSSION

Current WHC design criteria originate from DOE Order 6430.1A which specifies the use of the design criteria guidelines for natural phenomena hazards provided in UCRL-15910, 1990. These guidelines are implemented in the site-specific design criteria SDC 4.1 (DOE-RL, 1994) for the Hanford Site.

The seismic loading for the design of new structures is defined in SDC 4.1, Rev. 12. The 232-Z building is classified as a safety class 3 facility which houses the safety class 1 incinerator glovebox. The building is not intended to provide confinement for the radiological hazard confined within the incinerator glove box. The 232Z Building (safety class 3) whose failure would compromise the function of the safety class 1 incinerator glovebox must be evaluated to, and meet, the site non-reactor safety class 1 seismic loads. The applicable criterion is that the building must not catastrophically collapse and endanger the incinerator glovebox. Therefore, the UBC design requirements are to be applied.

The building is analyzed using the UBC methodology for the safety class 3 facility and the maximum ground acceleration for the safety class 1 glovebox. In accordance with safety class 1 criteria, loads from the three orthogonal seismic directions are combined to determine the overall seismic demand.

The ground acceleration used in the 232Z evaluation is 0.20 g zero period acceleration (ZPA) anchoring a Newmark-Hall median based spectra as specified in SDC 4.1 for safety class 1 facilities. In accordance with the UBC, the 5% damped, Newmark-Hall spectrum was used for the dynamic amplification factor.

The 232Z Building is to be analyzed in its present configuration for the appropriate seismic requirements of SDC-4.1, Rev. 12. The evaluation approach will be to apply the analyses procedures of UBC-1994. The structural

evaluation is limited to the 232Z masonry walls, reinforced concrete columns, and reinforced concrete roof slab and beam. The analysis method utilized equivalent static methods.

The mass of the snow was not included in the mass of the structure for the seismic dynamic response in accordance with UCRL-15910. The mass of the snow was included in the DW+LL for live load effects which were appropriately combined with seismic loads. Additional loads to account for floor mounted items were considered in the analysis.

3.3 VISUAL INSPECTION

A visual inspection of the 232Z Building was conducted. The purposes of the inspection were (1) Visually verify the building structure to the drawings, in particular the lateral load carrying elements; (2) Inspect for concentrated live loads that would influence the response of the structure; (3) Inspect for undocumented modifications, additions, or deletions; (4) Inspect for obvious deterioration.

Observations made during the visual inspections were (1) The drawings accurately depict the as-built configuration of the structure; (2) No obvious deterioration or corrosion was noted; (3) No concentrated loads that would significantly influence the response of the structure were noted; (4) Roof weather protection consists of 5 cm (2 in.) of built-up roofing, 3 cm (1-1/4 in.) tar and felt, and 2 cm (3/4 in.) gravel.

4.0 RESULTS

The following section describes the results of the analysis. Details of the results can be found in the Appendix. The attached appendix provides the detailed calculations for dead load, live load, seismic load, load combinations, and structural capacity.

Dead load calculations consisted of determining weights of materials based upon the design drawings. Dead load calculations are provided for the 232Z roof, the 232Z interior floor, and the masonry walls.

Weight and weight distribution are calculated and the seismic moment and shear in the 232Z south wall, the 232Z north wall, and the 232Z column line D wall are determined. The 232Z Building south wall and the 232Z Building column line D wall were evaluated for shear and moment capacity and found to

be adequate. The 232Z north wall having lower loads and a capacity equivalent to the 232Z column line D wall was not evaluated. The east, west and column line 3 walls of the building are longer and consequently have greater capacity than the north and column line D walls and were not checked.

Design load factors are applied in accordance with UCRL-15910 and the UBC.

Structural capacities are based on the methods of the UBC using design material properties and design configurations.

The structural capacities of the masonry walls and the reinforced concrete slabs and beams of the 232Z Building exceed the dead, live and seismic demand loadings. The building complies with the seismic loading requirements for the 0.20 g maximum ground acceleration earthquake.

The 232Z masonry walls were checked for compliance to current UBC minimum requirements for reinforcement. The vertical reinforcement as specified on drawing H-2-23102 satisfies requirements for vertical steel except (a) the drawing does not specifically call out for reinforcement to be installed in the corner cells and (b) the embedment length of the anchoring dowels is not specified. The horizontal reinforcement in a shear wall is considered non-structural (Ref. TM 5-809-10, Chapter 13).

5.0 DRAWINGS

DRAWINGS:

H-2-23102	Structural Foundation Plan and Details
H-2-23105	Architectural Details
H-2-23106	Architectural Elevations Sections and Details
H-2-23107	Architectural Doors and Finish Schedule and Details
H-2-23108	Architectural Details

6.0 REFERENCES

- ACI, 1985, Code Requirements for Nuclear Safety Related Concrete Structures, ACI-349, American Concrete Institute, Detroit, Michigan.
- ACI, 1989, Building Code Requirements for Reinforced Concrete, ACI-318, American Concrete Institute, Detroit, Michigan.
- ASTM C 94-58, 1958, Specifications for Ready-Mixed Concrete.
- ASTM C 90-52, 1952, Specification s for Hollow Load-bearing Concrete Masonry Units.
- ASTM A 15-58T, 1958, Specifications for Billet-Steel Bars for Concrete Reinforcement.
- ASTM A 185-58T, 1958, Specifications for Welded Steel Wire Fabric for Concrete Reinforcement.
- DOE Order 6430.1A, United States Department of Energy, General Design Criteria, U.S. Department of Energy, Washington, D.C.
- DOE-RL, 1994, Standard Architectural-Civil Design Criteria, Design Loads for Facilities, Hanford Plant Standards, SDC-4.1, Rev. 12, U.S. Department of Energy, Richland Operations Office, Richland, Washington, 1994.
- HWS-6495, 1959, Specifications for Phase I Construction, 232-Z Building Plutonium Recovery from Contaminated Materials. Project CGC-813, May 12, 1959.
- TM 5-809-10, Technical Manual, Seismic Design for Buildings, Department of the Army, the Navy, and the Air Force, 1982
- UBC, 1994, Volume 2, Uniform Building Code, UBC, International Conference of Building Officials, Whittier, California.
- UCRL, 1990, Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards, UCRL-15910, Livermore, California.

TYPICAL CHECKLIST FOR INDEPENDENT REVIEW

Document Reviewed WHC-SD-WM-DCR-002

Author J. A. RYAN

Yes No N/A

☒	☐	☐	Problem completely defined.
☒	☐	☐	Necessary assumptions explicitly stated and supported.
☐	☐	☒	Computer codes and data files documented.
☒	☐	☐	Data used in calculations explicitly stated in document.
☒	☐	☐	Data checked for consistency with original source information as applicable.
☒	☐	☐	Mathematical derivations checked including dimensional consistency of results.
☒	☐	☐	Models appropriate and used within range of validity or use outside range of established validity justified.
☒	☐	☐	Hand calculations checked for errors.
☒	☐	☐	Acceptability limits on analytical results applicable and supported. Limits checked against sources.
☒	☐	☐	Safety margins consistent with good engineering practices.
☒	☐	☐	Conclusions consistent with analytical results and applicable limits.
☒	☐	☐	Results and conclusions address all points required in the problem statement.

MANDATORY

Software QA Log Number NA

G. R. Wagenblast
Reviewer

9-30-94
Date

DESIGN VERIFICATION METHOD

The need for design verification has been reviewed with the method selected as indicated below:

<u> x </u>	Independent Review
<u> </u>	Alternate Calculations
<u> </u>	Qualification Testing
<u> </u>	Formal Design Review

T. J. Conrads
Cognizant/Project/Design Manager

SD # WHC-SD-WM-DCR-002

ECN # N/A

DWG(S) # N/A

Page A2 of

Checker _____ Date _____

SEISMIC ASSESSMENT

ANALYTICAL CALCULATIONS

WHC-SD-WM-DCR-002

REV 0
Page A1 of 36

Subject 232 Z BUILDING SEISMIC ASSESSMENT
Originator J.A. Ryan Date 09/08/94
Checker NRWagenblast Date 9/29/94

MATERIAL SPECIFICATIONS/REFERENCES

ROOF:

- METAL DECK, GRANCO STEEL PROD. CO.

- 24 ga x 1.25" (COFAR OR EQUAL)

- $t = 0.0239"$

- $w = 2.0 \text{ pcf}$ (WHC-SD-FF-SA-011)

- Concrete

- 3000 psi

- $w = 150 \text{ pcf}$

- REBAR

- ASTM A15-58T (INTERMEDIATE GRADE/DEFORMED)

WALLS:

- MASONRY WALLS

- HOLLOW CONCRETE BLOCK

- ASTM C 90-52

- 8 x 8 x 16

- 8 x 12 x 16

- REBAR

- ASTM A15-58T (INTERMEDIATE GRADE/DEFORMED)

FLOOR (SECOND):

- METAL DECK

- 22 ga x 1.25

- $t = 0.0299$

- $w = 2.5 \text{ pcf}$

- CONCRETE/REBAR - AS ABOVE

WHL-SD-WM-DCR-002

ANALYTICAL CALCULATIONS

REV 0

Page A2 of Subject 232Z BUILDING SEISMIC ASSESSMENTOriginator J.A. Ryan Date 09/08/94Checker YRWagrell Date 9/29/94

△ = COLUMN

WHC-SD-WM-DUR-002

ANALYTICAL CALCULATIONS

REV 0
Page A3 ofSubject 232 Z BUILDING SEISMIC ASSESSMENTOriginator J.A. Ryan Date 09/02/94Checker M.P. Jorgensen Date 9/29/94ROOF

CALCULATION OF WEIGHTS:

CONCRETE:	4in @ 150pcf	50psf
METAL:	24ga x 1.25"	2
INSUL:	2" @ 1.5psf/in	3
GRAVEL:	3/4" @ 109pcf	7
ASPH/FLT:	1 1/4"	8
		<hr/> 70 psf

USE 70psf

ZONE	AREA	WT (#)
A	338	23,660
B	133	9310
C	1155	80,850
D	453	31,710
	$\Sigma =$	<hr/> 145,530 #

ANALYTICAL CALCULATIONS

WHC-SD-WM-DCR-002

Page A4 of REV DSubject 232 Z BUILDING SEISMIC ASSESSMENTOriginator J.A. RyanDate 09/08/94Checker SRWDate 9/29/94SECOND FLOOR

COL. 1-4 / D-E

CONCRETE: 5 1/2" @ 150pcf

68.75 psf

METAL: 22ga @ 1.25"

2.5

WT. EQUIP. (ASSUMED)

5.0

AVG CONCR. DEPTH = h_{cA}

$$h_{cA} = \frac{(6.0 - 0.0299) + (4.75 - 0.0299)}{2} = 5.345"$$

$$WT = 68.75 (5.345 / 5.5) = 66.8 = 67 \text{ psf}$$

$$USE WT_{TOT} = 67 + 2.5 + 5.0 = 74.5 \text{ psf}$$

ZONE	AREA	WT (#)
A	338	25,200
B	133	9,900
Σ		35,100 #

ANALYTICAL CALCULATIONS

WHL-SD-WM-DLR-002

REV D

Page A5 of

Subject 232Z SEISMIC ASSESSMENTOriginator J.A. IZCDate 09/16/94Checker MRINagallantDate 9/29/94

8"-BLOCK X 8X16 (12X8X16)

Paranetical values for 12" BLOCK

(DIMENSIONS: ASTM C 90)

$$A_8 = (16)(2)(1\frac{1}{4}) + (3)(1)(5\frac{1}{8}) = 55.375 \text{ in}^2$$

$$A_{12} = (16)(2)(1\frac{1}{4}) + (3)(1\frac{1}{2})(9\frac{1}{2}) = 70.797 \text{ in}^2$$

$$V_8 = (55.375)(8) = 443 \text{ in}^3$$

$$V_{12} = (70.797)(8) = 566.375 \text{ in}^3$$

$$WT_8 = \frac{(443)(150)}{(1728)} \left\{ \frac{1}{(8)(16)/144} \right\} = 43.26 \text{ psf}$$

$$WT_{12} = \frac{(566.375)(150)}{(1728)} \left\{ \frac{1}{(8)(16)/144} \right\} = 55.31 \text{ psf}$$

● REPAIR: 2 #4 @ 48" - CELL GROUTED -

ANALYTICAL CALCULATIONS

REV 0

Page A6 of

Subject 232 Z BUILDING SEISMIC ASSESSMENT
 Originator J. A. Ryan Date 09/16/94
 Checker APU/agm/est Date 9/25/94

PLASTER

ROOMS ON GROUND FLOOR COL'S 1-4 & A-D

PLASTER (REF AISC MANUAL) $\frac{1}{2}$ " THICK 2.5 psf

1" THICK 5.0 psf

REBAR

$$\begin{aligned} WT &= (2)(0.668) \text{ \#/FT} \\ &= (2)(0.668)/4 \text{ psf} \\ &= 0.3 \text{ psf} \end{aligned}$$

GROUT (1 CELL / 4' LINEAR)

$$VOL_8 = \frac{(8)(5\frac{1}{2})(4\frac{1}{2})}{1728} = \frac{184.5}{1728} \text{ ft}^3$$

$$WT = \frac{(184.5)(150)}{(1728)(4)} = 4.00 \text{ psf (8" BLOCK)}$$

$$VOL_{12} = \frac{(8)(9\frac{1}{2})(4\frac{1}{2})}{1728} = 0.190 \text{ ft}^3$$

$$WT = (0.190)(150/4) = 7.13 \text{ psf (12" BLOCK)}$$

ANALYTICAL CALCULATIONS

Rev 0

Page A7 of

Subject 2322 BUILDING SEISMIC REASSESSMENT
Originator J. D. Ryan Date 09/16/94
Checker JRM/agc/lost Date 9/29/94

WALL WEIGHTS (PSF)

8" WALL (1/2" PLASTER)

$$WT = 43.26 + 2.5 + 0.3 + 4.0 = 50 \text{ pcf}$$

8" WALL (1" PLASTER)

$$WT = 52.5 \text{ pcf}$$

8" WALL (W/O PLASTER)

$$WT = 47.5 \text{ pcf}$$

12" WALL (1/2" PLASTER)

$$WT = 55.31 + 2.5 + 0.3 + 7.13 = 65.25 \text{ pcf}$$

12" WALL (W/O PLASTER)

$$WT = 62.75 \text{ pcf}$$

ANALYTICAL CALCULATIONS

REV D

Page A3 of Subject 232 Z BUILDING SEISMIC ASSESSMENTOriginator J.A. RyanDate 09/09/94Checker SPU WallertDate 9/29/94WEST WALL $\frac{1}{2}$ " PLASTER ON INTERIOR FACE (COL' A $\rightarrow$ D) EAST WALL $\frac{1}{2}$ " PLASTER ON INTERIOR FACE

SAME AS EAST WALL

$$\Sigma W_w = 33,250 + 5995 + 7515 = 46,760 \text{ #}$$

$$\Sigma W_e = \Sigma W_w = 46,760 \text{ #}$$

ANALYTICAL CALCULATIONS

REV 0

Page A9 of Subject 232 Z BUILDING SEISMIC ASSESSMENTOriginator J.A. Ryan Date 09/09/94Checker APM Inc. Host Date 9/29/99SOUTH WALL

$$\Sigma W = 27,570 \text{ #}$$

WALL @ COL D

The 5'-10" height is 1/2" plaster (one-side) - The equivalent (weight/5.5 ft) is

$$\{(47.5)(4) + (50)(5.833)\} / 9.833 = \underline{49 \text{ psf}}$$

$$\Sigma W = 12,850 + 4820 + 16,240 + 6070 = 39,980 \text{ #}$$

ANALYTICAL CALCULATIONS

REV 0

Page A 10 of _____

Subject 232 Z BUILDING SEISMIC ASSESSMENT

Originator J. A. Tolan

Date 09/16/94

Checker MRWagelast

Date 9/29/94

WALL AT COL 3

1" PLASTER (COL A-D)

$$\Sigma W = 34,900 + 5690 = 40,590 \#$$

NORTH WALL

$$\Sigma WT = 38,765 \#$$

ANALYTICAL CALCULATIONS

WHL-SD-WM-002-002

REV. 0
Page 111 of

Subject 232 Z Building Seismic Survey

Originator J.A. Ziga

Date 09/28/92

Checker J. Puljan

Date 9/29/92

EXCITATION IN X-DIRECTION (E/W)SOUTH WALL

ELEM	WEIGHT CALC.	$\bar{h}$	WT	(WT)*($\bar{h}$)
ROOF DL	(70)(1608/2)	15.167	56,280	853,580
ROOF LL	(20)(1608/2)	15.167	16,080	243,900
WEST WALL (15'-2"h)	(50)(43.833)(15.167)/2	7.583	16,620	126,000
COL "3" WALL (15'-2"h)	(52.5)(43.833)(15.167)/2	7.583	17,450	132,300
EAST WALL (15'-2"h)	(50)(43.833)(15.167)/2	7.583	16,620	126,000
SOUTH WALL (15'-2"h)	(50)(36.667)(15.167)	7.583	27,810	210,860
Σ (W/O LL)			134,780	1,448,740

$$\bar{h} = (1,448,740)/(134,780) = 10.75'$$

WALL @ COLUMN D

ELEM	WEIGHT CALC.	$\bar{h}$	WT	(WT)*($\bar{h}$)
ROOF DL	(70)(471/2)	19.167	16,500	316,900
ROOF LL	(20)(471/2)	19.167	4710	90,300
WEST WALL (9'-10"h)	(47.5)(12.833)(9.833)/2	14.25	3000	42,750
EAST WALL (9'-10"h)	(47.5)(12.833)(9.833)/2	14.25	3000	42,750
WEST WALL (9'-4"h)	(62.75)(12.833)(9.333)/2	4.667	3760	17,500
EAST WALL (9'-4"h)	(62.75)(12.833)(9.333)/2	4.667	3760	17,500
ROOF DL	(70)(1608/2)	15.167	56,280	853,600
ROOF LL	(20)(1608/2)	15.167	16,080	243,900
FLOOR	(74.5)(471/2)	9.333	17,540	163,800
WEST WALL (15'-2"h)	(50)(43.833)(15.167)/2	7.583	16,200	126,000
EAST WALL (15'-2"h)	(50)(43.833)(15.167)/2	7.583	16,200	126,000
COL "3" WALL (15'-2"h)	(52.5)(43.833)(15.167)/2	7.583	17,450	132,300
COL "3" WALL (9'-4"h)	(47.5)(12.833)(9.333)/2	4.667	2840	13,300
COL "D" WALL (9'-10"h)	(49)(36.667)(9.833)	14.25	17,670	251,800
COL "D" WALL (9'-4"h)	(65.25)(36.667)(9.333)	4.667	22,330	104,200
Σ (W/O LL)			196,530	2,208,400

$$\bar{h} = (2,208,400)/(196,530) = 11.24'$$

WHL-SD-WM-DLR-002

Page 12 of

Date 05/28/94

Date 9/29/94

ANALYTICAL CALCULATIONS

UHL-SD-WM-DCR-002

Rev 0
Page 13 of

Subject 232 E Building Seismic Assessment

Originator J.A. Bauer Date 05/20/94

Checker R. J. Bauer Date 9/29/94

EXCITATION IN X-DIRECTION (E/W)

NORTH WALL

ELEM	WEIGHT CALC.	h	WT	(WT)*(h)
ROOF DL	(70)*(471/2)	19.167	16,500	316,000
ROOF LL	(20)*(471/2)	19.167	4710	90,300
WEST WALL (9'-10"h)	(47.5)(12.833)(9.833)/2	14.333	3000	43,000
EAST WALL (9'-10"h)	(47.5)(12.833)(9.833)/2	14.333	3000	43,000
WEST WALL (9'-4"h)	(62.75)(12.833)(9.333)/2	4.667	3760	17,500
EAST WALL (9'-4"h)	(62.75)(12.833)(9.333)/2	4.667	3760	17,500
FLOOR	(74.5)(471/2)	9.333	17,540	163,800
COL "3" WALL (9'-4"h)	(47.5)(12.833)(9.333)/2	4.667	2840	13,300
NORTH WALL (9'-11½"h)	(47.5)(36.667)(9.958)	14.312	17,340	248,200
NORTH WALL (9'-4"h)	(62.75)(36.667)(9.333)	4.667	21,470	100,900
Σ (W/O LL)			89,210	963,200

$$h = (963,200)/(89,210) = 10.8'$$

ANALYTICAL CALCULATIONS

REV 0

Page A 14 of _____

Subject 232 Z BUILDING SEISMIC ASSESSMENTOriginator N.A. Ryan Date 09/15/94Checker YRW/consult Date 9/29/94SEISMIC FORCE DISTRIBUTION

THE 232 Z SEISMIC ANALYSIS IS DRIVEN BY THE PRESENCE OF THE SC 1 SYSTEM (INCINERATOR GLOVE BOX) IN THE SC 3 BUILDING. THE BUILDING DOES NOT PROVIDE CONFINEMENT - THE REQUIREMENT IS THAT IN A SEISMIC EVENT, EGRESS BE ENSURED - THE BUILDING IS NOT TO COLLAPSE. THE ANALYSIS WILL USE SC 1 LOADINGS IN AN EQUIVARIANT STATIC USING UBC.

DESIGN BASE SHEAR

$$V = \frac{ZIC}{R_w} W \quad (28-1/UBC)$$

$$Z = 0.29 \quad (\text{TABLE 16-I/ZONE 2})$$

$$C = 2.75 \quad (\text{MAXIMUM})$$

$$I = 1.25 \quad (\text{TABLE 16-K})$$

$$R_w = 6.0 \quad (\text{TABLE 16-N})$$

$$W = \text{TOTAL SEISMIC LOADS}$$

$$V = (0.20)(1.25)(2.75)(W/6)$$

$$= 0.11458 W$$

ANALYTICAL CALCULATIONS

WHC-SD-WM-DCR-002

Rev 0
Page 115 of

Subject 232 Z BUILDING Seismic Assessment

Originator J. A. Raga

Date 09/28/94

Checker M. R. Raga

Date 9/29/94

SOUTH WALL

$$V = 0.11458W = (0.11458)(134,780) = 15,443^{\#}$$

$$V/(\Sigma Wx) = (15,443)/(1,448,740) = 0.010660$$

$\bar{h}$	$F = \{(Wx)/(\Sigma Wx)\}V$	F	ΣF	M	ΣM
15.167	(56,280)(15.167)(0.010660)	9099	9099	138,000	138,000
7.583	(16,620)(7.583)(0.010660)	1344	10,443	10,190	148,190
7.583	(17,450)(7.583)(0.010660)	1411	11,854	10,700	158,890
7.583	(16,620)(7.583)(0.010660)	1344	13,198	10,190	169,080
7.583	(27,810)(7.583)(0.010660)	2248	15,446	17,050	186,130
Σ			15,446		186,130

ANALYTICAL CALCULATIONS

WHE-SD-WM-D-4-002

Rev 0

Page 116 of 116

Subject 232 E Building Seismic AssessmentOriginator J.A. RyzDate 07/29/14Checker Shouqian LiDate 7/29/14

COLUMN "D" WALL

$$V = (0.11458W = (0.11458)(196,530) = 22,518^{\#}$$

$$V/(\Sigma W_x) = (22,518)/(2,208,400) = 0.010197$$

h	$F = ((W_x)/(\Sigma W_x))V$	F	ΣF	M	ΣM
19.167	$(16,500)(19.167)(0.010197)$	3225	3225	61,810	61,810
15.167	$(56,280)(15.167)(0.010197)$	8704	11,929	132,010	193,820
14.25	$(2)(3000)(14.25)(0.010197)$	872	12,801	12,420	206,240
14.25	$(17,670)(14.25)(0.010197)$	2568	15,369	36,590	242,830
9.333	$(17,540)(9.333)(0.010197)$	1669	17,038	15,580	258,410
7.583	$(2)(16,200)(7.583)(0.010197)$	2505	19,543	19,000	277,410
7.583	$(17,450)(7.583)(0.010197)$	1349	20,892	10,230	287,640
4.667	$(2)(3760)(4.667)(0.010197)$	358	21,250	1670	289,310
4.667	$(2840)(4.667)(0.010197)$	135	21,385	630	289,940
4.667	$(22,330)(4.667)(0.010197)$	1063	22,448	4960	294,900
Σ			22,448		294,900

ANALYTICAL CALCULATIONS

WHE-SD-WM-DLR-001

Rev 0

Page A17 of

Subject

232-EBuildingSeismicAssessment

Originator

J. A. Ryzar

Date

09/20/94

Checker

Michael West

Date

9/29/94

NORTH WALL

$$V = 0.11458W = (0.11458)(89,210) = 10,222$$

$$V/(\Sigma Wx) = (10,222)/(963,200) = 0.01061$$

h	$F = ((Wx)/(\Sigma Wx))$	F	ΣF	M	ΣM
19.167	$(16,500)(19.167)(0.01061)$	3355	3355	64,310	64,310
14.333	$(2)(3000)(14.333)(0.01061)$	913	4268	13,090	77,400
14.312	$(17,340)(14.312)(0.01061)$	2633	6910	37,680	115,080
9.333	$(17,540)(9.333)(0.01061)$	1737	8638	16,210	131,290
4.667	$(2)(3760)(4.667)(0.01061)$	372	9010	1740	133,030
4.667	$(2840)(4.667)(0.01061)$	141	9151	660	133,690
4.667	$(21,470)(4.667)(0.01061)$	1063	10,214	4960	138,650
Σ			10,214		138,650

ANALYTICAL CALCULATIONS

REV 0
Page A18 of _____Subject: 232 Z BUILDING SEISMIC ASSESSMENTOriginator: J.A. RyanDate: 09/28/94Checker: M.R. WagantsootDate: 9/29/94232-Z SOUTH WALL• EVALUATE AS A SHEAR WALL

- $DL = 56,280 + 27,810 = 84,090 \text{ lbs}$
- $LL = 16,080 \text{ lbs}$
- $SEISMIC \text{ SHEAR} = 15,446 \text{ lbs}$
- $SEISMIC \text{ MOMENT} = 186,130 \text{ ft-lbs}$

• WALL PROPERTIES

- BLOCK - NOMINAL 8-in WIDTH
- WALL LENGTH = $36.667 \text{ ft} = 440 \text{ in}$
- WALL HEIGHT = $15.167 \text{ ft} = 182 \text{ in}$
- $f'_m = 1000 \text{ psi}$ (ASTM C90-52)
- CONCRETE STRENGTH $f'_m = 3000 \text{ psi}$ (HWS-6495)
- REINFORCING STEEL $f_c = 40,000 \text{ psi}$ (ASTM A15-58T)

• LOAD COMBINATIONS (UBC/SECTION 2108)

- $U = 1.4D + 1.7L$ (8-3)
- $U = 1.4(D + L + E)$ (8-1)

• FACTORED AXIAL LOAD

- $P_{FL} = 1.4D + 1.7L = (1.4)(84,090) + (1.7)(16,080)$
 $= 145,060 \text{ lbs}$

• AXIAL LOAD CAPACITY

- $P_o = \phi(0.8)\{(0.85)f'_m(A_c - A_s) + f_y A_s\}$ (8-43)
- $\phi = 0.65$ SECTION 2108.1.4.3.1)

ANALYTICAL CALCULATIONS

Rev 0

Page A19 of

Subject 232 Z BUILDING SEISMIC ASSESSMENT
 Originator J. A. Ryan Date 05/28/94
 Checker J. H. Wagonblast Date 9/29/94

NOTE: The vertical reinforcing steel will not be considered, and the masonry blocks will be treated as hollow blocks without grout filler.

- $P_o = (0.65)(0.8)(0.85)(1000)(A_c)$
- $A_c = (55.375)(440 - 76 - 40)/(16) = 1121 \text{ sq in}^*$
- $P_o = 495,600^\# > 145,060^\#$ (Therefore the axial load is okay)
- FACTORED SHEAR

- $V_{FL} = 1.4V_s = (1.4)(15,446) = 21,620^\#$

- $V_m = C_d A_{mv} (f'_m)^{1/2}$ (8-46)

FROM TABLE 21-K

- $M/(Vd) = (186,130)/\{(15,446)(36.667)\} = 0.329$

Since this values lies between 0.25 and 1.0, C_d is found by interpolation.

- $C_d = 2.4 - \{(1.2/0.75)(0.329 - 0.25)\} = 2.274$

- $A_{mv} = 1121 \text{ sq in}$

- $V_m = (2.274)(1121)(1000)^{1/2} = 80,600^\#$

Again, neglecting the steel reinforcement and using hollow block section;

- $V_u = \phi V_m = (0.6)(80,600) = 48,260^\# > 21,620^\#$

* * * * *

- CAPACITY OF THE STEEL

- $V_s = A_{mv} \rho f_y$ (2 #4's @ 48 in)

- $\rho = \{(2)(0.2)\}/\{(48)(7.625)\} = 1.093 \times 10^{-3}$

- $V_s = (1121)(1.093 \times 10^{-3})(40,000) = 49,000^\#$

- $\phi V_s = (0.8)(49,000) = 39,200^\# > 21,620^\#$

($\therefore$ Capacity of the steel is adequate if the wall is cracked)

* See next page for dimensions of wall.

ANALYTICAL CALCULATIONS

REV 0

Page A20 of Subject 232 Z BUILDING SEISMIC ASSESSMENTOriginator J. A. RyanDate 09/27/94Checker HPWDate 9/29/94

$$\bar{I} = (7.625) \left\{ \frac{(40)^2}{2} - (40)(60) + (92)(80+40) - (76)(210) + (192)(344) \right\} = 185.43$$

$$(7.625)(440-76-40)$$

CALCULATE 'I': THERE ARE 5 SECTIONS (3 ADDITIVE / 2 NEGATIVE)

$$+ ① \frac{(7.625)(40)^3}{(12)} + (7.625)(40)(185.43 - 20)^2 = + 8.383 \times 10^6$$

$$+ ② \frac{(7.625)(92)^3}{(12)} + (7.625)(92)(185.43 - 126)^2 = + 2.973 \times 10^6$$

$$+ ③ \frac{(7.625)(192)^3}{(12)} + (7.625)(192)(344 - 185.43)^2 = + 41.308 \times 10^6$$

$$- ④ \frac{(7.625)(40)^3}{(12)} + (7.625)(40)(185.43 - 60)^2 = - 4.833 \times 10^6$$

$$- ⑤ \frac{(7.625)(76)^3}{(12)} + (7.625)(76)(210 - 185.43)^2 = - 0.629 \times 10^6$$

$$\bar{I} = 47.201 \times 10^6 \text{ in}^4$$

$$S = \frac{\bar{I}}{C} = (47.201 \times 10^6) / (440 - 185.43) = 185,410 \text{ in}^3$$

ANALYTICAL CALCULATIONS

REV 0

Page A 21 of

Subject 237-Z Building Seismic Assessment
 Originator J.A. Rya Date 07/29/94
 Checker MR. W. J. O'Neil Date 9/29/94

- FACTORED MOMENT LOAD

- $M_{FL} = 1.4M_s = (1.4)(186,130) = 260,580$

- $M_{CR} = S\{0.9(P/A) + f_r\}$

where: $f_r = 2.5(f'_m)^{\frac{1}{2}} = 79 \text{ psi} \quad (8-41)$

$$P = DL = 84,090^{\#}$$

$$A = 1121 \text{ in}^2$$

- $M_{CR} = (185,410)\{(0.9)(84,090/1121) + 79\}$
 $= 27.165 \times 10^6 \text{ in-lbs}$
 $= 2.263 \times 10^6 \text{ ft-lbs} > 260,580 \text{ ft-lbs}$

Therefore the wall will not crack.

- CHECK OVERTURN

Dead load (DL) resisting moment is calculated by

$$(DL)(L/2) = (84,090)(36.667/2)$$

$$= 1.542 \times 10^6 \text{ ft-lbs}$$

$$SF = (1.542 \times 10^6)/(260,580) = 5.91 \text{ (OK)}$$

ANALYTICAL CALCULATIONS

REV 0

Page A22 of

Subject 232-Z Building Seismic Assessment
 Originator J. A. Rya Date 07/29/94
 Checker JRWagambest Date 9/29/94

- ULTIMATE CRACKED MOMENT CAPACITY

- $F_s = (2)(0.2)(40,000) = 16,000 \text{ psi}$
- Assume that the rebars displaced by the doors are not relocated. Thus there are only 7 rebars as shown in the sketch on the next page.
- Assume the neutral axis ($\bar{c}$) is 32.0" from the compression face. See sketch next page.
- The strain at the first bar (Where the maximum allowable concrete strain is $\epsilon = 0.003$)

- $\epsilon_b = \{(32-4)/32\}(0.003) = 0.0026$

$\therefore$ FULLY EFFECTIVE

- $T_s = (6)(16,000) = 96,000^\#$
- $C_s = 16,000^\#$ (Compression in steel bar)
- $C_m = (0.85)(0.85)(f'_m)(A)$
 $= (0.85)^2(1000)\{(55.375)(32/16)\} = 80,000^\#$
- $T - C_s - C_m = 96,000 - (16,000 + 80,000) = 0$

$\therefore$ The assumed neutral axis location is okay

THE MOMENT AT THE EXTREME COMPRESSIVE FIBER

See sketch on the next page.

- $M = - (80,000)(32.0/2) - (16,000)(4)$
 $+ (4 + 52 + 100 + 148 + 292 + 340 + 436)(16,000)$
 $= -1.344 \times 10^6 + (1372)(16,000)$
 $= 20.608 \times 10^6 \text{ in-lbs} = 1.717 \times 10^6 \text{ ft-lbs} > 260,580 \text{ ft-lbs}$
- $\therefore$ The margins for the south wall are all positive.

ANALYTICAL CALCULATIONS

REV D
Page 23 of

Subject 232 Z BUILDING SEISMIC ASSESSMENT

Originator J.A. Rza

Date 09/28/94

Checker J.P. Wagerlat

Date 9/29/94

STRAIN PROFILE & FORCE EQUILIBRIUM

ANALYTICAL CALCULATIONS

REV'D

Page A24 of

Subject 232-Z Seismic Assessment
 Originator J. A. Rya Date 09/29/94
 Checker M. R. Wagonblast Date 9/29/94

232-Z COLUMN 'D' WALL• EVALUATE AS A SHEAR WALL

- $DL = 16,500 + 56,280 + 17,540 + 17,670 + 22,330 = 130,320 \text{ lbs}$
- $LL = 4710 + 16,080 = 20,790 \text{ lbs}$
- $SEISMIC \text{ SHEAR} = 22,448 \text{ lb}$
- $SEISMIC \text{ MOMENT} = 294,900 \text{ ft-lbs}$

• WALL PROPERTIES

- BLOCK - NOMINAL 12-in WIDTH (To height of 9'-4")
 - NOMINAL 8-in WIDTH (9'-4" to roof)
- WALL LENGTH = 36.667 ft = 440 in
- WALL HEIGHT = 19.167 ft = 230 in
- $f'_m = 1000 \text{ psi}$ (ASTM C90-52)
- CONCRETE STRENGTH $f'_m = 3000 \text{ psi}$ (HWS-6495)
- REINFORCING STEEL $f_c = 40,000 \text{ psi}$ (ASTM A15-58T)

• LOAD COMBINATIONS (UBC/SECTION 2108)

- $U = 1.4D + 1.7L$ (8-3)
- $U = 1.4(D + L + E)$ (8-1)

• FACTORED AXIAL LOAD

- $P_{FL} = 1.4D + 1.7L = (1.4)(130,320) + (1.7)(20,790)$
 $= 217,790 \text{ lbs}$

• AXIAL LOAD CAPACITY

- $P_o = \phi(0.8)(0.85)\{f'_m(A_c - A_s) + f_y A_s\}$ (8-43)
- $\phi = 0.65$ SECTION 2108.1.4.3.1)

ANALYTICAL CALCULATIONS

REV 0
Page A25 of

Subject 232-Z Building Seismic Assessment
 Originator J.A. Rye Date 09/29/94
 Checker Y.R. Wagonblast Date 09/29/94

NOTE: The vertical reinforcing steel will not be considered, and the masonry blocks will be treated as hollow blocks without grout filler.

- $P_o = (0.65)(0.8)(0.85)(1000)(A_c)$
 - $A_c = (70.797)(440 - 40)/(16) = 1770 \text{ sq in}^*$
- $P_o = 782,300^{\#} > 217,790^{\#}$ (Therefore the axial load is okay)

- FACTORED SHEAR

- $V_{FL} = 1.4V_s = (1.4)(22,448) = 31,430^{\#}$
- $V_n = C_d A_{mv} (f'_m)^{1/2}$

FROM TABLE 21-K

- $M/(Vd) = (294,900)/(22,448)(36.667) = 0.358$

Since this values lies between 0.25 and 1.0, C_d is found by interpolation.

- $C_d = 2.4 - \{(1.2/0.75)(0.358 - 0.25)\} = 2.23$
- $A_{mv} = 1770 \text{ sq in}$
- $V_m = (2.23)(1770)(1000)^{1/2} = 124,820^{\#}$

Again, neglecting the steel reinforcement

- $V_u = \phi V_m = (0.6)(124,820) = 74,890^{\#} > 31,430^{\#}$

* * * * *

- CAPACITY OF THE STEEL

- $V_s = A_{mv} \rho f_y$ (2 #4's @ 48 in)
 - $\rho = \{(2)(0.2)\}/\{(48)(11.625)\} = 7.168 \times 10^{-4}$
- $V_s = (1770)(7.168 \times 10^{-4})(40,000) = 50,750^{\#}$
- $\phi V_s = (0.8)(50,750) = 40,600^{\#} > 31,430^{\#}$

($\therefore$ Capacity of the steel is adequate if the wall is cracked)

* See next page for wall dimensions.

ANALYTICAL CALCULATIONS

WHC-SD-WM-DLR-002

REV 0

Page A26 of

Subject 232-Z BUILDING SEISMIC ASSESSMENT

Originator J. A. R. G.

Date 09/20/94

Checker JRM/acc/last

Date 9/29/94

$$\bar{I} = \frac{(11.625) \left\{ \frac{136^3}{2} - (40)(156) + (264)(308) \right\}}{(11.625)(440 - 40)} = 210.8'$$

CALCULATE I:

$$\textcircled{1} \frac{(11.625)(136)^3}{12} + (11.625)(136)(1428)^2 = + 34.676 \times 10^6$$

$$\textcircled{3} \frac{(11.625)(264)^3}{12} + (11.625)(264)(97.2)^2 = + 46.820 \times 10^6$$

$$\textcircled{-2} \frac{(11.625)(40)^3}{12} + (11.625)(40)(54.8)^2 = - 1.455 \times 10^6$$

$$I = 80.038 \times 10^6$$

$$S = I/c = (80.038 \times 10^6) / (440 - 210.8) = 349,200 \text{ in}^3$$

ANALYTICAL CALCULATIONS

REV D

Page A27 of

Subject 232-Z Building Seismic Assessment
 Originator J. A. Ryan Date 09/21/94
 Checker MR Waggoner Date 9/29/94

- FACTORED MOMENT LOAD

- $M_{FL} = 1.4M_s = (1.4)(294,900) = 412,860 \text{ in-lbs}$

- $M_{CR} = S\{0.9(P/A) + f_r\}$

where $f_r = 2.5(f'_m)^{1/2} = 79 \text{ psi} \quad (8-41)$

$$P = DL = 130,320^\#$$

$$A = 1770 \text{ in}^2$$

- $M_{CR} = (349,200)\{0.9(130,320/1770) + 79\}$
 $= 47,968 \times 10^6 \text{ in-lbs}$
 $= 3.997 \times 10^6 \text{ ft-lbs} > 412,860 \text{ ft-lbs}$

Therefore the wall will not crack

- CHECK OVERTURN

Dead load (DL) resisting moment is calculated by

$$(DL)(L/2) = (130,320)(36.667/2)$$

$$= 2.389 \times 10^6 \text{ ft-lbs}$$

$$SF = (2.389 \times 10^6)/(412,860) = 5.78 \text{ (OK)}$$

ANALYTICAL CALCULATIONS

REV D

Page A2B of

Subject 232-Z Building Seismic Assessment
 Originator J. A. Rya Date 07/24/94
 Checker MR Wagoner Date 9/29/94

- ULTIMATE CRACKED MOMENT CAPACITY

- $F_s = (2)(0.2)(40,000) = 16,000 \text{ psi}$

- Assume that the one pair of rebars are displaced by the door is not relocated. Therefore there are 9 pairs of rebars to carry load. See sketch on the next page.
- Assume the neutral axis (c) is 35" from the compression face.
- The strain at the first bar (where the maximum allowable concrete strain is $\epsilon = 0.003$)

- $\epsilon_b = \{(35-4)/35\}(0.003) = 0.0027$

- $\therefore$ FULLY EFFECTIVE

- $T = (8)(16,000) = 128,000^\#$
 - $C_s = 16,000^\#$ (Compression in the steel bar)
 - $C_m = (0.85)(0.85)(f'_m)(A)$
 $= (0.85)(0.85)(1000)\{(70.797)(35/16)\} = 111,890^\#$
 - $T - C_s - C_m = 144,000 - (16,000 + 111,890) = 108 \approx 0$

- $\therefore$ The assumed neutral axis location is okay.

THE MOMENT AT THE EXTREME COMPRESSIVE FIBER

- $M = - \{(111,890)(35/2) + (16,000)(4)\}$
 $+ \{4 + 52 + 100 + 148 + 196 + 244 + 340 + 388 + 436\}(16,000)$
 $= 26.548 \times 10^6 \text{ in-lbs} = 2.212 \times 10^6 \text{ ft-lbs} > 412,860 \text{ ft-lbs}$

The margins for the Column 'D' wall are all positive.

ANALYTICAL CALCULATIONS

REV 0
Page A29 of _____Subject 232 Z BUILDING SEISMIC ASSESSMENTOriginator J.A. RyaDate 09/28/94Checker MRWagenblastDate 9/29/94

STRAIN PROFILE @ FORCE EQUILIBRIUM

ANALYTICAL CALCULATIONS

REV 0

Page A30 of Subject 232 E BOLDING Seismic AssessmentOriginator J. A. RyanDate 09/28/94Checker M. WagnallDate 9/29/94TOTAL COMPRESSION:

The vertical seismic loading is $\frac{1}{2}(0.2g)$, SDC-4.1, Rev 12. Also, in accordance with SDC-4.1, the combined compressive loadings will be calculated using:

$$100 + 40 + 40.$$

Because the horizontal seismic loadings are taken as shear in the transverse walls, there is no orthogonal horizontal loading to be calculated for the two walls. Therefore the following are the loadings:

$$V_h = \frac{(ZIC)W}{R_w} = \{(\frac{1}{2})(0.2)(1.25)(2.75)/(6)\}W = 0.076W$$

where the values for I, C, and R_w are as in 28-1/UBC.

SOUTH WALL

COMPRESSION LOADING

$$\begin{aligned} P_u &= 1.4(D + L + E) = 1.4\{D + L + [1.0E_v + 0.4(E_{h1} + E_{h2})]\} \\ &= (1.4)\{84,090 + 16,080 + 0\} \\ &\quad + 1.4\{0 + (1.0)[(0.076)(84,090) + 0]\} \\ &= 149,200^\# < 495,600^\# \text{ (Axial load capacity)} \quad \therefore \text{OKAY} \end{aligned}$$

CHECK OVERTURN

The DL resisting overturning is reduced by the vertical seismic component. Therefore,

$$\begin{aligned} (DL)(1.000 - 0.078)(L/2) &= (1.542 \times 10^6)(0.922) = 1.422 \times 10^6 \\ SF &= (1.422 \times 10^6)/(260,580) = 5.5 \text{ (OK)} \end{aligned}$$

ANALYTICAL CALCULATIONS

Rev 0

Page A31 of _____

Subject 232 E BUILDING Seismic AssessmentOriginator J.A. DyaDate 09/29/94Checker Stacy WestDate 9/29/94COLUMN 'D' WALLCOMPRESSION LOADING

$$\begin{aligned}
 P_u &= 1.4(D + L + E) = 1.4(1.0E_v + 0.4(E_{h1} + E_{h2})) \\
 &= 1.4(130,320 + 20,790 + 0) \\
 &\quad + 1.4\{0 + 1.0[(0.076)(130,320) + 0]\} \\
 &= 225,400^\# < 782,300^\# \text{ (Axial load capacity)} \quad \therefore \text{OKAY}
 \end{aligned}$$

CHECK OVERTURN

$$\begin{aligned}
 (DL)(1.000 - 0.078)(L/2) &= (2.389 \times 10^6)(0.922) = 2.203 \times 10^6 \\
 SF &= (2.203 \times 10^6)/(412,860) = 5.3 \text{ (OK)}
 \end{aligned}$$

ANALYTICAL CALCULATIONS

WMC-SD-WM-DCR-002

REV 0

Page A32 of

Subject 232-Z BUILDING SEISMIC ASSESSMENT

Originator J. A. RYAN

Date 22 SEPTEMBER, 1994

Checker SRWongenblast

Date 9/29/94

DRAWINGS:

H-2-23102	Structural Foundation Plan and Details
H-2-23105	Architectural Details
H-2-23106	Architectural Elevations Sections and Details
H-2-23107	Architectural Doors and Finish Schedule and Details
H-2-23108	Architectural Details

REFERENCES:

HWS-6495, 1959	Specifications for Phase I Construction, 232-Z Building Plutonium Recovery from Contaminated Materials. Project CGC-813, May 12, 1959.
ASTM C 94-58, 1958	Specifications for Ready-Mixed Concrete.
ASTM C 90-52, 1952	Specification s for Hollow Loadbearing Concrete Masonry Units.
ASTM A 15-58T, 1958	Specifications for Billet-Steel Bars for Concrete Reinforcement.
ASTM A 185-58T, 1958	Specifications for Welded Steel Wire Fabric for Concrete Reinforcement.
UBC, 1994,	Volume 2, Uniform Building Code, UBC, International Conference of Building Officials, Whittier, California.
DOE-RL, 1994,	Standard Architectural-Civil Design Criteria, Design Loads for Facilities, Hanford Plant Standards, SDC-4.1, Rev. 12, U. S. Department of Energy, Richland Operations Office, Richland, Washington, 1994.
ACI, 1985,	Code Requirements for Nuclear Safety Related Concrete Structures, ACI-349, American Concrete Institute, Detroit, Michigan.
ACI, 1989,	Building Code Requirements for Reinforced Concrete, ACI-318, American Concrete Institute, Detroit, Michigan.
UCRL, 1990,	Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards, UCRL-15910, Livermore, California.

ANALYTICAL CALCULATIONS

REV D

Page A33 of

Subject 232-Z Building Seismic Assessment
 Originator John Marron Date 9/30/94
 Checker MRWason Date 9/30/94

CODE COMPLIANCE ASSESSMENT

232-Z MASONRY CONSTRUCTION

File: 232-WALL.WK1

30-Sep-94

UBC-94

REINFORCEMENT CONFIGURATION REQUIREMENTS

					ACTUAL	COMPLIANCE
2106.....	GENERAL DESIGN REQUIREMENTS					
2106.1.12.3	Reinforcement for Seismic Zone 2					
	(1) Column		..	n/a		
	(2) Vertical wall	As vert @ 4' c-to-c max	>	0.20 (si)	0.40	Yes
		As vert @ each corner	>	0.20 (si)	??	??
	(3) Horizontal wall	As horz @ openings t-&b	>	0.20 (si)	Some	?? Spec II.15.h
		As horz @ top of wall	>	0.20 (si)	In conc.	Yes
		As horz @ bot of wall	>	0.20 (si)	In conc.	Yes
		As horz @ 10' max spacing	>	0.20 (si)	Some	??
2106.3.5	Shear reinforcement	Anchorage at each end	..	Full dev.	Some	??
2106.3.6	Lateral ties	Column A2:	>	#2 @10"	None	No
2108.....	STRENGTH DESIGN REQUIREMENTS					
2108.2.2.3	Cover for reinforcement	All surfaces	>	1.50 (in)	??	??
2108.2.5.2	Shear wall reinforcement					
	(1) 2106.1.12.4 #2.3	As vert	>	0.0007 Ag = 0.27 (si)	0.40	Yes
		As horz	>	0.0007 Ag = 0.27 (si)	??	??
		As vert + As horz	>	0.002 Ag = 0.77 (si)	??	??
		As horz @ corners		Cont.	??	??
	(2) Flexural failure		..	n/a		
	(3) Reinf. ratio:	As vert / As horz	>	0.50	??	??
2108.2.6.2.9	Joint reinforcement		..	Req'd.	??	??

REINFORCEMENT CONFIGURATION ASSESSMENT SUMMARY

VERTICAL REINFORCEMENT Dwg H-2-23102 specifies 2-#4 bars @ 4'-0" c-to-c.

This satisfies all requirements for vertical steel, except (a) corner steel is not specifically called out and (b) embedment length of anchoring dowels is not specified.

HORIZONTAL REINFORCEMENT Dwg H-2-23102 specifies metal lath below the top course, and "Wall-Mesh" stock #6 or #8 for each second course.

Manufacturer's data for the area of steel for these items is expected by 10/15. Code compliance will be reassessed then.

Horizontal steel in a shear wall is considered nonstructural reinforcement (ref Tri Services Technical Manual chap 13)

COLUMN TIES

Column A2 is a composite column, with an 8"x8" concrete core surrounded by an 4" thick masonry pilaster block.

No lateral ties are specified. The 4" pilaster with "Wall-Mesh" every 2nd course provides some lateral support.

CONCLUSION

Reinforcing steel configuration is unknown as noted above, but most significant structural requirements appear to be met.

Calculated capacities indicate conservative design and ample margin of safety.

This reserve strength and the relatively low loads provide reasonable assurance of the building surviving a 3-over-1 seismic event.

If more assurance is required exploratory material samples of the as-built construction should be obtained.

ANALYTICAL CALCULATIONS

Rev 0

Page A34 of

Subject BUILDING 232-Z SEISMIC ASSESSMENTOriginator J.F. MARRON Date 9/22/94Checker JR Wagnall Date 9/23/94

File: 232-LOAD 22-Sep-94

BUILDING 232-Z INCINERATOR ROOM ROOF AND ROOF BEAM LOAD ANALYSIS

VERTICAL LOADS

		ROOF	ROOF BEAM	SEISMIC FACTORS
Dead	D	70 (psf)	1056 (plf)	I = 1.25
Live	L	20 (psf)	260 (plf)	C = 2.75
Seismic	E = (2/3)*0.2D*(IC/Rw) =	7 (psf)	97 (plf)	Rw = 5.00
Load Case #1:	1.4*(D + L) =	126 (psf)	1842 (plf)	
#2:	1.4*(D + L + E) =	136 (psf)	1978 (plf)	MAXIMUM LOADING
#3:	0.9D + 1.4E =	73 (psf)	1086 (plf)	

MOMENT DISTRIBUTION ANALYSIS..... ONE FT STRIP OF E-W ROOF Note: K = 4 EI/L with fixed end

Column Line Joint Assumption	1 Fixed	2 Fixed	3 Fixed	4 Fixed
Ctr to Ctr Span (l) (ft)	13.00	13.00	10.00	
Relative Stiffness (I/l)	0.077	0.000	0.077	0.100
Distribution Factors	0.000 0.500	0.000 0.500	0.000 0.435	0.565 0.000
Unit Fixed End Moments (ft-lb)	-14.1 14.1	-14.1 14.1	-8.3 8.3	
Distributed Moments (ft-lb)	0.0 0.0	0.0 -2.5	-3.3 0.0	
Carryover Moments (ft-lb)	0.0 0.0	-1.3 0.0	0.0 -1.6	
Distributed Moments (ft-lb)	0.0 0.6	0.6 0.0	0.0 0.0	
Carryover Moments (ft-lb)	0.3 0.0	0.0 0.3	0.0 0.0	
Distributed Moments (ft-lb)	0.0 0.0	0.0 -0.1	-0.2 0.0	
Final Unit End Moments (ft-lb)	-13.8 14.7	-14.7 11.8	-11.8 6.7	
Simple Beam Moments (ft-lb)	21.1	21.1	12.5	
Final Unit Mid Moments (ft-lb)	6.9	7.9	3.3	
FINAL CASE #2 MOMENTS				
END MOMENTS (in-kip)	-22 24	-24 19	-19 11	
MID-SPAN MOMENTS (in-kip)	11	13	5	

MOMENT DISTRIBUTION ANALYSIS..... 10"x18.5" N-S ROOF BEAM Note: K' = 0.75 K with pinned end

Column Line Joint Assumption	A Pinned	B Fixed	C Fixed	D Pinned
Ctr to Ctr Span (l) (ft)	14.33	13.72	14.33	13.94
Moment of Inertia (I) (ft ⁴)	0.254	0.069	0.254	0.069
Relative Stiffness (I/l)	0.013	0.005	0.018	0.005
Distribution Factors	1.000 0.369	0.140 0.491	0.495 0.139	0.367 1.000
Unit Fixed End Moments (ft-lb)	-17.1 17.1	-17.1 17.1	-17.5 17.5	
Distributed Moments (ft-lb)	17.1 0.0	0.0 0.2	0.1 0.1	-17.5 17.5
Carryover Moments (ft-lb)	0.0 8.6	0.1 0.0	-8.8 0.0	
Distributed Moments (ft-lb)	0.0 -3.2	-1.2 4.3	1.2 3.2	0.0 0.0
Carryover Moments (ft-lb)	0.0 0.0	2.2 -1.1	-2.1 1.0	0.0 0.0
Distributed Moments (ft-lb)	0.0 -0.8	-0.3 1.0	0.3 0.8	0.0 0.0
Final Unit End Moments (ft-lb)	0.0 21.7	-1.5 -20.2	20.5 1.6	-22.2 0.0
Final Unit End Shears (lb)	5.7 8.7	7.1 7.2	8.8 5.7	
M max @ V = 0 (ft)	5.66	7.14	8.78	
Final Unit Max Moment (ft-lb)	16.0	5.3	16.4	
FINAL CASE #2 REACTIONS				
VERTICAL SHEARS (kip)	11.2 17.2	14.1 14.2	17.4 11.3	
COLUMN LOADS (kip)	11.2	31.3	31.6	11.3
END MOMENTS (in-kip)	0 514	-36 -480	37 488	0
MID-SPAN MOMENTS (in-kip)	380	126	388	

ANALYTICAL CALCULATIONS

REV 0
Page 35 of

Subject BUILDING 232-Z SEISMIC ASSESSMENT

Originator J.F. MARRO Date 9/23/94

Checker HRWagell et Date 9/23/94

PROCEDURE FOR CODE COMPLIANCE AND CAPACITIES DETERMINATION

23-Sep-94

File: 232-BEAM.WK1

232-Z ROOF BEAM DESCRIPTION

CONFIGURATION INPUT DATA

BASIS

Concrete.....	f'c	3,000	(psi)
Weight.....	W	145	(pcf)
Rectangular.....	b	10.00	(in)
.....	h	18.50	(in)
T-beam.....	bw (sec 8.10)	n/a	(in)
.....	h'	n/a	(in)
Span.....	l c-to-c (sec 8.9)	172.00	(in)
.....	l' clear	162.00	(in)
Lateral support....	l ^m	Deck	(in)
Topping.....	Neglect (sec 8.12)	4.50	(in)
Embedments.....	Consider (sec 6.3)	None	
Reinforcement.....	fy	40,000	(psi)
Tension Steel (same top & bottom)	#8 & 2-#6	(1/8")	
Spacing.....	s1	Center	(in)
Cover.....	c1	0.75	(in)
.....	dc1	1.13	(in)
Area.....	Ast1	1.67	(in ²)
Bottom steel (over columns)		2-#6	
Lateral steel		None	
Shear steel		2-#3	

Dwg H-2-23102

UBC-91	ACI-349-85	ACI-318-89	CONFIGURATION COMPLIANCE REQUIREMENTS		COMPLY?	
		8.5.1	Concrete.....	$E_c = 57*(f'c)^{0.5}$	3122 (ksi)	
		10.2.7.1		 $f_c \text{ max} = 0.85*f'c$	2550 (psi)	
		9.5.2.3		 $f_r = 7.5*(f'c)^{0.5}$	411 (psi)	
		10.2.5		 $f_c \text{ ten} =$	0 (psi)	
		10.2.3		 $e_c \text{ alw} = e_{ca} =$	0.003	
		10.2.7.3		 $B1 @ f'c < 4 \text{ ksi} =$	0.85	
		9.5.2.3	Deflection.....	$I_g = b^3h^3/12$	5276 (in ⁴)	
				 $y_t = h/2$	9 (in)	
		9.5.1	Thickness.....	$h \text{ min} > l/k \text{ (tab 9.5)}$	8 (in)	OK 30+ years
		10.7		 $h \text{ deep} > (4/5)*l'$	n/a (in)	
		15.7		 $h \text{ footing} > 6" + s1$	n/a (in)	
		10.4.1	Lateral support....	$l^m < 50*b$	n/a (in)	
		8.5.2	Reinforcement.....	E_s	29,000 (ksi)	
		9.4		 $f_y < 80,000$	40,000 (psi)	
		10.2.4		 $e_y = f_y/E_s$	0.00138	
			Tension Steel (at top)			
		10.6.3	Spacing.....	Well distributed		YES
			..A =	$2*dc*b/(\text{number of bars})$	8 (si)	
		10.6.4	..z =	$0.6*f_y*(dc*A)^{0.33} < 145$	52 (kpi)	YES
		7.7.1	Cover.....	c1 req'd =	0.75 (in)	YES
			Depth.....	d = h-dc-topping	12.88 (in)	
			Proportion....	pt = Ast/bd	0.013	
			Balanced Steel			
			Depth to NA....	xb = d*eca/(eca+ey)	8.82 (in)	
		10.3.2	Proportion....	pb = (fc/fy)(B1(xb/d))	0.037	
		10.3.3		...pmax = 0.75 pb > pt	0.028	YES > pt
		10.5.1		...pmin = 200/fy < pt	0.005	YES < pt
		21.3.2.1	Continuous Steel....	2 min top & bottom	None	NO continuous
		7.12.1	Lateral Steel.....	Not req'd for beams		

ULTIMATE STRENGTH CAPACITIES DETERMINATION

Case A: Tension in top

	Forces.....	T = Ast*fy	67	(k)	
		Cc = T	67	(k)	
10.2.7(g)	Depth of fc.....	a = Cc/(fc*b)	2.62	(in)	
	to NA.....	x = a/B1	3.08	(in)	
	Strains.....	ec max = eca	0.00300		
		est max = eca*(d-x)/x	0.00395		
	Normal moment.....	Mn = Cc*(d-0.5a)	773	(in-k)	
9.3.2	Stren red factor... Rho =		0.90		
	Capacity moment....	Mu = Rho*Mn = 0.90*Mn	695	(in-k)	YES Mu > 526
9.5.2.3	Cracking moment....	Mcr = fr*ig/yt	234	(in-k)	

ANALYTICAL CALCULATIONS

WAC-SD-WM-DCR-002

REV 0

Page A36 of A36

Subject BUILDING 232-Z SEISMIC ASSESSMENTOriginator J.F. MARRON Date 9/23/94Checker MR Wagellast Date 9/23/94

PROCEDURE FOR CODE COMPLIANCE AND CAPACITIES DETERMINATION

23-Sep-94

File: 232-ROOF.WK1

232-Z ROOF DESCRIPTION

CONFIGURATION INPUT DATA

Concrete.....f'c	3,000	(psi)
Weight.....W	145	(pcf)
Rectangular.....b	12.00	(in)
.....h	3.63	(in)
T-beam.....bw (sec 8.10)	n/a	(in)
.....h'	n/a	(in)
Span.....l c-to-c (sec 8.9)	156.00	(in)
.....l' clear	147.00	(in)
Lateral support...l"	Slab	(in)
Topping.....Neglect (sec 8.12)	Neg.	(in)
Embedments.....Consider (sec 6.3)	None	
Reinforcement.....fy	40,000	(psi)
Tension Steel (at top)	#5	(1/8")
Spacing.....s1	8.00	(in)
Cover.....c1	0.75	(in)
.....dc1	1.06	(in)
Area.....Ast1	0.47	(si)
Bottom steel	Deck	
Lateral steel	None	
Shear steel	None	

BASIS

Dwg H-2-23102

UBC-91	ACI-349-85	ACI-318-89	CONFIGURATION COMPLIANCE REQUIREMENTS		COMPLY?
		8.5.1	Concrete.....Ec = $57 \cdot (f'c)^{0.5}$	3122	(ksi)
		10.2.7.1	fc max = $0.85 \cdot f'c$	2550	(psi)
		9.5.2.3	fr = $7.5 \cdot (f'c)^{0.5}$	411	(psi)
		10.2.5	fc ten =	0	(psi)
		10.2.3	ec alw = eca =	0.003	
		10.2.7.3	B1 @ $f'c < 4$ ksi =	0.85	
		9.5.2.3	DeflectionIg = $b \cdot h^3 / 12$	48	(in ⁴)
			yt = $h/2$	2	(in)
		9.5.1	Thickness.....h min > l/k (tab 9.5)	6	(in)
		10.7	h deep > $(4/5) \cdot l'$	n/a	(in)
		15.7	h footing > $6" + s1$	n/a	(in)
		10.4.1	Lateral support...l" < $50 \cdot b$	n/a	(in)
		8.5.2	Reinforcement.....Es	29,000	(ksi)
		9.4	fy < 80,000	40,000	(psi)
		10.2.4	ey = fy/Es	0.00138	
			Tension Steel (at top)		
		10.6.3	Spacing.....Well distributed		YES
			..A = $2 \cdot dc \cdot b / (\text{number of bars})$	17	(si)
		10.6.4	..z = $0.6 \cdot fy \cdot (dc \cdot A)^{0.33} < 145$	65	(kpi)
		7.7.1	Cover.....c1 req'd =	0.75/1.50	(in)
			Depth.....d = $h - dc$	2.56	(in)
			Proportion....pt = Ast/bd	0.015	
			Balanced Steel		
			Depth to NA....xb = $d \cdot eca / (eca + ey)$	1.76	(in)
		10.3.2	Proportion....pb = $(fc/fy) \cdot [B1(xb/d)]$	0.037	
		10.3.3	...pmax = $0.75 \cdot pb > pt$	0.028	YES > pt
		10.5.1	...pmin = $200/fy < pt$	0.005	YES < pt
		21.3.2.1	Continuous Steel....2 min top & bottom	n/a	YES frames only
		7.12.1	Lateral Steel.....Not req'd for beams		

ULTIMATE STRENGTH CAPACITIES DETERMINATION

	Case A: Tension in top			
	Forces.....T = $Ast \cdot fy$	19	(k)	
	Cc = T	19	(k)	
10.2.7(g)	Depth of fc.....a = $Cc / (fc \cdot b)$	0.61	(in)	
	to NA.....x = $a/B1$	0.72	(in)	
	Strains.....ec max = eca	0.00300		
	est max = $eca \cdot (d-x)/x$	0.00339		
	Normal moment.....Mn = $Cc \cdot (d - 0.5a)$	42	(in-k)	
9.3.2	Stren red factor... Rho =	0.90		
	Capacity moment...Mu = $Rho \cdot Mn = 0.90 \cdot Mn$	38	(in-k)	YES Mu > 24
9.5.2.3	Cracking moment...Mcr = $fr \cdot Ig / yt$	11	(in-k)	

REVIEW COMMENT RECORD (RCR)	1. Date October 18, 1994	2. Review No. 1231-94
	3. Project No. 232Z-PFP	4. Page 1 of 1

5. Document Number(s)/Title(s) WHC-SD-DCR-002 Rev.0/ 232Z Building Seismic Assessment	6. Program/Project/ Building Number 232Z - PFP	7. Reviewer Omar Jaka	8. Organization/Group TWRS Nuclear Safety	9. Location/Phone 2751E/B101/2-2322
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17. Comment Submittal Approval:

10. Agreement with indicated comment disposition(s)

11. CLOSED

Organization Manager (Optional)

Date 10/21/94

m. D. on Taken
Reviewer/Point of Contact

M. Wagoner
Author/Originator

Date 10/21/94

m. D. on Taken
Reviewer/Point of Contact

M. Wagoner
Author/Originator

12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)	16. Status
1.	What is the justification for not including the snow load in the mass of the structure for the seismic dynamic response ?	OJ	The UBC 94 section 1628.1.3 specifies that snow loads of 30 psf or less need not be included in the seismic dead load. SDC 4.1 rev 12 specifies ground snow loads of 15 psf and minimum roof snow loads of 20 psf for the Hanford site. Therefore, the mass of the snow was not included in the seismic calculations. The snow was included in the dead weight and live load calculations.	Closed 10/25/94 skw
2.	How did you determine the yield strength of reinforcing steel for ASTM A15-58 steel ?	OJ	Drawing H-2-23102 specified in note 2 that the reinforcing bars are ASTM A15-58T. HWS-6495, Specifications for Phase 1 Construction, 232-Z Building, division 2, section 4.b specifies that the reinforcing bars are intermediate grade deformed bars conforming to ASTM A15-58T. A copy of the 1958 edition of the ASTM A15-58T was obtained in which the intermediate bars have a yield point of 40,000 psi and a tensile strength of 70,000 to 90,000 psi.	Closed 10/25/94 skw

REVIEW COMMENT RECORD (RCR)		1. Date October 18, 1994		2. Review No. 1231-94	
		3. Project No. 232Z-PFP		4. Page 2 of 1	
12. Item	13. Comment(s)/Discrepancy(s) (Provide technical justification for the comment and detailed recommendation of the action required to correct/resolve the discrepancy/problem indicated.)	14. Hold Point	15. Disposition (Provide justification if NOT accepted.)		16. Status
3.	Safety class 1 Gloveboxes are in Safety class 3 building. What kind of protection is provided from structure falling on them and releasing the radiation to atmosphere.	0J	The evaluation showing conformance to Uniform Building Code requirements ensures that the building will not collapse.		<i>closed</i> <i>10/25/94</i> <i>AKW</i>
4.	Is there any plan to run some test to determine the strength of structure beside the visual inspection ?	0J	No. In-place strength of materials is likely to be greater than the minimum specified design strength unless deteriorated by corrosion. No evidence of corrosion was found during the field inspection. The use of minimum specified strengths of materials is considered conservative.		<i>closed</i> <i>10/25/94</i> <i>AKW</i>

WHL-SD-WM-DCR-002
 Rev 0
 B2 of 2