ArevaEPRDCPEm Resource

From: Pederson Ronda M (AREVA NP INC) [Ronda.Pederson@areva.com]
Sent: Friday, May 29, 2009 9:49 PM
To: Getachew Tesfaye
Cc: BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC); VAN NOY Mark (EXT)
Subject: Response to U.S. EPR Design Certification Application RAI No. 155, Supplement 3
Attachments: RAI 155 Supplement 3 Response US EPR DC.pdf

Getachew,

AREVA NP Inc. (AREVA NP) provided responses to 5 of the 78 questions of RAI No. 155 on February 13, 2009. AREVA NP submitted Supplement 1 to the response on March 31, 2009, to address 20 of the remaining questions. AREVA NP submitted Supplement 2 to the response on April 30, 2009, to address 9 of the remaining questions. The attached file, “RAI 155 Supplement 3 Response US EPR DC.pdf” provides technically correct and complete responses to 20 of the remaining 44 questions, as committed.

Appended to this file are affected pages of the U.S. EPR Final Safety Analysis Report in redline-strikeout format which support the response to RAI 155 Questions 03.08.01-8, 03.08.01-10, 03.08.01-12, 03.08.03-3, 03.08.03-6, 03.08.03-10, 03.08.04-3, and 03.08.05-6.

The following table indicates the respective pages in the response document, “RAI 155 Supplement 3 Response US EPR DC.pdf” that contain AREVA NP’s response to the subject questions.

Question #	Start Page	End Page
RAI 155 — 03.08.01-8	2	9
RAI 155 — 03.08.01-9	10	10
RAI 155 — 03.08.01-10	11	17
RAI 155 — 03.08.01-12	18	19
RAI 155 — 03.08.01-16	20	21
RAI 155 — 03.08.01-22	22	24
RAI 155 — 03.08.01-27	25	26
RAI 155 — 03.08.02-5	27	27
RAI 155 — 03.08.02-6	28	31
RAI 155 — 03.08.02-10	32	32
RAI 155 — 03.08.03-3	33	35
RAI 155 — 03.08.03-6	36	37
RAI 155 — 03.08.03-10	38	38
RAI 155 — 03.08.03-11	39	40
RAI 155 — 03.08.03-12	41	41
RAI 155 — 03.08.04-3	42	45
RAI 155 — 03.08.04-4	46	47
RAI 155 — 03.08.04-5	48	48
RAI 155 — 03.08.05-2	49	50
RAI 155 — 03.08.05-6	51	52

The schedule for technically correct and complete responses to the remaining 24 questions is unchanged and provided below:

Question RAI 155 #	Response Date
RAI 155 — 03.08.01-03	October 30, 2009
RAI 155 — 03.08.01-06	October 30, 2009

RAI 155 — 03.08.01-11	June 30, 2009
RAI 155 — 03.08.01-20	October 30, 2009
RAI 155 — 03.08.01-24	October 30, 2009
RAI 155 — 03.08.02-01	June 30, 2009
RAI 155 — 03.08.02-02	July 31, 2009
RAI 155 — 03.08.02-04	June 30, 2009
RAI 155 — 03.08.02-07	July 31, 2009
RAI 155 — 03.08.02-08	July 31, 2009
RAI 155 — 03.08.03-04	July 31, 2009
RAI 155 — 03.08.03-16	July 31, 2009
RAI 155 — 03.08.03-17	July 31, 2009
RAI 155 — 03.08.04-06	October 30, 2009
RAI 155 — 03.08.05-01	July 31, 2009
RAI 155 — 03.08.05-07	June 30, 2009
RAI 155 — 03.08.05-08	July 31, 2009
RAI 155 — 03.08.05-10	July 31, 2009
RAI 155 — 03.08.05-12	July 31, 2009
RAI 155 — 03.08.05-13	June 30, 2009
RAI 155 — 03.08.05-14	June 30, 2009
RAI 155 — 03.08.05-15	June 30, 2009
RAI 155 — 03.08.05-16	June 30, 2009
RAI 155 — 03.08.05-18	June 30, 2009

Sincerely,

Ronda Pederson
ronda.pederson@areva.com
Licensing Manager, U.S. EPR Design Certification
AREVA NP Inc.
An AREVA and Siemens company
3315 Old Forest Road
Lynchburg, VA 24506-0935
Phone: 434-832-3694
Cell: 434-841-8788

From: Pederson Ronda M (AREVA NP INC)
Sent: Thursday, April 30, 2009 9:16 PM
To: Getachew Tesfaye (gxt2@nrc.gov)
Cc: BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC); VAN NOY Mark (EXT)
Subject: Response to U.S. EPR Design Certification Application RAI No. 155, Supplement 2 (part 4 of 4)

Getachew,

Response file, "RAI 155 Supplement 2 Response US EPR DC (Part 4 of 4).pdf" is attached.

Sincerely,

Ronda Pederson

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Licensing Manager, U.S. EPR Design Certification
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An AREVA and Siemens company
3315 Old Forest Road
Lynchburg, VA 24506-0935
Phone: 434-832-3694
Cell: 434-841-8788

From: Pederson Ronda M (AREVA NP INC)
Sent: Thursday, April 30, 2009 9:12 PM
To: Getachew Tesfaye (gxt2@nrc.gov)
Cc: BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC); VAN NOY Mark (EXT)
Subject: Response to U.S. EPR Design Certification Application RAI No. 155, Supplement 2 (part 3 of 4)

Getachew,

Response file, "RAI 155 Supplement 2 Response US EPR DC (Part 3 of 4).pdf" is attached.

Sincerely,

Ronda Pederson

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An AREVA and Siemens company
3315 Old Forest Road
Lynchburg, VA 24506-0935
Phone: 434-832-3694
Cell: 434-841-8788

From: Pederson Ronda M (AREVA NP INC)
Sent: Thursday, April 30, 2009 9:11 PM
To: Getachew Tesfaye (gxt2@nrc.gov)
Cc: BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC); VAN NOY Mark (EXT)
Subject: Response to U.S. EPR Design Certification Application RAI No. 155, Supplement 2 (part 2 of 4)

Getachew,

Response file, "RAI 155 Supplement 2 Response US EPR DC (Part 2 of 4).pdf" is attached.

Sincerely,

Ronda Pederson

ronda.pederson@areva.com
Licensing Manager, U.S. EPR Design Certification
AREVA NP Inc.

An AREVA and Siemens company
3315 Old Forest Road
Lynchburg, VA 24506-0935
Phone: 434-832-3694
Cell: 434-841-8788

From: Pederson Ronda M (AREVA NP INC)
Sent: Thursday, April 30, 2009 9:09 PM
To: Getachew Tesfaye (gxt2@nrc.gov)
Cc: BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC); VAN NOY Mark (EXT)
Subject: Response to U.S. EPR Design Certification Application RAI No. 155, Supplement 2 (part 1 of 4)

Getachew,

AREVA NP Inc. (AREVA NP) provided responses to 5 of the 78 questions of RAI No. 155 on February 13, 2009. AREVA NP submitted Supplement 1 to the response on March 31, 2009 to address 20 of the remaining questions. The response document, “RAI 155 Supplement 2 Response U.S. EPR DC” provides technically correct and complete responses to 9 of the remaining 53 questions, as committed.

Due to transmittal size limitations, the response file has been separated to e-mailthe response in four parts. Attached is "RAI 155 Supplement 2 Response U.S. EPR DC (Part 1 of 4).pdf."

Appended to this file are affected pages of the U.S. EPR Final Safety Analysis Report in redline-strikeout format which support the response to RAI 155 Questions 03.08.01-07, 03.08.02-03, 03.08.03-05, 03.08.03-14 and 03.08.03-15.

The following table indicates the respective pages in the response document, "RAI 155 Supplement 2 Response U.S. EPR DC,” that contain AREVA NP’s response to the subject questions.

Question #	Start Page	End Page
RAI 155 — 03.08.01-07	2	5
RAI 155 — 03.08.01-17	6	6
RAI 155 — 03.08.02-03	7	7
RAI 155 — 03.08.03-05	8	15
RAI 155 — 03.08.03-14	16	16
RAI 155 — 03.08.03-15	17	37
RAI 155 — 03.08.04-02	38	38
RAI 155 — 03.08.05-05	39	42
RAI 155 — 03.08.05-11	43	43
RAI 155 — 03.08.05-12	44	44

AREVA NP's response to RAI 155 Question 03.08.05-12 has been deferred to July 31, 2009 to be provided concurrently with the response to a similar question regarding the Nuclear Island common structure. With this exception, the schedule for technically correct and complete responses to the remaining 44 questions is unchanged and is provided below:

Question RAI 155 #	Response Date
RAI 155 — 03.08.01-03	October 30, 2009
RAI 155 — 03.08.01-06	October 30, 2009
RAI 155 — 03.08.01-08	May 29, 2009
RAI 155 — 03.08.01-09	May 29, 2009

RAI 155 — 03.08.01-10	May 29, 2009
RAI 155 — 03.08.01-11	June 30, 2009
RAI 155 — 03.08.01-12	May 29, 2009
RAI 155 — 03.08.01-16	May 29, 2009
RAI 155 — 03.08.01-20	October 30, 2009
RAI 155 — 03.08.01-22	May 29, 2009
RAI 155 — 03.08.01-24	October 30, 2009
RAI 155 — 03.08.01-27	May 29, 2009
RAI 155 — 03.08.02-01	June 30, 2009
RAI 155 — 03.08.02-02	July 31, 2009
RAI 155 — 03.08.02-04	June 30, 2009
RAI 155 — 03.08.02-05	May 29, 2009
RAI 155 — 03.08.02-06	May 29, 2009
RAI 155 — 03.08.02-07	July 31, 2009
RAI 155 — 03.08.02-08	July 31, 2009
RAI 155 — 03.08.02-10	May 29, 2009
RAI 155 — 03.08.03-03	May 29, 2009
RAI 155 — 03.08.03-04	July 31, 2009
RAI 155 — 03.08.03-06	May 29, 2009
RAI 155 — 03.08.03-10	May 29, 2009
RAI 155 — 03.08.03-11	May 29, 2009
RAI 155 — 03.08.03-12	May 29, 2009
RAI 155 — 03.08.03-16	July 31, 2009
RAI 155 — 03.08.03-17	July 31, 2009
RAI 155 — 03.08.04-03	May 29, 2009
RAI 155 — 03.08.04-04	May 29, 2009
RAI 155 — 03.08.04-05	May 29, 2009
RAI 155 — 03.08.04-06	October 30, 2009
RAI 155 — 03.08.05-01	July 31, 2009
RAI 155 — 03.08.05-02	May 29, 2009
RAI 155 — 03.08.05-06	May 29, 2009
RAI 155 — 03.08.05-07	June 30, 2009
RAI 155 — 03.08.05-08	July 31, 2009
RAI 155 — 03.08.05-10	July 31, 2009
RAI 155 — 03.08.05-12	July 31, 2009
RAI 155 — 03.08.05-13	June 30, 2009
RAI 155 — 03.08.05-14	June 30, 2009
RAI 155 — 03.08.05-15	June 30, 2009
RAI 155 — 03.08.05-16	June 30, 2009
RAI 155 — 03.08.05-18	June 30, 2009

Sincerely,

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Cell: 434-841-8788

From: Pederson Ronda M (AREVA NP INC)
Sent: Tuesday, March 31, 2009 8:16 PM
To: Getachew Tesfaye
Cc: BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC); VAN NOY Mark (EXT); HEDRICK Gary E (AFS)
Subject: Response to U.S. EPR Design Certification Application RAI No. 155, Supplement 1

Getachew,

AREVA NP Inc. (AREVA NP) provided responses to 5 of the 78 questions of RAI No. 155 on February 13, 2009. The attached file, "RAI 155 Supplement 1 Response U.S. EPR DC" provides technically correct and complete responses to 20 of the remaining 73 questions, as committed.

Appended to this file are affected pages of the U.S. EPR Final Safety Analysis Report in redline-strikeout format which support the response to RAI 155 Supplement 1 Questions 03.08.01-04, 03.08.01-05, 03.08.01-21, 03.08.02-09, 03.08.03-02, 03.08.03-09, 03.08.05-03, and 03.08.05-04.

The following table indicates the respective page(s) in the response document, "RAI 155 Supplement 1 Response U.S. EPR DC," that contain AREVA NP's response to the subject questions.

Question #	Start Page	End Page
RAI 155 — 03.08.01-01	2	2
RAI 155 — 03.08.01-02	3	9
RAI 155 — 03.08.01-04	10	12
RAI 155 — 03.08.01-05	13	16
RAI 155 — 03.08.01-13	17	19
RAI 155 — 03.08.01-21	20	20
RAI 155 — 03.08.01-23	21	21
RAI 155 — 03.08.01-25	22	22
RAI 155 — 03.08.02-09	23	23
RAI 155 — 03.08.03-01	24	31
RAI 155 — 03.08.03-02	32	33
RAI 155 — 03.08.03-07	34	34
RAI 155 — 03.08.03-08	35	36
RAI 155 — 03.08.03-09	37	37
RAI 155 — 03.08.03-13	38	38
RAI 155 — 03.08.04-01	39	40
RAI 155 — 03.08.05-03	41	41
RAI 155 — 03.08.05-04	42	46
RAI 155 — 03.08.05-09	47	48
RAI 155 — 03.08.05-17	49	53

The schedule for technically correct and complete responses to the remaining 53 questions is unchanged and provided below:

Question RAI 155 #	Response Date
RAI 155 — 03.08.01-03	October 30, 2009
RAI 155 — 03.08.01-06	October 30, 2009
RAI 155 — 03.08.01-07	April 30, 2009
RAI 155 — 03.08.01-08	May 29, 2009
RAI 155 — 03.08.01-09	May 29, 2009
RAI 155 — 03.08.01-10	May 29, 2009
RAI 155 — 03.08.01-11	June 30, 2009
RAI 155 — 03.08.01-12	May 29, 2009
RAI 155 — 03.08.01-16	May 29, 2009
RAI 155 — 03.08.01-17	April 30, 2009
RAI 155 — 03.08.01-20	October 30, 2009
RAI 155 — 03.08.01-22	May 29, 2009
RAI 155 — 03.08.01-24	October 30, 2009
RAI 155 — 03.08.01-27	May 29, 2009
RAI 155 — 03.08.02-01	June 30, 2009
RAI 155 — 03.08.02-02	July 31, 2009
RAI 155 — 03.08.02-03	April 30, 2009
RAI 155 — 03.08.02-04	June 30, 2009
RAI 155 — 03.08.02-05	May 29, 2009
RAI 155 — 03.08.02-06	May 29, 2009
RAI 155 — 03.08.02-07	July 31, 2009
RAI 155 — 03.08.02-08	July 31, 2009
RAI 155 — 03.08.02-10	May 29, 2009
RAI 155 — 03.08.03-03	May 29, 2009
RAI 155 — 03.08.03-04	July 31, 2009
RAI 155 — 03.08.03-05	April 30, 2009
RAI 155 — 03.08.03-06	May 29, 2009
RAI 155 — 03.08.03-10	May 29, 2009
RAI 155 — 03.08.03-11	May 29, 2009
RAI 155 — 03.08.03-12	May 29, 2009
RAI 155 — 03.08.03-14	April 30, 2009
RAI 155 — 03.08.03-15	April 30, 2009
RAI 155 — 03.08.03-16	July 31, 2009
RAI 155 — 03.08.03-17	July 31, 2009
RAI 155 — 03.08.04-02	April 30, 2009
RAI 155 — 03.08.04-03	May 29, 2009
RAI 155 — 03.08.04-04	May 29, 2009
RAI 155 — 03.08.04-05	May 29, 2009
RAI 155 — 03.08.04-06	October 30, 2009
RAI 155 — 03.08.05-01	July 31, 2009
RAI 155 — 03.08.05-02	May 29, 2009

RAI 155 — 03.08.05-05	April 30, 2009
RAI 155 — 03.08.05-06	May 29, 2009
RAI 155 — 03.08.05-07	June 30, 2009
RAI 155 — 03.08.05-08	July 31, 2009
RAI 155 — 03.08.05-10	July 31, 2009
RAI 155 — 03.08.05-11	April 30, 2009
RAI 155 — 03.08.05-12	April 30, 2009
RAI 155 — 03.08.05-13	June 30, 2009
RAI 155 — 03.08.05-14	June 30, 2009
RAI 155 — 03.08.05-15	June 30, 2009
RAI 155 — 03.08.05-16	June 30, 2009
RAI 155 — 03.08.05-18	June 30, 2009

Sincerely,

Ronda Pederson

ronda.pederson@areva.com
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AREVA NP Inc.
An AREVA and Siemens company
3315 Old Forest Road
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Phone: 434-832-3694
Cell: 434-841-8788

From: Pederson Ronda M (AREVA NP INC)
Sent: Friday, February 13, 2009 7:18 PM
To: 'Getachew Tesfaye'
Cc: BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC); VAN NOY Mark (EXT); HARRIS Carolyn A (AREVA NP INC)
Subject: Response to U.S. EPR Design Certification Application RAI No. 155, FSAR Ch. 3

Getachew,
Attached please find AREVA NP Inc.'s (AREVA NP) response to the subject request for additional information (RAI). The attached file, "RAI 155 Response US EPR DC.pdf" provides technically correct and complete responses to 5 of the 78 questions.

Appended to this file are affected pages of the U.S. EPR Final Safety Analysis Report in redline-strikeout format which support the responses to RAI 155 Questions 03.08.01-15, 03.08.01-18, 03.08.01-19, and 03.08.01-26.

The following table indicates the respective pages in the response document, "RAI 155 Response US EPR DC.pdf," that contain AREVA NP's response to the subject questions.

Question #	Start Page	End Page
RAI 155 — 03.08.01-01	2	2
RAI 155 — 03.08.01-02	3	3
RAI 155 — 03.08.01-03	4	4
RAI 155 — 03.08.01-04	5	5
RAI 155 — 03.08.01-05	6	6

RAI 155 — 03.08.01-06	7	7
RAI 155 — 03.08.01-07	8	8
RAI 155 — 03.08.01-08	9	9
RAI 155 — 03.08.01-09	10	10
RAI 155 — 03.08.01-10	11	11
RAI 155 — 03.08.01-11	12	12
RAI 155 — 03.08.01-12	13	13
RAI 155 — 03.08.01-13	14	14
RAI 155 — 03.08.01-14	15	17
RAI 155 — 03.08.01-15	18	19
RAI 155 — 03.08.01-16	20	20
RAI 155 — 03.08.01-17	21	21
RAI 155 — 03.08.01-18	22	22
RAI 155 — 03.08.01-19	23	24
RAI 155 — 03.08.01-20	25	25
RAI 155 — 03.08.01-21	26	26
RAI 155 — 03.08.01-22	27	27
RAI 155 — 03.08.01-23	28	28
RAI 155 — 03.08.01-24	29	30
RAI 155 — 03.08.01-25	31	31
RAI 155 — 03.08.01-26	32	34
RAI 155 — 03.08.01-27	35	35
RAI 155 — 03.08.02-01	36	36
RAI 155 — 03.08.02-02	37	37
RAI 155 — 03.08.02-03	38	38
RAI 155 — 03.08.02-04	39	39
RAI 155 — 03.08.02-05	40	40
RAI 155 — 03.08.02-06	41	41
RAI 155 — 03.08.02-07	42	42
RAI 155 — 03.08.02-08	43	43
RAI 155 — 03.08.02-09	44	44
RAI 155 — 03.08.02-10	45	45
RAI 155 — 03.08.03-01	46	46
RAI 155 — 03.08.03-02	47	47
RAI 155 — 03.08.03-03	48	48
RAI 155 — 03.08.03-04	49	49
RAI 155 — 03.08.03-05	50	50
RAI 155 — 03.08.03-06	51	51
RAI 155 — 03.08.03-07	52	52
RAI 155 — 03.08.03-08	53	53
RAI 155 — 03.08.03-09	54	54
RAI 155 — 03.08.03-10	55	55
RAI 155 — 03.08.03-11	56	56
RAI 155 — 03.08.03-12	57	57

RAI 155 — 03.08.03-13	58	58
RAI 155 — 03.08.03-14	59	59
RAI 155 — 03.08.03-15	60	60
RAI 155 — 03.08.03-16	61	61
RAI 155 — 03.08.03-17	62	63
RAI 155 — 03.08.04-01	64	64
RAI 155 — 03.08.04-02	65	65
RAI 155 — 03.08.04-03	66	67
RAI 155 — 03.08.04-04	68	68
RAI 155 — 03.08.04-05	69	69
RAI 155 — 03.08.04-06	70	70
RAI 155 — 03.08.05-01	71	71
RAI 155 — 03.08.05-02	72	72
RAI 155 — 03.08.05-03	73	73
RAI 155 — 03.08.05-04	74	75
RAI 155 — 03.08.05-05	76	76
RAI 155 — 03.08.05-06	77	77
RAI 155 — 03.08.05-07	78	78
RAI 155 — 03.08.05-08	79	80
RAI 155 — 03.08.05-09	81	81
RAI 155 — 03.08.05-10	82	82
RAI 155 — 03.08.05-11	83	83
RAI 155 — 03.08.05-12	84	84
RAI 155 — 03.08.05-13	85	85
RAI 155 — 03.08.05-14	86	86
RAI 155 — 03.08.05-15	87	87
RAI 155 — 03.08.05-16	88	88
RAI 155 — 03.08.05-17	89	89
RAI 155 — 03.08.05-18	90	90

A complete answer is not provided for 73 of the 78 questions. The schedule for a technically correct and complete response to these questions is provided below.

Question #	**Response Date**
RAI 155 — 03.08.01-01	March 31, 2009
RAI 155 — 03.08.01-02	March 31, 2009
RAI 155 — 03.08.01-03	October 30, 2009
RAI 155 — 03.08.01-04	March 31, 2009
RAI 155 — 03.08.01-05	March 31, 2009
RAI 155 — 03.08.01-06	October 30, 2009
RAI 155 — 03.08.01-07	April 30, 2009
RAI 155 — 03.08.01-08	May 29, 2009
RAI 155 — 03.08.01-09	May 29, 2009

RAI 155 — 03.08.01-10	May 29, 2009
RAI 155 — 03.08.01-11	June 30, 2009
RAI 155 — 03.08.01-12	May 29, 2009
RAI 155 — 03.08.01-13	March 31, 2009
RAI 155 — 03.08.01-16	May 29, 2009
RAI 155 — 03.08.01-17	April 30, 2009
RAI 155 — 03.08.01-20	October 30, 2009
RAI 155 — 03.08.01-21	March 31, 2009
RAI 155 — 03.08.01-22	May 29, 2009
RAI 155 — 03.08.01-23	March 31, 2009
RAI 155 — 03.08.01-24	October 30, 2009
RAI 155 — 03.08.01-25	March 31, 2009
RAI 155 — 03.08.01-27	May 29, 2009
RAI 155 — 03.08.02-01	June 30, 2009
RAI 155 — 03.08.02-02	July 31, 2009
RAI 155 — 03.08.02-03	April 30, 2009
RAI 155 — 03.08.02-04	June 30, 2009
RAI 155 — 03.08.02-05	May 29, 2009
RAI 155 — 03.08.02-06	May 29, 2009
RAI 155 — 03.08.02-07	July 31, 2009
RAI 155 — 03.08.02-08	July 31, 2009
RAI 155 — 03.08.02-09	March 31, 2009
RAI 155 — 03.08.02-10	May 29, 2009
RAI 155 — 03.08.03-01	March 31, 2009
RAI 155 — 03.08.03-02	March 31, 2009
RAI 155 — 03.08.03-03	May 29, 2009
RAI 155 — 03.08.03-04	July 31, 2009
RAI 155 — 03.08.03-05	April 30, 2009
RAI 155 — 03.08.03-06	May 29, 2009
RAI 155 — 03.08.03-07	March 31, 2009
RAI 155 — 03.08.03-08	March 31, 2009
RAI 155 — 03.08.03-09	March 31, 2009
RAI 155 — 03.08.03-10	May 29, 2009
RAI 155 — 03.08.03-11	May 29, 2009
RAI 155 — 03.08.03-12	May 29, 2009
RAI 155 — 03.08.03-13	March 31, 2009
RAI 155 — 03.08.03-14	April 30, 2009
RAI 155 — 03.08.03-15	April 30, 2009
RAI 155 — 03.08.03-16	July 31, 2009
RAI 155 — 03.08.03-17	July 31, 2009
RAI 155 — 03.08.04-01	March 31, 2009
RAI 155 — 03.08.04-02	April 30, 2009
RAI 155 — 03.08.04-03	May 29, 2009
RAI 155 — 03.08.04-04	May 29, 2009

RAI 155 — 03.08.04-05	May 29, 2009
RAI 155 — 03.08.04-06	October 30, 2009
RAI 155 — 03.08.05-01	July 31, 2009
RAI 155 — 03.08.05-02	May 29, 2009
RAI 155 — 03.08.05-03	March 31, 2009
RAI 155 — 03.08.05-04	March 31, 2009
RAI 155 — 03.08.05-05	April 30, 2009
RAI 155 — 03.08.05-06	May 29, 2009
RAI 155 — 03.08.05-07	June 30, 2009
RAI 155 — 03.08.05-08	July 31, 2009
RAI 155 — 03.08.05-09	March 31, 2009
RAI 155 — 03.08.05-10	July 31, 2009
RAI 155 — 03.08.05-11	April 30, 2009
RAI 155 — 03.08.05-12	April 30, 2009
RAI 155 — 03.08.05-13	June 30, 2009
RAI 155 — 03.08.05-14	June 30, 2009
RAI 155 — 03.08.05-15	June 30, 2009
RAI 155 — 03.08.05-16	June 30, 2009
RAI 155 — 03.08.05-17	March 31, 2009
RAI 155 — 03.08.05-18	June 30, 2009

Sincerely,

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From: Getachew Tesfaye [mailto:Getachew.Tesfaye@nrc.gov]
Sent: Wednesday, January 14, 2009 9:33 AM
To: ZZ-DL-A-USEPR-DL
Cc: Jim Xu; Samir Chakrabarti; Sujit Samaddar; Michael Miernicki; Joseph Colaccino; ArevaEPRDCPEm Resource
Subject: U.S. EPR Design Certification Application RAI No. 155 (1671, 1831,1672, 1834, 1833, 1836), FSAR Ch. 3

Attached please find the subject requests for additional information (RAI). A draft of the RAI was provided to you on December 12, 2008, and discussed with your staff on January 13, 2009. No changes were made to the Draft RAI Questions as a result of that discussion. The schedule we have established for review of your application assumes technically correct and complete responses within 30 days of receipt of RAIs. For any RAIs that cannot be answered within 30 days, it is expected that a date for receipt of this information will be provided to the staff within the 30 day period so that the staff can assess how this information will impact the published schedule.

Thanks,

Getachew Tesfaye
Sr. Project Manager
NRO/DNRL/NARP
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Hearing Identifier: AREVA_EPR_DC_RAIs
Email Number: 536

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Response to

Request for Additional Information No. 155, Supplement 3

01/14/2009

U. S. EPR Standard Design Certification
AREVA NP Inc.
Docket No. 52-020
SRP Section: 03.08.01 - Concrete Containment
SRP Section: 03.08.02 - Steel Containment
SRP Section: 03.08.03 - Concrete and Steel Internal Structures of Steel or Concrete Containments
SRP Section: 03.08.04 - Other Seismic Category I Structures
SRP Section: 03.08.05 - Foundations
Application Section: FSAR Section 3.8

QUESTIONS for Structural Engineering Branch 2 (ESBWR/ABWR Projects) (SEB2)

Question 03.08.01-8:

FSAR Section 3.8.1.4 describes the design and analysis procedures for the post-tensioned RCB, which utilizes a finite element model (FEM) of the containment. AREVA is requested to address the items listed below related to the FEM and load applications:

1. Confirm that one FEM representing the RCB, RB internal structures, RSB, FB, SB, and common basemat is utilized for design analysis. Also, confirm that this one model is used for analysis of all loads identified in Section 3.8.1.3.1. Provide a description of how each of the different loads is applied to the model. In the case of seismic loads, explain which seismic model and seismic analysis they are taken from, in what form (e.g., maximum acceleration value from the time history analysis in each direction at each node) and how are they applied to the FEM.
2. FSAR Section 3.8.1.4.1 indicates that five layers of ANSYS SOLID45 elements are used through the thickness of the containment wall and dome. Explain why FSAR Figure 3.8-15 only shows four elements through the thickness of the containment dome. Provide the technical basis for concluding that four or five elements through the thickness of the containment shell are considered to be sufficient.
3. FSAR Section 3.8.1.4.1 indicates that the ANSYS SOLID45 finite element is a three-dimensional, four node brick element that is suitable for moderately thick shell elements. Explain whether this should have stated that the SOLID45 element is an eight node brick element instead.
4. Describe how the reinforcement is represented/modeled in the concrete brick type finite elements used in the model.
5. Explain where and why the ANSYS SOLID95 and SOLID92 finite elements are utilized.
6. Describe how the liner and anchorage of the liner were modeled in the RCB FEM, including the liner anchorage attachment method and spacing compared to the actual liner anchorage spacing. If the liner anchor spacing in the FEM does not match the actual spacing, explain (a) why the liner strains obtained from this analysis are considered to be accurate for checking against the strain limits specified in the ASME Code and (b) how are the liner anchor loads determined from the FEM analysis results and how are the loads used in checking the design adequacy of the anchors. As noted in FSAR Section 3.8.4.1, the strength of the liner is not relied upon to carry structural loadings; explain how this was achieved in the FEM.
7. FSAR Section 3.8.4.1, states that forces from the tendons are applied to the finite element "links" by imposing stains along the lengths of the modeled tendons and tensioning losses are explicitly included in these calculations. The calculated reactions forces from the tendon model are then applied as forces to the RCB model. Explain whether the analysis of the RCB model was performed for the maximum tendon forces due to initial pre-tensioning of the tendons, as well as the minimum (reduced) tendon forces occurring at the end of the 60 year period of performance of the EPR. If both cases were not analyzed, explain why not.

Response to Question 03.08.01-8:

1. A finite element model is used for common basemat structures of the Nuclear Island including Reactor Containment Building (RCB), Reactor Building Internal Structures (RBIS), Safeguard Buildings (SB), Fuel Building (FB), Reactor Shield Building (RSB) and

Shield Building for Safeguard Buildings 2&3 and Fuel Building. This model is referred as "NI static model".

This NI static model is used for analysis of all loads identified in U.S. EPR FSAR Tier 2, Section 3.8.1.3.

The following loads described in U.S. EPR FSAR Tier 2, Section 3.8.1.3.1, are applied to the RCB of the U.S. EPR NI static model by accelerating mass or applying forces to elements. SURF154 elements are applied over the entire model NI static model where loads occur to facilitate the application of loads. This element was chosen because it has capabilities to orient an applied load to the element other than just normal to the element face or in-plane. In some cases, multiple layers of this element were used in order to apply loads in different directions on the same element.

Dead Loads (D)

Dead loads applied to the NI static model include the dead weight of the NI structure, as well as additional uniform and concentrated dead loads to account for the equipment and other permanent items having significant mass. The dead weight of the NI structure is accounted for by accelerating the structure in the direction opposite to gravity or applying an acceleration value of 1.0g in the vertical-up direction. This acceleration, in conjunction with the NI structure mass properties, represents the dead weight of the NI structure concrete and liner plate. Uniform dead loads for the weight of the miscellaneous equipment are applied as uniform pressure to the slabs and both faces of the wall surfaces of the RCB using SURF154 elements. The total dead weight of the Reactor Building polar crane is applied by assigning one-quarter of the crane dead weight to each of the four end trucks. Based on the size of the end trucks and the spacing of the polar crane rail support brackets, the load from each end truck is applied to two brackets. The polar crane dead weight is applied as concentrated forces to the eight brackets.

Live Loads (L)

There are no live loads or precipitation loads applied to the RCB of the NI static model.

Soil Loads (H)

There are no static soil loads or lateral earth pressure loads applied to the RCB of the NI static model.

Hydrostatic Loads (F)

There are no direct hydrostatic loads applicable to the RCB of the NI static model.

Thermal Loads (To)

The temperature differential through the thickness of the RCB wall is very small. During normal operating conditions (To) and during containment testing (Tt), the annulus is approximately 79ºF while the RCB service compartments are approximately 72.5ºF, resulting in a temperature differential of 6.5ºF. This small temperature differential will result in insignificant loadings on the RCB areas designed therein, and therefore are not explicitly applied in the RCB.

Normal Pipe Reactions (Ro)

Normal pipe reactions are not applied to the RCB of the NI static Model. This load is considered as part of the local design where applicable.

Tendon Loads (J)

Post-tensioning effects in the containment wall and dome are applied by equivalent post-tensioning loads. Post-tensioning loads corresponding to equivalent pre-stressing forces for a 0 year and a 60 year period are calculated considering the three dimensional tendon profiles, geometric and material properties of the tendons and containment materials, wobble and curvature effects, creep and shrinkage properties of concrete, relaxation of tendon materials and number of jacking ends.

The containment model part of the NI static model contains nodes along the path of each type of tendon. To create tendons, nodes belonging to the solid elements along the tendon path are copied to the same location. Thus, for each tendon node there is a solid node at the same location. LINK8 elements with initial strains assigned as real constants are used for finite element modeling of the tendons. Initial strains at the end elements of the each tendon are directly calculated from the jacking stress for 0 year and 60 year period. Jacking stress of the tendons was calculated considering the creep and shrinkage of concrete and the relaxation of the post-tensioning tendons for the specified period. Initial strains for the interior link elements are calculated considering the friction coefficients and distance from the jacking end as well as curvature coefficients and geometric curvature of the tendon profile. At each end of the tendons, BEAM4 elements are added to distribute the jacking stress over a large area. The tendon model, consisting of LINK8 and BEAM4 elements for the vertical, gamma and hoop tendons are separated from the solid containment model and used for post-tensioning tendon load calculations for the U.S. EPR containment model.

The tendon model consists of 51 vertical, 104 gamma and 119 hoop tendons. For each tendon, all degrees of freedom are locked in all directions and solved for reactions. Three nodal X, Y, and Z reactions at each node for each tendon are stored for both the 0 and 60 year period. The containment part of the NI static model uses the solid elements only. Tendon nodal reactions are applied to corresponding solid nodes (physically located at the same location as nodal loads) as nodal forces with the opposite sign to simulate the post-tensioning effect from the tendons for the 0 or 60 year period.

Relief Valve loads (G)

There are no relief valve loads applied to the RCB of the NI static model. This will be considered as part of the local design, where applicable.

Pressure Variant Loads (Pv)

During normal operating conditions, the pressure in the annulus and in the RCB service compartment is maintained at -0.03 psig. The pressure differential (Pv) across the RCB wall is zero and is not applied to the RCB of the NI static model.

Construction Loads

Construction loads are not explicitly considered in RCB of the NI static model.

Test Loads (Pt and Tt)

During the test condition, the temperature remains constant throughout. The test pressure gradually increases to a peak value and stays constant for a period, then gradually reduces. Since the temperature remains constant for the test period, the maximum forces and moments in the containment sections are developed for the test pressure and temperature condition when the applied pressure is at a maximum. Test pressure loads are applied to the containment interior corresponding to maximum internal pressure. This pressure is applied as normal uniform pressure on the liner elements (SHELL181) on RCB wall and dome. This pressure is also applied as normal uniform pressure on the top of the RBIS foundation using SURF154 elements.

Temperature Loads (Ta)

Accidental temperature loads are applied for four different critical time points of the design temperature transient profile. A thermal transient analysis is performed for the containment. Design forces and moments are calculated from the thermal stress analysis performed for temperature distribution of transient analysis as well as structural analysis for the design pressure transient. These analyses results are used for identification of the critical time points. Temperature distributions obtained from this transient thermal analysis for these critical time points are applied as nodal temperature loads on the NI model.

In the accidental case, due to the increase of temperature, the containment liner expands. This expansion of the liner exerts additional pressure on the concrete containment walls and dome. Such additional pressure loads are calculated for the corresponding critical time points. The effect of these additional pressures is considered as acting together with accidental load cases and applied as normal pressure with SURF154 elements on containment wall and dome.

Pressure Loads (Pa)

Accidental pressure loadings are applied for four different critical time points identified from the design temperature and pressure transient. This pressure is applied as a normal uniform pressure on the liner elements (SHELL181) on the RCB wall and dome. This pressure is also applied as normal uniform pressure on the top of the RBIS foundation using SURF154 elements.

Accident Pipe Reactions (Ra)

There are no accidental pipe reaction loads applied to the RCB of the NI static model. This will be considered as part of the local design, where applicable.

Pipe Break Loads (Rr)

There are no pipe rupture loads applied to the RCB of the NI static Model. This is considered as part of the local design, where applicable.

Seismic Loads (E')

Appropriate zero period accelerations (ZPA) in three principal directions (for 100-40-40 rule) are calculated for different elevations of the RCB. The seismic loading on the dead weight of the structure was applied using the CMACEL command. A particular portion of the building was selected and made into an ANSYS component. The appropriate ZPA was applied to the component using the CMACEL command to represent the seismic effect on it. As discussed in the ANSYS Help Manual, subsequent CMACEL commands on a component will overwrite earlier CMACEL commands on the same component. Therefore, for a given component, it was necessary to apply the appropriate ZPAs for all three principal directions (X, Y, and Z) at the same time. For the additional dead loads other than self weight, additional dead loads (applied as normal pressure) are multiplied with appropriate ZPA values to calculate the equivalent pressure in three principal directions. These three equivalent seismic pressures on walls and dome of RCB for seismic loads are applied as directional pressure loads (parallel to principal directions) using SURF154 elements. Since there are no additional loads on the RCB wall and dome due to live loads, such as hydrostatic loads, static soil pressure, etc., these loads are not included.

For the purpose of capturing the dynamic response and generating the in-structure response spectra for the Nuclear Island (NI) structures, a stick model is developed. This stick model is composed of nine sticks; each representing a structure of the NI. This stick model is tuned so that its modal properties are similar to those of the NI static model. This tuned NI Stick model is used for seismic load calculation to be applied in the NI static model. Ten generic soil conditions are considered that could potentially exist in the United States, along with three different free-field seismic control motions. Each of the ten generic soil conditions is associated with one or, in a few cases, two of the seismic control motions. For each seismic control motion, three components of spectrum-compatible synthetic time history of motion are generated for use as seismic input motions to the Soil Structure Interaction (SSI) analysis. SSI analyses are performed for all considered soil cases and three different free field seismic control motions. The ZPAs at the major floor elevations of the Nuclear Island are determined from SSI analysis. For each elevation of interest, the maximum accelerations at the center-of-mass and the corner locations of perimeter walls are enveloped. The maximum response acceleration at a given nodal location on the stick model of the structure is the maximum amplitude (i.e., the ZPA) of the corresponding nodal response acceleration time history output from the SSI analysis. To consider the contributions from all three components of input motion, the three output floor acceleration time histories in a given direction resulting from the three components of input motions are algebraically summed to produce the resultant floor acceleration response time history in the same given direction. For example, the resultant acceleration time history in the X-direction is the algebraic sum of the X-direction time-histories due to input motions in the X, Y and Z directions. The corresponding ZPA is taken as the maximum amplitude of the resultant floor acceleration time history in the respective directions.

2. The wall and dome of RCB model are modeled with five and four layers of solid elements along the thickness, respectively. U.S. EPR FSAR Tier 2, Figure 3.8-15 correctly shows four elements through the thickness of the containment dome.

   The statement in U.S. EPR FSAR Tier 2, Section 3.8.1.4.1 will be revised as follows.

   Current Statement:

   "Five layers of SOLID45 elements are used to model through the thickness of the cylindrical shell wall and dome."

   Revised Statement:

   "Four and five layers of SOLID45 elements are used to model through the thickness of the dome and cylindrical shell wall, respectively."

   Based on the mesh study performed for the RCB wall and dome, the following has been observed:

   – Under accidental thermal loading, four and five layers of concrete overestimate the thermal gradient across the thickness at the beginning of the accident. Therefore, thermal moments calculated with the current mesh of RCB are conservative compared to a refined mesh of RCB at the beginning of the accident period.
   – Under structural loading (e.g., dead, pressure and prestressing loads) changes in forces and moments are insignificant for the mesh refinement.

3. The statement in U.S. EPR FSAR Tier 2, Section 3.8.1.4 is as follows:

   "SOLID45 is a three-dimensional, four-node brick element that is suitable for moderately thick shell structures."

   U.S. EPR FSAR Tier 2, Section 3.8.1.4 will be revised to read as follows:

   "SOLID45 is a three-dimensional, eight node brick element that is suitable for moderately thick shell structures."

4. Reinforcement is not explicitly modeled in the NI Static model. The RCB of the NI Static model is modeled with solid elements: SOLID45, SOLID 95 and SOLID 92.

5. The RCB of the NI Static model is predominantly modeled with brick shaped eight node SOLID45 elements. SOLID95 and SOLID92 elements are used to avoid degenerated SOLID45 elements and used in places where meshing with regular SOLID45 elements is not possible.

   The RCB model is generated in two parts: dome with ring-girder and containment walls. First, the geometry of the dome and ring girder is created with explicit lines of tendons.

This dome is meshed with SOLID45 elements by sweeping the volumes. Due to crossing of the upper and lower set of tendons in the ring girder zone, four regions of the ring girder cannot be meshed using volume sweeping. Due to the complex shape of the dome tendons, dome mesh cannot match with regularly meshed cylinder walls. To mesh the four ring girder zones and to provide a connection between containment ring girder and walls, it is necessary to use the ANSYS auto meshing option. In general, degenerated higher-order 20-node brick elements are better than degenerated linear SOLID45 elements. To avoid degenerated SOLID45 elements, meshing for these zones is done with 20-node SOLID95 and 10-node SOLID92 elements. ANSYS has a well-defined methodology for transition of higher order elements to lower order elements. At the junction of the SOLID45 and higher order elements, these higher order elements drop the mid side nodes to match with lower order elements.

6. The liner is modeled with 4-node SHELL181 elements applied on the inner surface as a pressure load transfer element, smeared over the inner face of the SOLID45 concrete elements. The liner and its anchorages are not considered as structural elements in the structural design of the containment and the liner anchorage is, thus, not explicitly modeled in the RCB portion of the NI static model. Liner elements share the same nodes with the inside faces of the concrete elements. In general, containment SOLID45 elements are approximately 0.6 m height, 2^{o} span in hoop direction and 0.25 m in depth. .

   The modulus of elasticity of the liner material is reduced to 1% of the actual strength to make the liner structurally inactive in the NI static model analysis as required by ASME Section III, Div. II. Since the liner elements and concrete elements share the same nodes, displacement of these common nodes is correct. Strain in the liner elements is obtained from the strain displacement matrix and nodal displacements. As the strain displacement matrix is a function of element shape function, the strain in the liner is accurate.

   Liner anchor loads are not determined from finite element analysis. Using the strains from FEM, a separate calculation determines the liner plate anchorage load based on the energy method described in Bechtel Topical Report BC-TOP-01. The loads in the anchorage are checked against the ASME Subsection CC-3700 requirement.

   The modulus of elasticity of the liner materials are reduced to 1 percent of the actual strength to make the liner structurally inactive in NI static model analysis.

7) The U.S. EPR NI static model considers tendon loads for both initial pre-stressing loads (zero year period) and reduced prestress loads corresponding to the end of a 60 year period. Selection of zero (0) or sixty (60) year period tendon induced load is based upon the load combination associated with the worst case or controlling design load. A summary of these loading combinations is provided as follows:
   a. Prestressing load aids load combinations with pressure loads. Therefore, prestressing loads associated with the 60 year period are considered for load combinations used with design basis accident pressure loads.
   b. Initial (zero year period) prestressing loads are considered for loading combinations that include the initial test pressure loads. Initial test pressure loads are applied after completion of plant construction and before fuel load, thus it is appropriate to use the 0 year period pretension in these combinations.

c. Prestressing loads corresponding to the initial (zero year period) are considered for loading combinations associated with normal operating conditions (no pressure or seismic loading). The worst case condition arises from the increased compression within the concrete due to prestressing loads. Since the zero period creates the larger compressive load it is considered in these analyses.
d. Prestressing loads associated with the 60 year period are considered for load combinations that include seismic loading.

References for Question 03.08.01-8:

1. BC-TOP-01, Revision 1, “Containment Building Liner Plate Design Report, Bechtel Corporation, Johnson, T.E.; Wedellsborg, B.W., OSTI ID: 4550930.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.8.1.4 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.01-9:

FSAR Section 3.8.1.4.5 describes how creep, shrinkage, and cracking of concrete were considered in the design of the RCB. It states that moments, forces, and shears are obtained on the basis of uncracked section properties in the static analysis. However, cracking of concrete sections was considered for the thermal loading case. If cracking can occur due to the thermal loading case, internal accident pressure, and/or the structural integrity test (SIT), what is the technical basis for not considering cracked section properties for loads other than the thermal loading case? It should be noted that ASME Code Section III, Division 2, Article CC-3320 – Shells, indicates that "Containments are normally thin shell structures. Elastic behavior shall be the accepted basis for predicting internal forces, displacements, and stability of thin shells. Effects of reduction in shear stiffness and tensile membrane stiffness due to cracking of the concrete shall be considered in methods for predicting maximum strains and deformations of the containment."

Response to Question 03.08.01-9:

A study was conducted which shows that design forces and moments are not significantly affected by considering cracked section properties for loads other than the thermal loading case. Specifically, a nonlinear model of a 6^{o} slice of the Reactor Containment Building (RCB), away from the discontinuities, is developed. In this model, solid elements (SOLID65) represent concrete, which has the capability to crack, and reinforcing steel. The model also provides explicit representation of prestressing tendons with link elements (LINK8). This model is converted to a linear model by removing the capacity for concrete cracking. Both the linear and nonlinear models are subjected to a series of loading combinations. First the models are subjected to a structural integrity test, then to an accidental temperature and pressure loading, and lastly to a pressure loading. Prestressing and dead loads are applied throughout the study. For comparison between the cracked and uncracked models, design forces and moments are calculated during the last pressure loading. It is observed that cracking of the concrete in the Containment Building does not significantly alter the design forces and moments as compared with those produced in the linear model.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.08.01-10:

FSAR Section 3.8.1.4.11 describes the calculation to determine the ultimate pressure capacity of the RCB. AREVA is requested to address the items listed below.

1. The introductory sentence to this FSAR section states that “The ultimate capacity of the RCB is determined for use in probabilistic risk assessments (see Section 19) and severe accident analyses.” NRC RG 1.136 indicates that the ultimate capacity of the concrete containment should be performed and refers to the guidance provided in SRP 3.8.1. As noted in SRP 3.8.1.II.4.K (Revision 2 – March 2007), the purpose of the containment ultimate pressure capacity evaluation is to obtain a measure of the safety margin above the design-basis accident pressure. This should be done utilizing deterministic calculations with minimum code-specified material stress-strain curves. The calculation of containment ultimate pressure capacity for use in probabilistic risk assessments (PRAs) should be evaluated separately using different criteria than those presented in SRP 3.8.1.II.4.K. These PRAs should be presented in Section 19 of the FSAR.Thus, FSAR Section 3.8.1.4.11 should be revised to reflect the intent of this section and AREVA is requested to confirm whether the approach and criteria utilized to calculate the containment ultimate pressure capacity was performed in accordance with the guidance in SRP 3.8.1.II.4.K. Otherwise, provide the technical basis for any deviations from this guidance.
2. FSAR Section 3.8.1.4.11 indicates that the pressure capacity for various structural elements were based on the median pressure capacity. As discussed under item 1 above, the containment ultimate pressure capacity should not be determined on a probabilistic basis. Provide the containment ultimate pressure capacity for the various containment elements on a deterministic basis in accordance with SRP 3.8.1.II.4.K, or provide the technical basis for alternative criteria.
3. To support the results presented in FSAR Table 3.8-6, provide a description (including figures) which summarize and show: the models, material properties and material modeling, computer codes, loading sequences, tendon relaxation effects, concrete shrinkage & creep, potential failure modes, assumptions, and results.
4. Confirm that all of the material properties were based on code-specified material properties at the design-basis accident temperature.
5. The end of the last paragraph of FSAR Section 3.8.1.4.11 indicates that the ultimate pressure capacity reported corresponds to the ASME Service Level C stress limits for the hatch cover and cylinder. Explain why this limit was selected to determine the ultimate pressure capacity of the hatch cover and cylinder rather than the true ultimate capacity of the components.
6. In addition to the structural integrity calculations, how was leakage from the various containment elements (e.g., penetrations, bolted connections, seals, hatches, bellows) evaluated and what leakage acceptance criteria were utilized to verify the final ultimate capacity of the containment?

Response to Question 03.08.01-10:

1. The current U.S. EPR FSAR Tier 2, Section 3.8.1.4.11 will be revised to clarify that the methodology used to determine containment ultimate capacity was performed in accordance with the guidance provided in SRP 3.8.1.II.4.K (Revision 2—March 2007).
2. Containment ultimate capacity deterministic analyses for the various containment structural elements was performed in accordance with the guidance provided in SRP 3.8.1.II.4.K, including base criterions and minimum code-specified material stress-strain curves. Therefore, all statements with term “median pressure capacity” in U.S. EPR FSAR Tier 2, Section 3.8.1.4.11 will be changed to “ultimate pressure capacity”.
3. The revised U.S. EPR FSAR Tier 2, Table 3.8-6 will be revised as follows:

Table 3.8-6—Containment Ultimate Pressure Capacity (P_u) at Accident Temperature of 309°F

Section	P_u (psig)	Ratio P_u/P_d	Failure Mode/Location
Cylinder	267	4.31	Failure due to maximum allowable membrane strains away from structural discontinuities.
Dome	249	4.02	Failure due to maximum allowable membrane strains away from structural discontinuities.
Dome Belt	173	2.79	Failure due to maximum allowable flexural strains at structural discontinuities.
Gusset Base	315	5.08	Failure due to maximum allowable flexural strains at structural discontinuities.
Equipment Hatch(1)	156	2.52	“Loss” of structural integrity in protruding sleeve area due to principal strain which approach ultimate.
Equipment Hatch (1)	125	2.02	“Loss” of leak tightness in protruding sleeve due to principal strain which approach ultimate.

NOTES:

Pd – design pressure

(1) conservatively calculated under Accident Temperature of 338°F

a) The ANSYS finite element program is used to model the Containment Building as a two degrees wide wedge (slice) with simulated axisymmetric boundary conditions for the modeling of the cracked concrete section behavior. SOLID65 elements (reinforced concrete) are used to model concrete with cracking set at $4\sqrt{f'_c}$. Reinforcement capabilities of the SOLID65 are not used. Instead, tendon and passive reinforcement are modeled as a membrane corresponding to the reinforcement location using SHELL43 elements (four nodes plastic large strain shell elements). Separate SHELL43 elements are overlaid to model vertical vs. hoop reinforcing since these reinforcing amounts may be different. SHELL43 is also used on the liner plate using isotropic material properties. Geometric and material non-linearity (elastic-perfectly plastic material) are accounted for with the large displacement option turned on in ANSYS. The finite element mesh is shown in Figure 03.08.01-10-1.

Loading sequence: Load steps 1 to 3 apply initial loads (i.e., dead weight and post-tension) first and reach equilibrium. Then, load steps 4 to 5 apply the accident loads (accident temperature followed by incremental internal pressure up to the ultimate capacity).

1) First Load Step – no loads for initialization purpose.
2) Second Load Step – dead load (Global Z-direction) only.
3) Third Load Step – post-tension loading in hoop, vertical and dome tendons.
4) Fourth Load Step - accident temperature load to liner elements.
5) Remaining Load Steps – the containment pressure is incrementally increased in 1 psi steps from zero up to the ultimate pressure.

The ANSYS non-linear finite element analysis is performed for the equipment hatch evaluation. SA516 Grade 70 steel is used for the hatch cover, flanges and sleeve in the analysis. The structure contains a hatch cover (of tore-spherical shape), a sleeve, and two flanges plus 40 clamps that link hatch and sleeve. The model used SHELL43 for hatch, flanges and sleeve (including protruding sleeve), CONTAC52 for contact between flanges, LINK8 for tightness joints and clamps, and PIPE16 to connect contact flange to the middle plane. The equipment hatch FE-Model is shown in Figure 03.08.01-10-2.

The analyses considered dead load plus pre-stressing of clamps, accident temperature, an imposed displacement field from the containment wall (dead weight, tendon post-tension and accident thermal) and an incrementally increasing pressure load (applied to the convex part of hatch cover and protruding part of sleeve) until strain criteria are reached. The stress-strain relationship used for the analysis is elastic-perfectly plastic. (i.e., bilinear kinematic hardening based on Von Mises yield criteria). Geometric nonlinearity is accounted for by using the large displacement procedure. The temperature inside the Containment Building is based on 338^{o}F (170^{o}C) which is larger than the accident temperature of 309^{o}F.

Two types of failure mechanism are analyzed:

1) Possible loss of leak tightness (contact element opening) under pressure of 125 psi, P_u/P_d = 125/62 = 2.02 (about 2 times above design pressure);
2) Possible loss of structural integrity (principal strain approach the ultimate) under pressure of 156 psi, P_u/P_d = 156/62 = 2.5 (about 2.5 times above design pressure)

b) The material properties at elevated temperatures used in ANSYS analysis are given below:

- Liner steel SA516 Gr. 70: f_y = 33.5 ksi and E = 28,300 ksi at accident temperature of 309^{o}F:
- Concrete material properties at temperature of 176^{o}F, in accordance with the results from the heat transfer analysis across the concrete thickness:
  - ♦ Concrete Strength Ratio at 176^{o}F (80^{o}C) $S_{rc} = e^{-(80/632)^{\wedge}1.8} = e^{-0.024} = 0.98$
  - ♦ Concrete Compressive Strength f_c' = 0.98(7000 psi) = 6860 psi

- ♦ Concrete Young's Modulus $E_c = 57{,}000\ (6860)^{0.5} = 4721$ ksi

- Because, the rebars and tendons material properties are only slightly changed in temperature range from 70°F (room temperature) to 400°F > 309°F (accident temperature) >> 176°F (concrete temperature), therefore:
  - ♦ Tendons steel ASTM A416 Gr. 270: f_y = 243 ksi and E = 28,000 ksi at T = 70°F
  - ♦ Rebars steel ASTM A615: f_y = 60 ksi and E = 29,000 ksi at T = 70°F
  - ♦ Equipment hatch steel SA516 Grade 70: f_y = 33.5 ksi and E = 28,300 at T = 309°F.

c) The post-tension load applied in ANSYS FE containment model is modeled as an equivalent temperature decrease load case in tendons layers (SHELL43 elements) with taking into account all losses after 60 years due to: relaxation, shrinkage, creep, friction and elastic shorting as shown in Table 03.08.01-10-1 below:

Table 03.08.01-10-1—Tendon Stress

Tendon Stress (ksi)	Hoop	%	Vertical	%	Gamma	%
Initial	197.10	100.0%	197.10	100.0%	197.10	100.0%
Friction	-49.39	-25.1%	-9.66	-4.9%	-43.81	-22.2%
Elastic Shorting	-6.98	-3.5%	-3.42	-1.7%	-4.90	-2.5%
Creep	-21.28	-10.8%	-11.76	-6.0%	-19.64	-10.0%
Shrinkage	-11.20	-5.7%	-11.20	-5.7%	-11.77	-6.0%
Steel Relaxation	-9.86	-5.0%	-9.86	-5.0%	-9.86	-5.0%
Final Tendon Stress	98.39	49.9%	151.20	76.7%	107.12	54.3%
Equivalent Temp (°F)	-541		-831		-589	

d) The results of the provided analyses with the controlled failure modes are presented in U.S. EPR FSAR Tier 2, Table 3.8-6—Containment Ultimate Pressure Capacity at Accident Temperature of 309°F, as modified in the Response to Question 03.08.01-10, Item 3 preceding, where the failure modes for the different structural elements/sections are:

- Cylinder & Dome in sections away from structural discontinuities:
- Failure due to maximum strains in tendons which is approached the allowable strain of 0.8%.
- Dome Belt & Gusset Base for sections direct in discontinuities:
- Failure due to maximum strains in rebars which is approached the allowable strain of 0.8%.
- Equipment Hatch:
  - ♦ Failure Mechanism 1: Possible loss of leak tightness due to contact element opening.
  - ♦ Failure Mechanism 2: Possible loss of structural integrity due to maximum principle strain which is approached the allowable strain of:
    - Allowable membrane strain: $\varepsilon_{compression}$ = 0.5% $\varepsilon_{tension}$ = 0.3%
    - Allowable membrane + bending strain: $\varepsilon_{compression}$ = 1.4% $\varepsilon_{tension}$= 1.0%

4. All of the material properties used in “Containment Ultimate Capacity Deterministic Analyses” are based on code-specified material properties at the design-basis accident temperature as it is described in the Response to Question 03.08.01-10, Item 3 preceding.

5. The end of the last paragraph in U.S. EPR FSAR Tier 2, Section 3.8.1.4.11 will be revised as follows:

   “Since the equipment hatch performs a leak tightness role, the allowable strain criteria in accordance with ASME Code, Section III, Div. 2, Subsection CC, Article CC-3720 is conservatively used for the hatch ultimate pressure capacity evaluation. These allowable strains are:

   membrane strain: $\varepsilon_{compression}$ =0.5%, $\varepsilon_{tension}$ =0.3% and

   membrane + bending strain $\varepsilon_{compression}$=1.4%, $\varepsilon_{tension}$=1%.

   The estimated ultimate pressure capacities are determined from the principal strain levels which approach ultimate in the protruding sleeves, while remaining below yield in the hatch and flange areas”.

6. All connection elements and boundary contacted surfaces in the equipment hatch are modeled as special non-linear contact elements. Strain conditions of these elements associated with the ultimate capacity pressure and temperature design criteria did not open (no positive strain) at the contact surfaces. Therefore, no special leakage criteria are considered for this accident condition.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.8.1.4.11 and Table 3.8-6 will be revised as described in the response and indicated on the enclosed markup.

Figure 03.08.01-10-1—ANSYS Finite Element Model of Containment (2 Degree Slice) with Axisymmetric Boundary Conditions and Internal Pressure (red arrows)

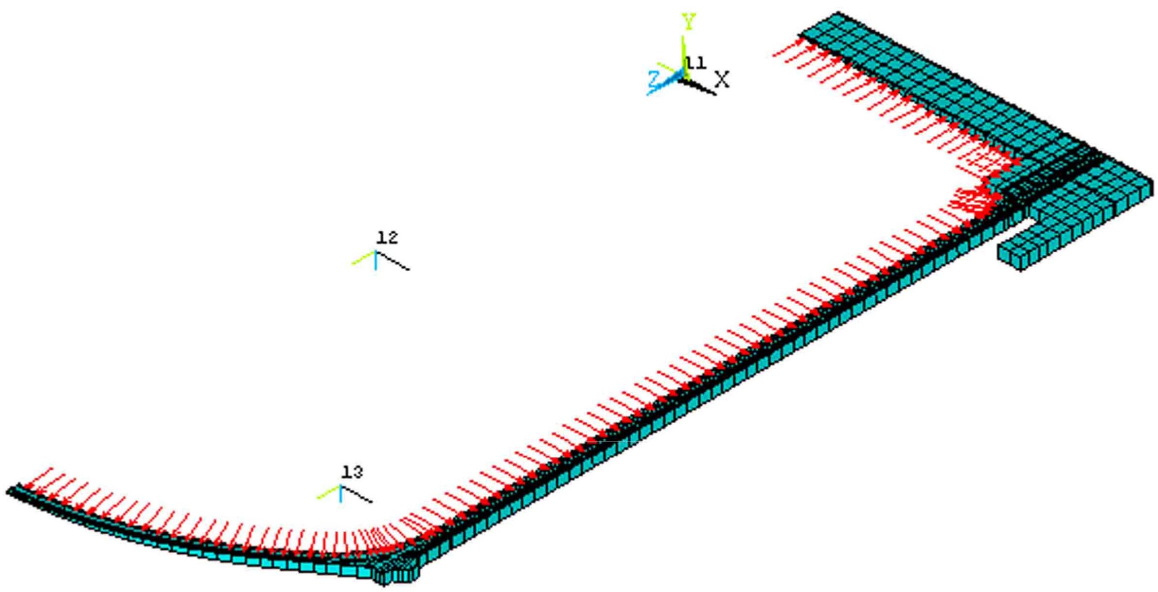
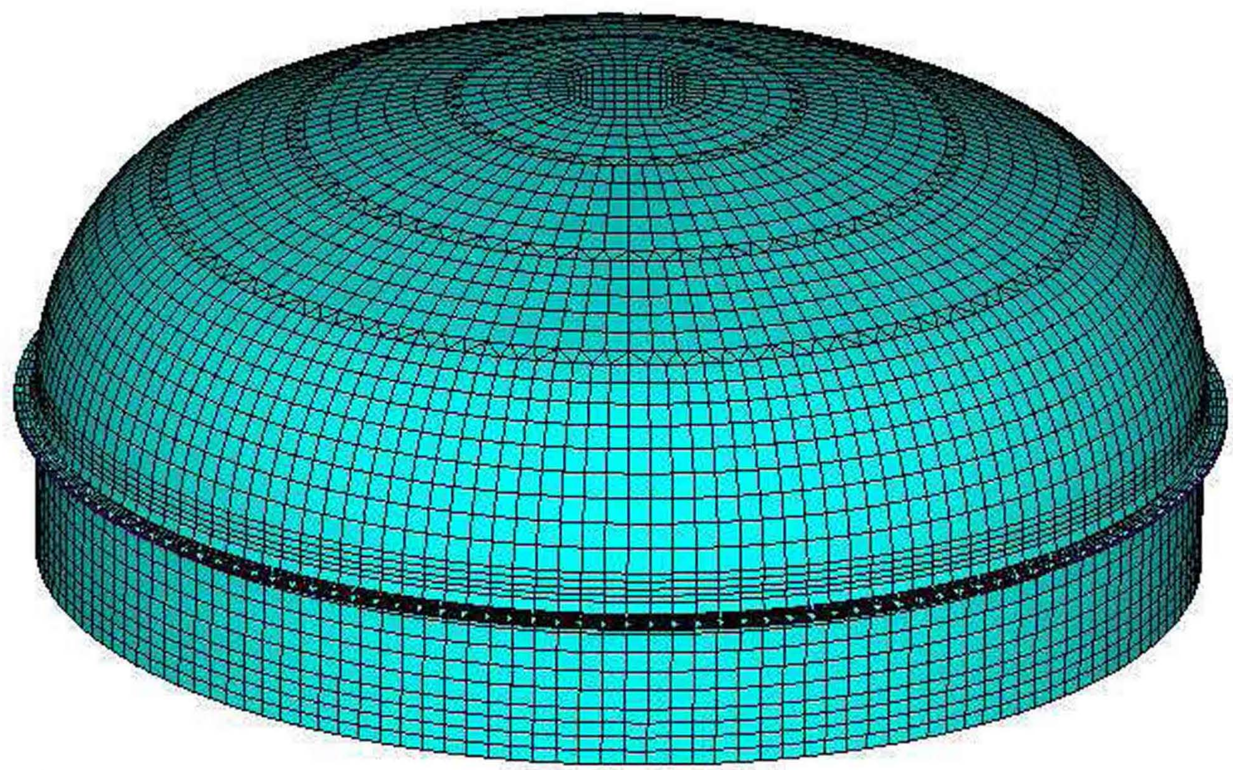


Figure 03.08.01-10-2—Equipment Hatch ANSYS Finite Element Model



Question 03.08.01-12:

RG 1.90 requires that the reactor containment be tested to 1.15 times the design pressure at years three and seven. In FSAR Section 3.8.1.7.2, it states that pressurization at years three and seven uses P_a instead of 1.15 times the design pressure. It also states that testing at 1.15 times the design pressure unduly fatigues the structure. Provide sufficient technical justification for not following the criterion for pressure testing in RG 1.90 and the basis for stating that testing at 1.15 times the design pressure unduly fatigues the structure.

In addition, FSAR Section 3.8.1.7.2 states that an exception is taken with respect to RG 1.90 whereby the force monitoring of ungrouted tendons is not provided. The FSAR states that this "is acceptable because all tendons used with the RCB are fully grouted." This is not an acceptable technical basis for taking an exception to providing three tendons in each tendon group (horizontal, vertical, and dome) as specified in RG 1.90. AREVA is requested to provide a valid technical basis for not meeting RG 1.90 or provide an alternate method for meeting the intent of this provision in RG 1.90.

Response to Question 03.08.01-12:

Based on NRC guidance provided by RG 1.90, one major issue is the potential for widespread corrosion of the tendon steel that could occur and remain undetected. In other types of structure, the grouting of the tendons is an effective means of protection against corrosion. In addition to the grouting of tendons, the U.S. EPR design has the added advantage of the Reactor Shield Building (RSB) in that the Reactor Containment Building (RCB) is fully protected against the potential of water intrusion due to the exposure to environmental conditions (i.e., rain, snow, freeze-thaw action).

By using $1.15P_D$ during the initial structural integity test, the design assumptions and quality of construction are confirmed, and measured strains and deformation are evaluated to verify containment integrity. Prestressing containment counterbalances the tensile forces arising from the design pressure. Once proven during the initial structural integrity test,continued pressurization of the containment to $1.15P_D$ induces unnecessary cyclic loading of the structure. The use of P_a from ISI forward will establish a continuous basis for comparison of results, will minimize gradual propagation of cracking during subsequent pressure tests, and will be in compliance with the ISI requirements of ASME BVP Code Subsection IWL, Paragraph IWL-5220.

RG 1.90 requires three ungrouted test tendons to be installed in each tendon direction for ISI lift-off or load cells testing. Per RG 1.90 the purpose of these tendons is to evaluate the extent of concrete creep and shrinkage, and tendon relaxation. Since each one of these tendons is to be tested during every ISI, the tendons will be subject to cyclic loading thus introducing an additional factor that can affect measured results. It has been acknowledged in NRC Information Notice 99-10, Attachment 3 that more appropriate methodology is the random selection of tendons for testing, which would not be possible in U.S. EPR containment with remaining tendons being grouted. It should also be noted that when testing ungrouted tendons an assumption is made that the tension at the anchorage during lift-off represents the tension along the tendons. Rather than using ungrouted tendons for monitoring volumetric changes due to prestress losses, the U.S. EPR ISI program will implement monitoring of containment deformation under design basis accident pressure Pa, and compare results with expected deformation and ISI deformation. The method of comparison of deformation of the structure during ISI pressure testing with deformation during ISI has been accepted previously by NRC as a basis for evaluating the functionality of the structure for the Three Mile Island and Forked River Nuclear Power Stations, as identified by RG 1.90, Section 2.b. Further support of this ISI scheme has been demonstrated at the Quinshan Nuclear Power Plant (1). There it has been demonstrated that overall deformation monitoring of the prestress level in the containment is a practical and accurate alternative for tendon force measurements.

References for Question 03.08.01-12:

1. Z. Sun, S. Liu, S. Lin, Y. Xie, "Strength Monitoring of a Prestressed Concrete Containment with Grouted Tendons", Nuclear Engineering and Design Journal, Volume 216, Issues 1-3, pp 213-220, pub. Elsevier B.V., July 2002.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.8.1.7.2 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.01-16:

FSAR Section 3.8.1.4.8 states that in the design and analysis of the RCB consideration is given to the effects of possible variations in the physical properties of material on the analysis results. It further states that the properties used were established based on past engineering experience with similar construction and materials. Provide a discussion of how the variation of properties in the design of the containment was addressed in Tables 3.8-1, 3.8-2, 3.8-3, and 3.8-4 and provide a technical basis for using the properties listed. In addition, explain how variation in material properties was considered for other structures described in FSAR Sections 3.8.2 through 3.8.5. This should include the potential effects of high irradiation on structural members close to the reactor pressure vessel such as the reactor vessel concrete support structure.

Response to Question 03.08.01-16:

The values delineated in U.S. EPR FSAR Tier 2, Tables 3.8-1 through 3.8-4 are best-estimate values used for the analysis and design of the Reactor Containment Building (RCB). A parametric study was performed with the axisymmetric model of the RCB to identify the changes in design forces and moments corresponding to the changes in thermal and mechanical properties.

U.S. EPR FSAR Tier 2, Table 3.8-1 lists the thermal properties (i.e., thermal conductivity and specific heat) of the containment materials. Thermal conductivity of steel and concrete are insignificantly influenced by the rise in temperature, whereas the specific heat of concrete and steel may change. Based on the study performed with the axisymmetric model of the RCB, changes in specific heat of concrete and steel do not significantly change the design forces and moments.

U.S. EPR FSAR Tier 2, Table 3.8-2 states the mechanical properties of concrete, post-tensioning cable and reinforcements. Modulus of elasticity of concrete may decrease with increase in temperature. Reduction in modulus of elasticity decreases the thermal moments, whereas design forces and moments due to pressure loads do not change significantly.

U.S. EPR FSAR Tier 2, Table 3-8-3 list the friction loss coefficients for tendon materials which are the best-estimate value based on the tests performed for similar tendons used in nuclear power plants. These values are the same values used for the European version of the EPR plant, and are considered best estimates since both the European EPR plant and U.S. EPR plant use functionally identical tendon systems. No variation in the friction loss coefficients is provided.

U.S. EPR FSAR Tier 2, Table 3.8-4 lists the convection parameters between the atmosphere and internal/annulus face of containment for heat transfer analysis. To be conservative, convection film coefficient is considered infinite between the atmosphere and interior face of the containment. Based on the minimum and maximum temperature of the annulus area, convection parameters will change the design forces and moments insignificantly.

In the Reactor Building internal structure (RBIS), the reactor pressure vessel is shielded by primary and secondary shield walls. Based on the arrangement of the primary and secondary shield walls around the reactor pressure vessel, potential detrimental effects of irradiation on material properties are considered insignificant.

The variation of material properties was not considered for structures described in U.S. EPR FSAR Tier 2, Sections 3.8.2 through 3.8.5. The analysis and design of these structures was performed utilizing best-estimate values, as was done for the RCB.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.08.01-22:

FSAR Section 3E.1.1 and other sections of Appendix E state that a separate analysis was performed to estimate the effects of cracked concrete and based on the results of the analysis the thermal moments carried by portions of the RCB were reduced. Describe the analysis performed including a description of computer codes, identify other concurrent loads that were considered in the analysis, the method used for reducing the thermal moments, how the final design loads were determined, and identify the portions of the RCB where this was done. Provide a similar description for the treatment of thermal moments in FSAR section 3.8.3, 3.8.4, and 3.8.5. Include in this discussion under what conditions these moments were considered and where in each structure thermal moments were reduced.

Response to Question 03.08.01-22:

A calculation was performed to evaluate the change in magnitude in thermal moments in prestressed concrete containment model resulting from mesh refinement and cracking of the concrete section. The analysis was performed with finite element program ANSYS 11.0 (Service pack 1).

The analysis approach for calculating the thermal moment modification factor is as follows:

- A typical sector of the containment away from the discontinuities is identified and shown in Figure 03.08.01-22-1. A 6^{o} slice of the containment is selected for study.
- An “equivalent slice model” of 6^{o} sector is developed with SOLID45 (linear) element and similar mesh density of Reactor Containment Building (RCB) is first developed to reproduce the thermal and structural results of RCB of the Nuclear Island (NI).
- This equivalent slice model is further refined for the mesh density to calculate the change in thermal moments due to mesh refinement. This “refined slice model” is solved for thermal loading only.
- Comparison of thermal moments between the equivalent and refined slice models provides the thermal modification factor due to mesh refinement.
- This linear slice model is converted to nonlinear slice model by changing the SOLID45 elements to SOLID65 elements allowing cracking as well as considering reinforcement.
- The cracking of the concrete depends on the stress-strain status under the load combinations and load history. When the containment is built, prestressing tendons are activated and the containment is subjected to test pressure loads. To account for the possibility of an accident, when the plant goes to operation, the nonlinear slice model is solved for two load combinations. In the first load combination, initially the dead load is applied to prestressed nonlinear model, then test pressure is applied and removed and finally the model is subjected to accidental pressure loads. In the second load combination, initially the dead load is applied to prestressed nonlinear model, then test pressure is applied and removed, and finally the model is subjected to accidental temperature and pressure loads. The moments of the nonlinear slice model under thermal loads are calculated as the differences between the moments from these two load combination transients.

- The thermal moments from the nonlinear slice model are compared with the thermal moments from refined slice model. These results provide the modification in thermal moment due to cracking of concrete.
- Since the modification factors due to mesh refinement and cracking of concrete are independent of each other, the thermal moment reduction factor is calculated through multiplication two thermal moment modification factors.

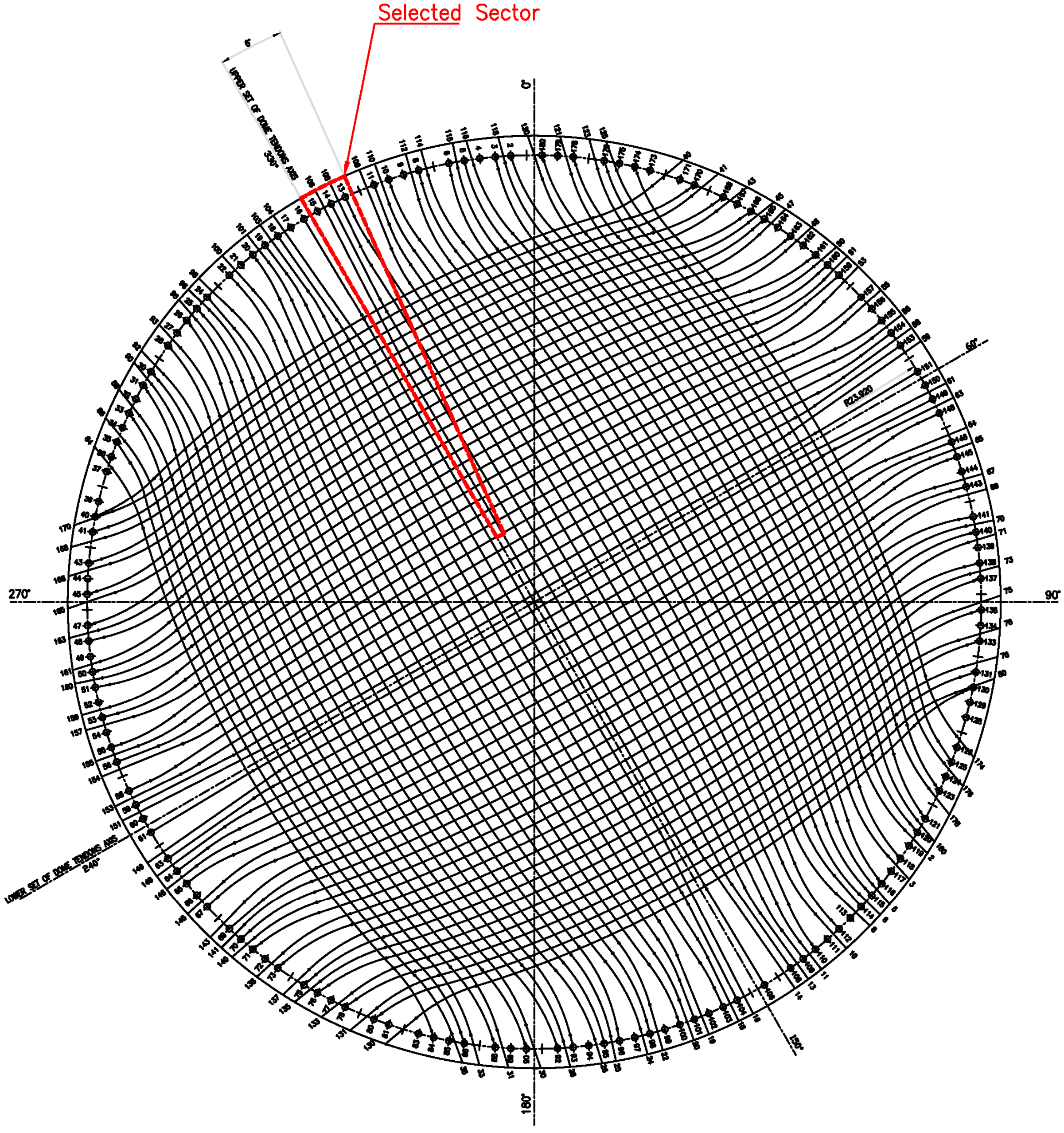
The RCB is the only structure expected to develop a significant thermal gradient across its thickness. Thermal loading was not considered for the RBIS, Emergency Power Generating Building and Essential Service Water Cooling Tower and Pump Structure.

Thermal moment reduction factor is used for design throughout the RCB.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Figure 03.08.01-22-1—Location of the Selected Sectors in Containment Slice Model



Question 03.08.01-27:

FSAR Section 3.8.1.4.4 summarizes the finite element procedures used to model the thermal and pressure transients from LOCA events. AREVA is requested to address the items listed below related to this analysis:

1. FSAR Figure 3.8-22 provides the thermal transient that RCB experiences. With 5 linear elements through the thickness, the element size appears to be about .36m (in the thickness direction). The large thermal gradients illustrated in Figure 3.8-22 for times shortly after initiation of the event (660 seconds and 2hrs) occur over a distance of about .2m. Explain how the heat transfer model was validated for the mesh refinement used since a more refined mesh is often needed for the thermal portion of a thermal/structural analysis.
2. The physical variation of material properties with temperature should be accounted for in the thermal analysis. FSAR Table 3.8-2 lists one value of elastic modulus, presumably at room temperature. Concrete properties vary with temperature and this can be an important factor to consider. Explain whether temperature dependent material property changes were included in the LOCA transient analyses. If not, justify why they were not.
3. FSAR Section 3.8.1.4.4, paragraph 3, states that "additional internal pressure was added to the RCB due to the heating of the liner plate." Explain how this additional pressure was determined and applied to the finite element model.
4. FSAR Section 3.8.1.4.5 discusses the modeling of concrete cracking during accident thermal loading. Explain whether the ANSYS smeared concrete cracking constitutive models were used for this purpose. If so, describe how these were applied. If not, clarify how the modeling of concrete cracking was accomplished.

Response to Question 03.08.01-27:

1. A six degree slice of the containment is studied for mesh refinement in consideration with thermal moment calculation. Based on the study with slice of containment, the existing 4/5 layers of mesh through the thickness of the Reactor Containment Building (RCB) overestimates the thermal gradient across the thickness at the beginning of the accident period and estimates thermal gradient well at the later period of accident compared to thermal gradient for a refined mesh. This overestimation of thermal gradient conservatively calculates the thermal moment.

2. In the loss of coolant accident (LOCA) transient analysis of the RCB, only the temperature-dependent material properties for the containment liner were considered. Temperature-dependent material properties for the concrete were not considered in the LOCA transient analysis of the RCB. The modulus of elasticity of concrete decreases with an increase in temperature. A parametric study was performed with an axisymmetric model of the RCB to compare changes in forces and moments corresponding to changes in mechanical properties of the RCB concrete. The study concluded that reduction in modulus of elasticity decreases the thermal moments whereas design forces and moments due to pressure loads do not change significantly.

3. For the purpose of calculating additional pressure due to expansion of the liner under accident temperature loads, a separate containment liner model is created. The containment liner model is created from the structural model of containment through separation of the liner elements (SHELL181) from the concrete and adding contact elements

(surface to surface) between the liner elements and containment concrete wall elements. The liner is provided with temperature dependent bilinear elastic-plastic material properties (ASTM A516). Containment temperatures at critical time points during accident condition are considered as temperature loads applied on the interior face of the liner. Therefore, the liner tends to expand and the containment wall restricts liner plate expansion. Pressure exerted by the liner on containment concrete due to restricted thermal expansion is calculated from contact pressure distribution in the contact elements. Based on contact pressure distribution, additional liner pressures are applied at critical time points during the bounding accident state.

This additional liner pressure is applied to the RCB wall and dome surfaces along with accident temperature distributions for the critical time points. Liner pressure is applied to the containment wall and dome as normal pressure using SURF154 elements.

4. ANSYS smeared concrete cracking constitutive models are used to model concrete cracking during thermal loading.

   A pseudo-axisymmetric nonlinear model is developed with SOLID65 elements allowing concrete cracking. This model represents a containment slice of 6^{o} width, away from discontinuities along with explicit representation of tendons. This slice model, when converted as linear model, can reproduce the results of the 360^{o} RCB model for thermal and tendon loading. In this nonlinear slice model, the RCB basemat, walls and dome are divided into multiple layers (based on the distribution of reinforcement) with different volumetric ratios of reinforcement. The presence of reinforcement on containment section is modeled with smeared reinforcement with SOLID65 elements. The SOLID65 elements are provided with concrete constitutive materials which allows cracking of concrete under tension.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.08.02-5:

Under the acceptance criteria of SRP 3.8.2, the computer codes used for design and analysis should be described and validated by procedures or criteria in Subsection II.4.e of SRP 3.8.1. In FSAR Section 3.8.2.4, describe the methods of analysis that are used to qualify the ASME III, Division 1, Subsection NE components covered in FSAR Section 3.8.2, including a description of the computer codes and their validation basis.

Also identify the detailed reports/calculations for the NE components that will be available for audit by the staff.

Response to Question 03.08.02-5:

The ASME BPV Code, Section III, Division 1, Subsection NE components addressed in U.S. EPR FSAR Tier 2, Section 3.8.2 are procured and are dependent upon vendor selection. The design and analysis of these items will be in accordance with ASME BPV Code, Section III, Division 1, Article NE-3000 as stated in U.S. EPR FSAR Tier 2, Section 3.8.2.4. The design and analysis will include a description and validation of the computer codes used.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.08.02-6:

GDC 16 requires that reactor containment and associated systems shall be provided to establish an essentially leak tight barrier against the uncontrolled release of radioactivity. FSAR Section 3.8.2.1.3 discusses electrical penetrations through the containment boundary. What qualification and testing will be done, or has been done, to assure that electrical penetrations will meet the requirements of GDC 16 and will withstand the pressure and temperature conditions under the design basis accident? Provide details of the electrical penetrations including any spares.

Response to Question 03.08.02-6:

Testing and inservice inspection requirements for steel containment components not backed by concrete are provided in U.S. EPR FSAR Tier 2, Section 3.8.2.7. As indicated in U.S. EPR FSAR Tier 2, Section 3.8.2, design, analysis, material selection, and testing of steel pressure retaining components complies to ASME BPV Code, Section III, Division 1, Subsection NE. Containment leakage rate testing is given in U.S. EPR FSAR Tier 2, Section 6.2.6.

Electrical penetrations are dependent upon vendor selection. The appended drawings (Figures 03.08.02-6-1 through 03.08.02-6-3) represent typical medium voltage, low voltage, and spare penetrations.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Figure 03.08.02-6-1—Medium Voltage Electrical Penetration

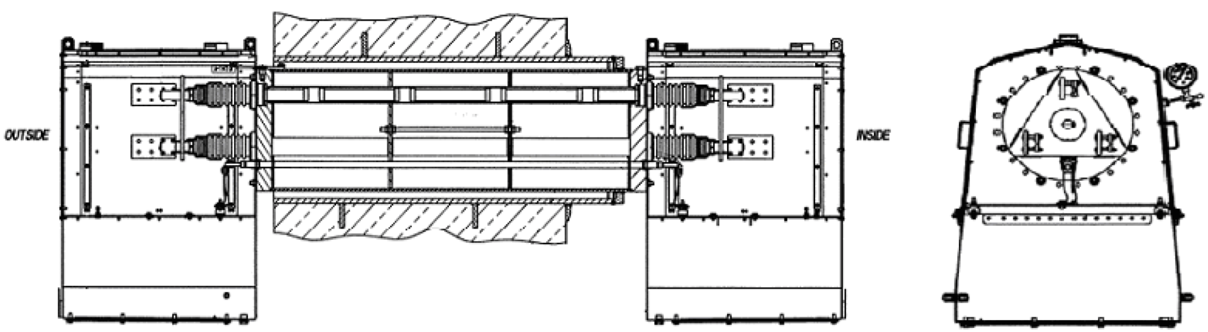


Figure 03.08.02-6-2—Low Voltage Electrical Penetration

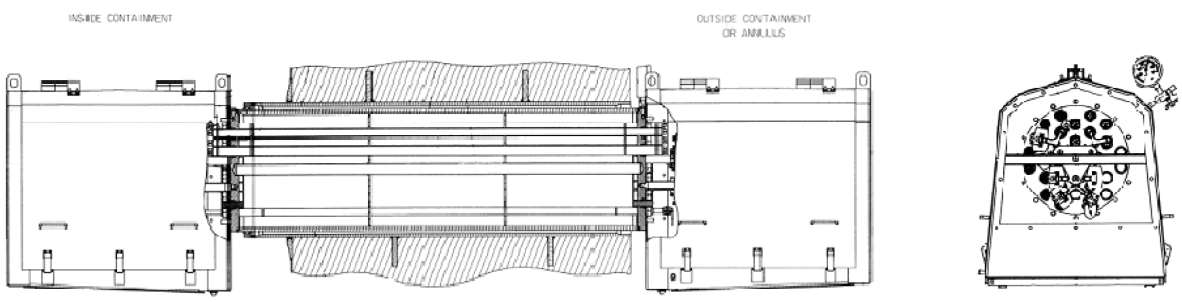
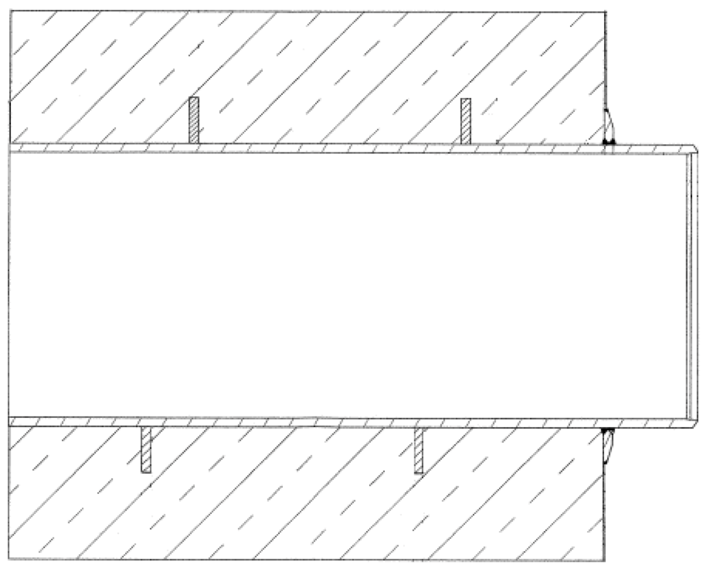


Figure 03.08.02-6-3—Medium Voltage Electrical Penetration Spare



Question 03.08.02-10:

FSAR Section 3.8.2.1.1 describes the equipment hatch, personnel air lock and emergency air lock as having doors with sealed double gaskets. Since the gaskets for the equipment hatch and air locks must assure a leak tight boundary during the design-basis LOCA event, describe the basis for qualification of these seals under the design-basis LOCA pressure and temperature conditions.

Response to Question 03.08.02-10:

The equipment hatch, personnel air lock, and emergency air lock are procurement items that are dependent upon vendor selection. The vendor will design, fabricate, and perform shop tests for each component. U.S. EPR FSAR Tier 2, Section 6.2.6 states that pre-operational and periodic Type B leakage rate tests are performed on the containment penetrations in accordance with 10 CFR 50 Appendix J. This section provides the maximum leakage rate requirements for the penetrations under the peak containment internal pressure associated with the design basis accident. The seals will therefore maintain a leak-tight boundary during the design-basis loss of coolant accident (LOCA) event if these testing criteria are met.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.08.03-3:

FSAR Section 3.8.3.2 as well as Sections 3.8.4.4.1 and 3.8.5.4, indicate that ACI 349-01 with exceptions described in FSAR Section 3.8.4.4 and 3.8.4.5 is utilized for design and construction of reinforced concrete structures inside and outside containment. Currently, NRC Regulatory Guide 1.142 endorses the use of ACI 349-97 (with certain regulatory positions) for design of reinforced concrete members. Since ACI 349-01 is not endorsed by Regulatory Guide 1.142, the staff reviews the applicability of ACI 349-01 on a case-by-case basis. Some prior NPP designs have utilized ACI 349-01; however, the acceptance of this ACI standard was reviewed and accepted on a case-by-case basis considering the application of this standard to the particular plant and subject to certain limitations/exceptions. Therefore, AREVA is requested to provide the following:

1. Identify the differences between ACI 349-01 and ACI 349-97.
2. Which of these differences are as relaxations of the provisions in ACI 349-01.
3. The technical basis for the use of these relaxed provisions.
4. FSAR Sections 3.8.4.4 and 3.8.4.5 state that the design of concrete members is performed using the strength design methods described in ACI 349-2001, with the exception that the shear strength reduction factor of 0.85 is used as allowed in ACI 349-06. The staff notes that Section 9.3.2 of ACI 349-01 allows a shear strength reduction factor of 0.85 for shear. Explain what AREVA is proposing to do that is different by referring to ACI 349-06.

Response to Question 03.08.03-3:

1. RG 1.142 identifies that ACI 349-97 is principally based on ACI 318-95 (the code states that the format of this Code is based on the "Building Code Requirement for Reinforced Concrete (ACI 318-89) (Revised 1992)" and incorporates recent revisions of that standard, except for Chapter 12, which is based on ACI 318-95). ACI 318-95 is also the technical basis document for ACI 349-01 with the exception of ACI 349 Chapter 12. ACI 349-01 Chapter 12 is identical to ACI 349-97 Chapter 12. The format of ACI 349-01 incorporates format, section numbering and editorial revisions of ACI 318-95 that were not included in the 1997 edition.

   Comparison of ACI 349-97 and ACI 349-01 indicates that changes impose more prescriptive language (in some instances more conservatism is incorporated), clarify provisions, are editorial in nature, or are based on additional development of concepts due to research, thus not creating technical issues.

   Significant differences between ACI 349-97 and ACI 349-01 are in:

   - Chapter 11 – Shear and Torsion
     - 11.6 – Design for Torsion
   - Chapter 13 – Two Way Slab Systems
   - Chapter 14 – Walls
   - Chapter 15 – Footings
   - Chapter 16 – Precast Concrete

Detailed comparison results are provided in a proprietary AREVA document, which is available for NRC inspection.

2. The principal change accomplished with ACI 349-2001 edition is alignment with ACI 318. Design approaches and methodologies remain substantially unchanged, with the exception of the sway frame approach to design. The ACI 349-2001 sway frame approach to design removes excess conservatism, which in certain conditions would allow for a reduction in reinforcement.

3. The use of second-order analyses (ACI 349-2001, Section 10.10) is an improvement for dealing with braced and sway frames. These provisions set limits on the use of refined second-order analysis, use of elastic analyses and moment magnifier approach. For sway frames, the use of second-order analyses will generally result in a more economical design. However, the results of a second-order analysis give more realistic values of the moments than those obtained from an approximate analysis using magnified moment.

   Changes in the magnified moments section include using the moment magnifier concept to account for slenderness effects. The moments at the column ends are computed using an elastic first-order frame analysis taking into account cracked regions along the length of the members. While the provisions for radius of gyration, unsupported and effective lengths of compression members are maintained, provisions for distinguishing sway from non-sway frames are introduced and designs for non-sway and sway frames are separated.

   ACI 349-2001 design for sway frames is revised to include three steps: calculate magnified sway moments, add magnified sway moments and unmagnified non-sway moments. and check for moments at points between column ends that may be larger than those at the end for slender columns with high axial load. The moment magnifier method for sway frames is a significant improvement over the reduction factor method for long columns specified in ACI 349-1997 and provides a good approximation of the actual magnified moments at the ends of the columns in sway frame.

   Improvements in the design process better define the actual load transfer process in a structure, thus allowing the removal of excess conservatism in ACI 349-1997. ACI 349-2001 yields less steel reinforcement material for this design but the analyzed stress ratios and implied safety factors remain functionally unchanged from ACI 349-1997. Detailed results of this comparison are available for NRC inspection.

4. According to ACI 349-01, Section 9.3.2.3, the strength reduction factor for shear and torsion should be φ=0.85. However, ACI 349-01 has an additional requirement in Section 9.3.4, which is based on ACI 318, to use shear strength reduction factor of 0.6 for any structural member if its nominal shear strength is less than the shear corresponding to the development of the nominal flexural strength of the member. ACI 349-06 removes the ACI 349-01, Section 9.3.4 requirement, which is based on the lower ductility of shear-critical members. The explanation for removal of this extra requirement is based on the fact that structural members in safety-related nuclear structures are analyzed elastically and detailed for inelastic response. The inelastic deformation demands on members of nuclear structures are not as great as those in conventional building structures which are typically analyzed with consideration given to inelastic behavior. Therefore, loss of strength and stiffness due to cyclic inelastic loading in structural members of nuclear structures will be

smaller than those in conventional building structures. The strength-reduction factors that ACI 349-06 provides are associated with the loading combinations listed in Section 9.2.1 of the code, which are different than the combination in the earlier version of the code. However, ACI 349-06, Appendix C "Alternative Load and Strength-Reduction Factors" allows structural concrete design with the load combinations and strength reduction factors provided in Sections C.2.1 and C.3.2, respectively. The load combinations listed in Section C.2.1 match the combinations listed in 9.2.1 of ACI 349-01, except for the addition of moving crane load where applicable. The shear and torsion strength reduction factor for use with these combinations is $\varphi=0.85$. Therefore, a shear strength reduction factor of 0.85 is used. No additional reduction of this factor is needed as required by Section 9.3.4 of ACI 349-01, based on the reasons given in ACI 349-06 for the removal of this additional requirement.

The design of anchorage to concrete will be performed in accordance with Appendix D of ACI 349-06. The U.S. EPR FSAR will be revised to use ACI 349-06, Appendix D instead of ACI 349-01, Appendix B. Since NRC currently endorses Appendix B of ACI 349-01 in RG 1.199 (with exceptions), reconciliation of ACI 349-06, Appendix D is provided in a proprietary AREVA document which is available for NRC inspection.

FSAR Impact:

U.S. EPR FSAR, Tier 2, Sections 3.8.1.2.1, 3.8.1.4.10, 3.8.3.2.1, 3.8.3.2.3, 3.8.3.4, 3.8.3.4.2, 3.8.3.5, 3.8.4.2.1, 3.8.4.2.3, 3.8.4.4.1, 3.8.4.5 and 3.8.6 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.03-6:

FSAR Sections 3.8.3.4.1 and 3.8.4.4.1 describe the overall analysis and design of containment internal structures and other Category I structures, respectively. AREVA is requested to address the following items related to the analysis and design criteria in this FSAR section:

1. This FSAR section states that "For steel members, thermal loads may be neglected when it can be shown that they are secondary and self limiting in nature." Provide the technical basis for this statement or revise the criteria to be consistent with the provisions in ANSI/AISC N690.
2. This FSAR section states that "For load combinations including loads Rrr, Rrj, and Rrm, the load combinations are first satisfied with these loads set to zero. However, when considering these concentrated loads, local section strength capacities may be exceeded under the effect of these concentrated loads, provided there is not a loss of function of any safety-related SSC." Provide the definition of loss of function for both concrete and steel structures. Also, confirm whether this means that the methodology and acceptance criteria for impactive loads are consistent with ANSI/AISC N690 for steel structures and ACI 349 (and RG 1.142, Rev. 2) for concrete structures.

Response to Question 03.08.03-6:

1. The statement "For steel members, thermal loads may be neglected when it can be shown that they are secondary and self limiting in nature" will be removed from U.S. EPR FSAR Tier 2, Sections 3.8.3.4.1 and 3.8.4.4.1. Secondary stresses resulting from thermal loads will be considered according to ANSI/AISC N690.

2. A local analysis and design of concrete members will be performed for impactive and impulsive loads according to ACI 349, with exceptions noted in RG 1.142. A local analysis and design of steel members will be performed for impactive and impulsive loads according to ANSI/AISC N690. For members of the Reactor Building internal structures (RBIS), this is stated in U.S. EPR FSAR Tier 2, Section 3.8.3.4.2. For other Seismic Category I structures, this is stated in U.S. EPR FSAR Tier 2, Section 3.8.4.4.1.

   In the evaluation of the response of reinforced concrete and steel structures subject to impactive or impulsive loads, it is acceptable to assume non-linear (elasto-plastic) response of the structural members. Deformation under impactive and impulsive loads is controlled by limiting the ductility ratio, μ_d, which is defined as the ratio of maximum acceptable displacement, χ_m (or maximum strain, ε_m), to the displacement at the effective yield point, χ_y (or yield strain, ε_y), of the structural member. In addition to the specified deformation limits, the maximum deformation shall not result in the loss of intended function of the structural member nor impair the safety-related function of other systems and components.

   In terms of structural capacity, the member will not lose the ability to perform its intended function as long as the ductility limits for concrete and steel members presented in U.S. EPR FSAR Tier 2, Table 3.5-3, are satisfied. However, the deformation limits of the member may be governed by structures, systems and components (SSC) attached to it; therefore, the member must also satisfy the deformation limits imposed upon it by the SSC so that a loss of function is not experienced.

The statement of concern will be revised as follows:

> "For load combinations including loads R_{rr}, R_{rj}, and R_{rm}, the load combinations are first satisfied with these loads set to zero. However, when considering these concentrated loads, local section strength capacities may be exceeded under the effect of these concentrated loads, provided there is not a loss of intended function of the structural member or a loss of function of any safety-related SSC."

FSAR Impact:

U.S. EPR FSAR Tier 2, Sections 3.8.3.4.1 and 3.8.4.4.1 will be revised as stated in the response and as shown on the enclosed markups.

Question 03.08.03-10:

FSAR Sections 3.8.3.4.4, 3.8.4.4, and 3.8.5.4.1 indicate that the seismic loads from the three components of the earthquake are combined using the SRSS method or the 100-40-40 percent rule described in ASCE 4-98. The staff has noted from past experience that the application of the 100-40-40 method may not always give results consistent with the guidance provided in Regulatory Guide 1.92, Rev. 2. If the FSAR is not revised to use the 100-40-40 method defined in RG 1.92, Rev. 2, AREVA is requested to provide the technical basis which demonstrates the adequacy of the 100-40-40 method taken from ASCE 4-98. This should include a quantitative demonstration, using the set of member forces for critical concrete element(s) that govern the design and where seismic loads are significant, which shows that the results from the 100-40-40 method are the same or more conservative than the results using the RG 1.92, Rev. 2 method.

Response to Question 03.08.03-10:

The 100-40-40 method described in ASCE 4-98 is mathematically equivalent to the 100-40-40 method described in RG 1.92, Rev. 2. The U.S. EPR FSAR will be revised as indicated in the attached markups to clarify any ambiguity regarding the 100-40-40 method.

We will replace the description of the 100-40-40 percent rule, which appears in U.S. EPR FSAR Tier 2, Sections 3.8.3.4.4 and 3.8.4.4.1, with the following:

> "Let R_1, R_2, R_3, be the maximum responses of an SSC caused by each of the three earthquake components calculated separately. The maximum seismic response attributable to earthquake loading in three orthogonal directions shall be evaluated as:
>
> $R = \pm 1.0R_1 \pm 0.4R_2 \pm 0.4R_3$,
>
> $R = \pm 0.4R_1 \pm 1.0R_2 \pm 0.4R_3$, or
>
> $R = \pm 0.4R_1 \pm 0.4R_2 \pm 1.0R_3$,
>
> whichever is greatest."

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.8.3.4.4 and Section 3.8.4.4.1 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.03-11:

FSAR Section 3.8.3.1.8 provides a brief description of the polar crane support structure and FSAR Section 3.8.3.4.4 provides a description of the development of polar crane seismic loads. Since these descriptions are presented in FSAR Section 3.8.3, provide the following information:

1. Explain what structural members are considered to be within the scope of containment internal structures. Provide a detail showing the boundary of these structural members and the crane assembly, and the jurisdictional boundary between these structural members and the RCB.
2. Describe the analysis methods including computer codes that were used to analyze and design these intervening structural members between the polar crane assembly and the RCB wall.
3. Provide the materials and design codes that were used for the crane girder and the intervening structural members.

Response to Question 03.08.03-11:

1. The polar crane support brackets (steel) are included in the design of the containment vessel. The boundary between the brackets and the crane is the location at which the crane runway support system (girders) attaches to the brackets. A detail showing this interface (boundary) is provided in Figure 03.08.03-11-1.

   The polar crane brackets are included as part of the containment ANSYS model. This is done to provide locations for transfer of the loads from the polar crane into the containment structure.

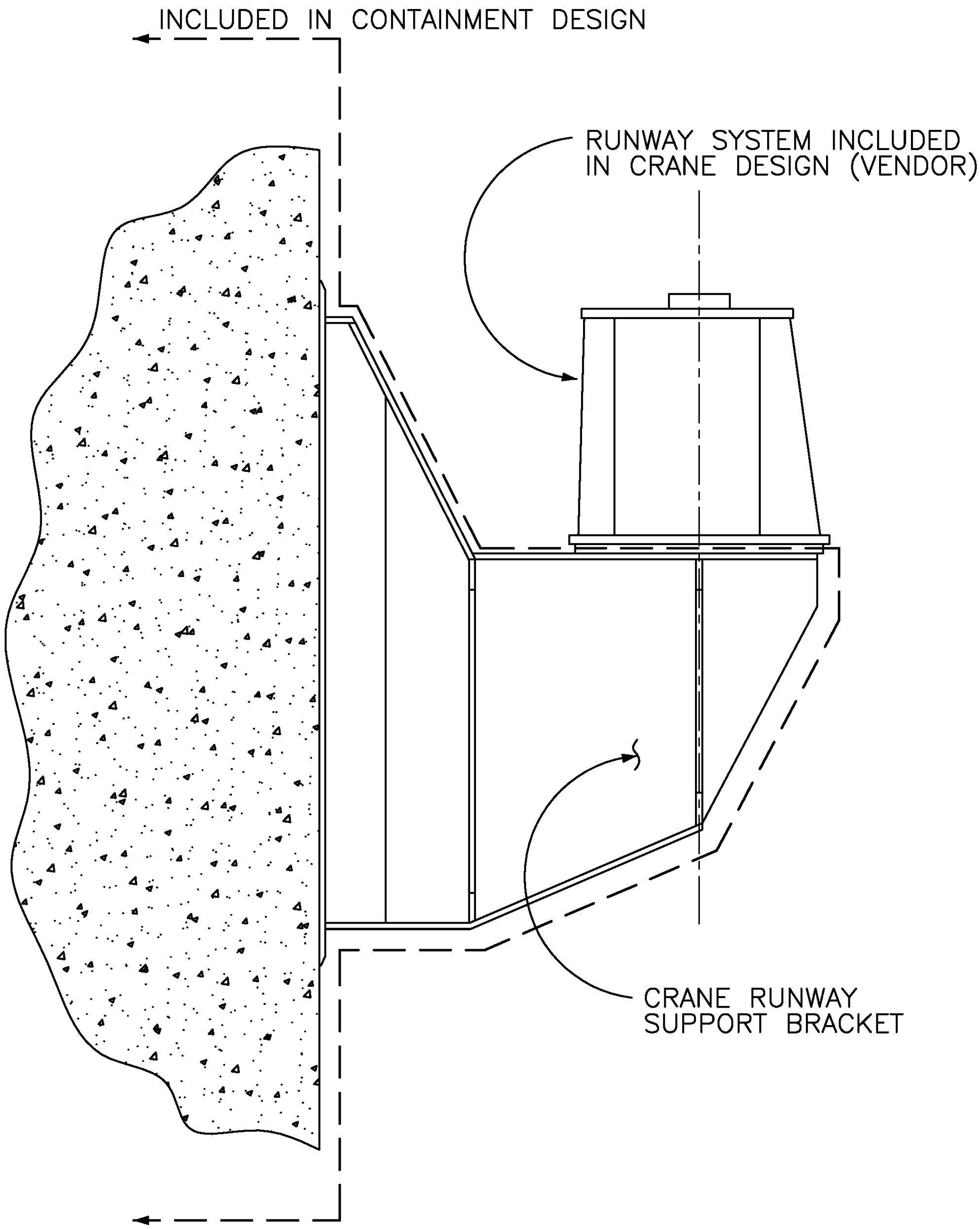
2. The polar crane runway system design and the loading condition considered in the structural design are based upon an assumed crane size and accommodate the worst-case in accordance with code requirements. The current intervening structural member is based upon the European design for a similar crane. Reactions used on polar crane brackets in containment analysis will be validated once final loads from the polar crane and polar crane runway system are received from the polar crane manufacturer. Based on this information, bracket design will be finalized.

3. The crane girder and intervening structural steel members will be designed in accordance with the requirements for design and materials specified in AISC N690.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Figure 03.08.03-11-1—Boundary Between Containment Structure and Crane Assembly



Question 03.08.03-12:

Table 3.8-8 provides materials for structural steel shapes and plates used for design of steel members for containment internal structures and other seismic Category I structures addressed in FSAR Sections 3.8.3 through 3.8.5. Provide the information requested below related to the steel materials:

1. Steel materials ASTM A333, A537, and A633 are not listed as accepted materials under ANSI/AISC N690, including Supplement No. 2. Provide the technical basis for the use of these materials or revise the FSAR to be consistent with the ANSI/AISC Standard.
2. The actual material specifications, along with their procurement and supplemental requirements are not identified. The materials specifications, along with procurement and supplemental requirements, for the actual steel structural materials to be used should be provided.

Response to Question 03.08.03-12:

1. NUREG-0554 provides guidance that cranes are to be designed in accordance with ASME standards. Table 3.8-8, Material for Structural Steel Shapes and Plates, lists materials used in these applications, including those used in construction of the crane and its support system. Steel materials ASTM A333, A537 and A633 are listed as acceptable in ASME NOG-1-2004, “Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder)”, and ASME NUM-1-2004, “Rules for Construction of Cranes, Monorails, and Hoists (With Bridge or Trolley or Hoist of the Underhung Type)”. These materials are commonly used by the manufacturers of cranes similar to those that may be provided for the U.S. EPR. These standards are cited as references for use in the design and fabrication of the cranes in U.S. EPR FSAR Tier 2, Section 9.1.5.

2. The material specifications, along with the procurement and supplemental requirements for the structural steel materials will be developed later in the design process.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.08.04-3:

FSAR Section 3.8.4.3.1 defines loads on other Seismic Category I structures in accordance with ACI 349-2001 and RG 1.142, Revision 2, November 2001 for concrete structures, and in accordance with ANSI/AISC N690-1994, including Supplement 2 (2004) for steel structures. Provide the following additional information to clarify certain assumptions in defining loads used in the design:

1. Provide the basis for selecting a live load of 100 psf applied to concrete floors and to steel grating floors and platforms in Seismic Category I structures other than the FB. Also explain the basis for the live load of 400 psf applied to FB concrete floors, as well as RB internal structures as discussed in FSAR Section 3.8.3.3.1. Furthermore, explain how it is ensured that these live load limits are not exceeded.
2. For buried items, the live load includes the effects of surface traffic such as truck loads, rail loads, construction equipment, and construction or maintenance activities. Provide the live load to be used for buried items.
3. Provide justification for assuming a ground temperature of 50F.
4. FSAR Section 3.8.4 indicates that the evaluation of structures resulting from external hazards of aircraft, explosion, and missile loading, are considered as part of the plant safeguards and security measures. However, no discussion is given about the external hazards of aircraft hazard, explosion, and missile loading required for the design of the plant structures as described in SRP 3.8.4. FSAR Sections 3.5.1.5 and 3.5.1.6 indicate that the COL applicant will evaluate the effects of aircraft hazard, explosion, and missile loading applicable to the specific site. Therefore, provide in FSAR Section 3.8.4 a description of these external hazard loadings and the need by the COL applicant to evaluate the effects of these loadings on plant structures.
5. The AREVA response to RAI 93 Supplement 1, entitled “Response to Request for Additional Information No. 93 Supplement 1 (1085), Revision 0,” dated 10/9/2008, related to FSAR Section 2.3.1 – Regional Climatology, provided a proposed revision to FSAR Section 3.8.4 on the subject of live load due to rain, snow, and ice. The proposed revision indicates that the design live load due to rain, snow, and ice is based on 100 psf on the ground, as described in FSAR Section 2.4. This value is postulated as a meteorological site parameter for the extreme winter precipitation load and includes the weight of the normal winter precipitation event and the weight of the extreme winter precipitation event. Roof snow and ice loads are determined using Chapter 7 of ASCE/SEI 7-05, “Minimum Design Loads for Buildings and Other Structures.” From this description it is not clear what the calculated live load is for rain, snow, and ice on the roof. Using the information given in FSAR Section 2.4, describe in FSAR Section 3.8.4 the magnitude of the calculated roof live loads for use in design for all Seismic Category I structures. Since the proposed wording in the RAI 93 response suggests that a 100 psf roof load is applicable for normal and extreme precipitation, explain how the single value of live load is utilized in the load combinations for concrete and steel roof structures. Also, explain how the calculation of the live load for roofs and its use in the load combinations compare to the current NRC interim staff guidance (ISG) entitled “Interim Staff Guidance on Assessment of Normal and Extreme Winter Precipitation Loads on the Roofs of Seismic Category I Structures,” available from the NRC web site.

Response to Question 03.08.04-3:

1. U.S. EPR FSAR Tier 2, Section 3.8.4.3.1, will be revised to state:

   “In general, a live load of 500 pounds/ft^2 is applied to FB concrete floors and a load of 175 pounds/ft^2 applied to FB and SB steel grating floors and platforms. A live load of 300 pounds/ft^2 is applied to SB concrete floors. Finally, a live load of 100 pounds/ft^2 is applied to concrete floors, steel grating floors, and platforms in other Seismic Category I structures.”

   U.S. EPR FSAR Tier 2, Section 3.8.3.3.1, will be revised to state:

   “In general a live load of 500 pounds per square foot is applied to RB internal structures concrete floors, and 175 pounds per square foot live load is applied to steel grating floors and platforms.

   The floor live loads applied to Seismic Category I structures (other than containment) are as follows:

	Concrete Floors	Steel Platforms
Reactor Building Internal Structures (RBIS) -	500 psf	175 psf
Fuel Building (FB) -	500 psf	175 psf
Safeguard Buildings (SB) -	300 psf	175 psf
Essential Service Water Buildings (ESWB) -	100 psf	100 psf
Emergency Power Generation Buildings (EPGB) -	100 psf	100 psf

   For all buildings on the Nuclear Island, concrete floor live loads were formulated based on experience from upgrades at existing plants as well as the expected live loads for the U.S. EPR. In addition, the 500 pounds per square foot (psf) live load in Reactor Building internal structures (RBIS) and the Fuel Building (FB) is based on the anticipated high loads during a shutdown condition. The loads in the Safeguard Buildings (SB) are not anticipated to be as severe as the loads in the Reactor Building (RB). However, the 300 psf live load compares favorably with similar occupancy live loads from ASCE 7-05. The occupancy levels and expected loading conditions in the Essential Service Water Building (ESWB) and Emergency Power Generating Buildings (EPGB) are low enough that the 100 psf live load is conservative. The live load for steel platforms in Seismic Category I structures exceeds or matches the minimum live loads given in ASCE 7-05.

   The control associated with preventing the actual live loading from exceeding design values is an operational responsibility and will be included in the appropriate operating procedures and/or design drawings.

2. The load for buried items is computed using Boussinesq's equation:

$$P_p = 0.48 \frac{P_s}{H^2\left[1+\left(\frac{d}{H}\right)^2\right]^{2.5}}$$

Where
P_p = surface load transmitted to the buried item
d = offset distance from the surface load to buried item
H = thickness of soil cover above the item
P_s = concentrated surface load

The calculated value of P_p is multiplied by an impact factor which can be found in Table 4.1-2 of *Guidelines for the Design of Buried Steel Pipe*, American Lifelines Alliance, 2001. For the U.S. EPR, specific live loads for trucks are based on the AASHTO H20 and HS20 trucks. Rail loads are based on the Cooper E80 railroad loads but may be controlled by anticipated shipping weights. Recommended surface loads transmitted to the buried pipe (P_p) for the H20 truck and E80 railway loads can be found in Table 4.1-1 of *Guidelines for the Design of Buried Steel Pipe*.

For the U.S. EPR, the load for buried items is considered a soil load (H). The same load factors are applied for live load and soil load in all applicable load combinations so the net effects of load for buried items are the same.

3. Climate data and maps available from the National Climatic Data Center show an average air temperature that ranges from approximately 40°F to 70°F for the central and eastern portions of the U.S. Given this range, a ground temperature of 50°F was selected to account for the average temperatures.

4. A description of loads associated with potential aircraft hazard, explosion hazards and missile hazards is provided U.S. EPR FSAR Tier 2, Section 3.8.4.3.1, under Other Loads. These subjects are also addressed in U.S. EPR FSAR Sections 2.2 and 3.5. COL Items requiring site-specific evaluation of these potential loading conditions are provided in U.S. EPR Table 1.8-2, U.S. EPR Combined License Information Items, Items 2.2-1, 2.2-2, and 3.5-6.

5. As stated in the Response to RAI 93, Question 02.03.01-12, there are no parapets on the roof of any Seismic Category I structure. This makes the contribution of rain to the calculated roof live load negligible. The calculated portion of roof live load due to frozen extreme winter precipitation is 30 psf. The calculated live load due to normal winter precipitation is 70 psf. When the normal and extreme components are combined the total roof live load due to winter precipitation is 100 psf. The single value of 100 psf live load is used in both normal live loading combinations and in extreme live loading combinations. The calculated normal roof precipitation load of 70 psf could be used for normal live loading cases but is not for the purposes of being conservative.

   For the normal precipitation event, items (1), (2), (3), and (4) on page 4 of the ISG are not directly applicable as they are site-specific conditions. However, using the conversion algorithms on page 5 of the ISG, it can be determined that a historical maximum snowpack depth of 75.5 inches would be equivalent to a load of 100 psf. In addition, a snow depth of 128 inches due to the 100-year return period snowfall event or the historical maximum snowfall event would be equivalent to a load of 100 psf. After reviewing NOAA information, these values would be exceeded in very limited areas of the U.S. The Response to RAI 93, Question 02.03.01-12, stated that the 100 year snowpack value of 100 psf would be exceeded in limited areas of the U.S. and therefore the normal winter precipitation event load of 100 psf is conservative for most locations in the central and eastern U.S. when compared with the ISG. The extreme winter precipitation events also compare favorably to items “b” and “c” on page 6 of the ISG. The specific details and methodology of determining the extreme winter precipitation loads were addressed in the Response to RAI 93, Question 02.03.01-12.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.8.3.3.1 and Section 3.8.4.3.1 will be revised as described in the response and indicated on the enclosed markup

Question 03.08.04-4:

FSAR Section 3.8.4.4.2 states that gaps are maintained between structures adjacent to Seismic Category I structures to allow for structural movements during seismic events, containment pressurization, missile strikes, aircraft impact, explosions, and other loading conditions. In addition, exterior walls and roofs of the hardened SBs 2 and 3, RSB, and the FB are modeled to be independent of the internal structures, because there is no physical connection of internal walls and slabs in these structures with the outside walls and roof. Provide the following additional information on the gaps between the structures:

1. Specify the dimensions of the gaps to be provided between all structures adjacent to Seismic Category I structures and compare them to the calculated building responses.
2. Specify the dimensions of the gaps to be provided between the hardened structures noted above and the internal structures. Also, compare them to the calculated structural responses.

Response to Question 03.08.04-4:

1. The designed gap sizes between the structures adjacent to Seismic Category I nuclear island structures are as follows:
   - 18 inches – Full separation gap between Adjacent Nuclear Auxiliary Building and Fuel Building Shield Structure;
   - 18 inches – Full separation gap between Adjacent Nuclear Auxiliary Building and Safeguard Building 4;
   - 12 inches – Full separation gap between Adjacent Access Building and Safeguard Building 3 Shield Structure;
   - 16 inches – Full separation gap between Adjacent Access Building and Safeguard Building 4.

   The evaluated structural response for the different load cases/events are as follow:

   - Load Case – Commercial Aircraft Impact: The adjacent reinforced-concrete Nuclear Auxiliary Building and Access Building are considered as an additional protection for the direct aircraft impact on the Nuclear Island in the Aircraft Impact Screening evaluation in accordance with: "*Methodology for Performing Aircraft Impact Assessments for New Plant Designs*," NEI 07-13, Final Draft Rev 6, Nuclear Energy Institute, prepared by ERIN Engineering & Research, Walnut Creek, CA, August, 2008. Therefore, no direct calculation for the structural response/displacement of these buildings is performed.
   - Load Case – Seismic: Time history analyses predict a maximum interaction distance of 11.2 inches between adjacent structures. This value takes into account maximum tilt due to differential settlement.
   - All other Load Cases: Maximum interaction distance between adjacent structures for all other load cases are enveloped by the seismic load case.
2. The gap sizes between the hardened structures (shield structures) and the internal Nuclear Island structures are as follows:
   - 0.6 m ≈ 23.62 inches – Separation gap between all shield and internal structure walls and roofs in Fuel Building and Safeguard Buildings 2 &3;

- 1.8 m ≈ 70.87 inches – Separation gap between shield wall and containment wall in cylindrical part;
- 2.0 m ≈ 78.74 inches – Separation gap between shield dome and containment dome in spherical part;
- 0.6 m ≈ 23.62 inches – Minimum (nearest) separation gap between shield structure and dome ring belt in nearest place.

The calculated structural response for the different load cases/events are as follows:

- Load Case – Commercial Aircraft Impact: The calculated maximum displacements are smaller than the separation gap between hardened and internal structures.
- Load Case – Seismic: The maximum seismic interaction distance between hardened structures is calculated to have an upper bound of 1.78 inches. Calculated relative displacements between the shield and internal structures due to seismic events are much smaller than the above specified gaps because
  - i. The shield and internal structures have a closed/boxed shape with the common basemat.
  - ii. The structural response due to seismic remains in quasi-elastic stage compared to post-ultimate plastic response due to aircraft impact.
- All other Load Cases: The structural responses/displacements are smaller compared to the above specified load cases.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.08.04-5:

FSAR Sections 3.8.4.1.2 and 3.8.4.4.2 state that FSAR Section 9.1.2 addresses fuel storage racks. FSAR Section 9.1.2 states that the design of the spent fuel storage racks are the responsibility of the COL applicant and that the COL applicant will provide a summary of the structural dynamic and stress analyses associated with fuel racks. Describe whether the spent fuel racks will be free standing or anchored to the fuel pool. In either case, describe the analysis and procedures for the spent fuel pool and racks, and explain how they compare to the criteria in Appendix D to SRP Section 3.8.4, “Guidance on Spent Fuel Pool Racks.” This description should include an explanation of how the loads from the fuel racks are included in the design of the spent fuel pool. This description of the analysis and design approach for the spent fuel pool and racks should be presented in the FSAR.

Response to Question 03.08.04-5:

The fuel rack design does consider the racks as free-standing and the rack design is addressed in U.S. EPR FSAR Tier 2, Section 9.1.2, with supplemental information provided within the Responses to RAI 84.

The Fuel Building (FB) is located on the common Nuclear Island basemat and is designed as part of the common finite element model (FEM) analyses. The loading conditions considered in the analyses were provided in the Response to RAI 155, Supplement 2, Question 03.08.01-7. The critical sections within the FB will be provided as identified in the deterministic Critical Section Selection Criteria and the resulting sections will be provided in the Response to RAI 155, Question 03.08.04-6.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.08.05-2:

FSAR Section 3.8.5.1.1 states that the connection of the tendon gallery to the NI Common Basemat Structure foundation basemat allows for differential movement between the concrete structures. Discuss how this connection will be designed and provide a figure showing the details of this connection. Also discuss how the tendon gallery, including the above connection, will be designed to prevent water infiltration into the tendon gallery space. An accumulation of water into this space could lead to corrosion of the tendon anchorages and inhibit inspection procedures.

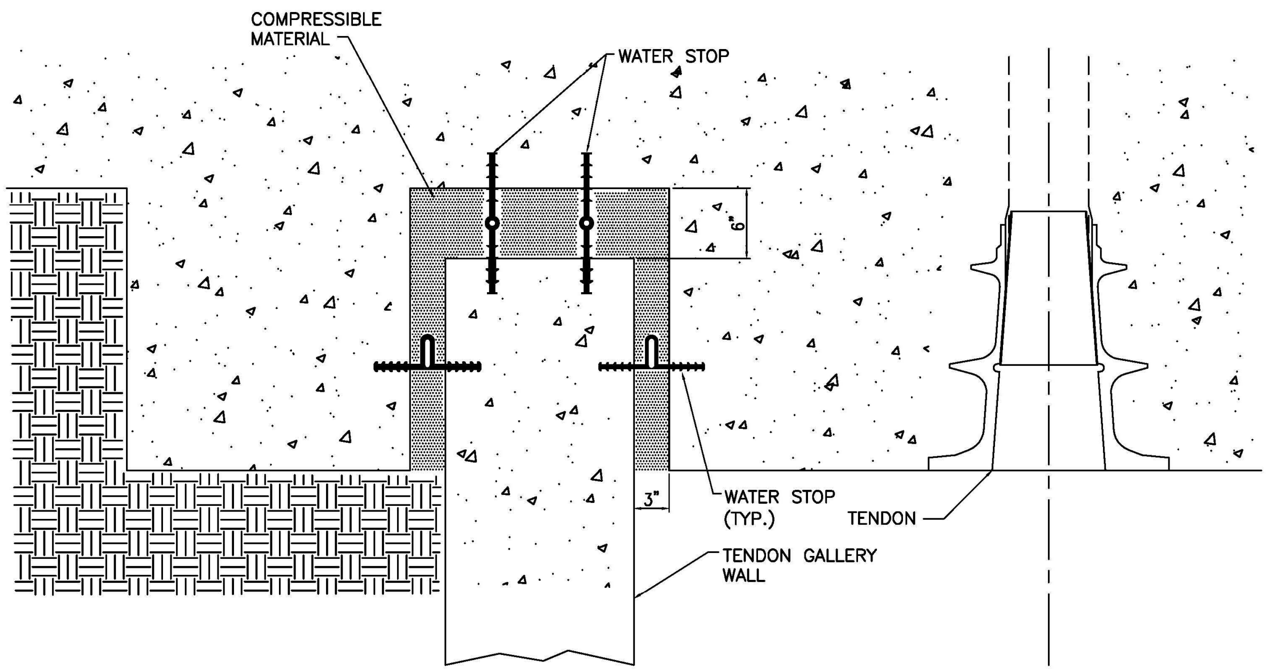
Response to Question 03.08.05-2:

The tendon gallery walls are in close proximity to the Nuclear Island (NI) basemat, allowing movement of the NI structure without immediate transfer of load. Consideration has also been given to the structural design of the tendon gallery to tolerate differential movement that may occur over its circumferential length. Groundwater infiltration into this annular space would be a maintenance issue and the use of waterstops and water proofing of below grade structures will be incorporated as described in U.S. EPR FSAR Tier 2, Section 3.4.2, to minimize maintenance and to mitigate the affects of groundwater intrusion into the tendon gallery. A conceptual detail of this joint is shown in Figure 03.08.05-2-1.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Figure 03.08.05-2-1— Conceptual Tendon Gallery Waterstop Detail



Question 03.08.05-6:

In FSAR Section 3.8.5.4.2, an equation is provided for determining spring constants used to represent the soil that provides support for the foundation basemat in the ANSYS FEM model. AREVA is requested to provide the following additional information regarding the development of the soil springs used in the model:

1. Provide the source and justification for the use of this equation. As the plan view of the foundation mat cannot be quantified as a simple shape, explain how the constants A and B used in this equation and tabulated in FSAR Table 3.8-13 were determined. Discuss any variations considered in the properties of the subgrade modulus in determining the values of the spring constants.
2. The FSAR states that the Gazetas equation was used to evaluate the total soil spring (K_o) for the foundation of the common basemat NI structure. It further states that although Gazetas addresses the dynamic stiffness of the foundation basemat, the use of one-half the dynamic shear modulus in the equation approximates the total stiffness of the supporting soil medium under static conditions. Provide the justification for this approximation and state why the Gazetas equation is acceptable for determining K_o.
3. FSAR Figure 3.8-106 does not appear to provide the elastic displacement for soil case 1u. This information should be provided similar to Figures 3.8-107 through 115.

Response to Question 03.08.05-6:

1. For an appropriate representation of the subgrade modulus, the distribution of the stiffness under the foundation mat should reflect the anticipated pressure distribution. An elliptical distribution of the subgrade modulus represented by the equation provided in U.S. EPR FSAR Tier 2, Section 3.8.5.4.2, approximates the theoretical distribution of the bearing pressure under a relatively rigid mat foundation supported on an elastic medium.

   The elliptical distribution for the Winkler soil springs is defined by coefficients A and B, which are developed by an iterative process. The iterative process uses the actual foundation geometry (ANSYS Model) to calculate the bearing pressure and the distribution, which is subsequently used in the process to evaluate settlements and the Winkler springs. This process is repeated iteratively until the total stiffness is consistent with the area of the foundation, and the resulting distribution of the bearing pressure is consistent with soil settlements.

   Different soil cases are considered in the analysis and a corresponding subgrade modulus for the stiffness expression is calculated for each case.

2. Static loading of soils/rock are associated with large strains relative to the strains expected during the design seismic event. Typically, the static modulus may vary from about 0.2 to 0.5 of the dynamic modulus. The upper range of the modulus is selected because the resulting forces in the mat are somewhat conservative and the 50 percent reduction in shear modulus is consistent with the strain typically observed under static loading for most soils. The Gazetas equations are used since they are more suitable for inhomogeneous subsurface conditions assumed in the AREVA evaluation. Other methods for obtaining dynamic spring constant (Wong-Luco) are also evaluated and found to provide similar results.

3. The U.S. EPR FSAR Tier 2, Figure 3.8-106 will be revised.

FSAR Impact:

U.S. EPR FSAR Tier 2, Figure 3.8-106, will be revised as described in the response and as indicated on the enclosed markup

U.S. EPR Final Safety Analysis Report Markups

Section 3.8.2 contains a description of the penetrations through the containment liner, including the equipment hatch, airlocks, piping penetration sleeves, electrical penetration sleeves, and the fuel transfer tube penetration sleeve.

No load transfer attachments are used at the bottom portion of the liner plate to transfer loads from the concrete RB internal structures into the lower portion of the NI Common Basemat Structure foundation basemat. RB internal structure lateral reaction loads are transferred through the liner plate. This is achieved by lateral bearing on the haunch wall at the bottom of the RB internal structures foundation where it is embedded in concrete above the NI Common Basemat Structure foundation basemat.

Structural attachments to the containment walls and dome include various pipe, HVAC, electrical, and equipment support brackets, as well as the polar crane rail supports. The liner plate is continuously welded to embedded plate areas and areas with thickened plates so that a continuous leak-tight barrier is maintained.

3.8.1.2 Applicable Codes, Standards, and Specifications

The following codes, standards, specifications, design criteria, regulations, and regulatory guides are used in the design, fabrication, construction, testing, and in-service inspection of the RCB (GDC 1, GDC 2, GDC 4, GDC 16, and GDC 50).

3.8.1.2.1 Codes

- ACI 117-90/117R-90, Specification for Tolerances for Concrete Construction and Materials (Reference 6).
- ACI 301-05, Specifications for Structural Concrete for Buildings (Reference 7).
- ACI 304R-00, Guide for Measuring, Mixing, Transporting, and Placing Concrete (Reference 8).
- ACI 305.1-06, Specification for Hot-Weather Concreting (Reference 9).
- ACI 306.1-90, Standard Specification for Cold-Weather Concreting (Reference 10).
- ACI 347-04, Guide to Form Work for Concrete (Reference 11).
- ACI 349-01, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (exception described in Sections 3.8.4.4 and 3.8.4.5) (Reference 12).

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- ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of D.4.5(c) requires the use of Condition B even when supplemental reinforcement is provided (Reference 63).

Appendix 3E provides details of the design and reinforcement for the containment wall to foundation connection.

Appendix 3E provides details of the design and reinforcement for the containment cylinder wall and buttresses.

The following sections provide details of design and analysis of the RCB.

3.8.1.4.1 Computer Programs

The containment structure is included in an overall model developed for analysis of the NI Common Basemat Structure, which includes the RCB with the RB internal structures, the RSB, the SBs, the FB, and the NI Common Basemat Structure foundation basemat. The RCB is modeled and analyzed using the ANSYS computer program. ANSYS is a validated and verified, quality-controlled computer program that has been used for a number of years in the nuclear power industry. Refer to Chapter 17 for a description of the quality assurance program for the U.S. EPR design certification.

The ANSYS model is used to analyze the RCB for the loads defined in Section 3.8.1.3.1. The results from these load case analyses are combined and factored using the loading combinations defined in Section 3.8.1.3.2. The design of the RCB shell wall and dome is generally controlled by load combinations containing the +62/-3 psig design internal pressure load and SSE seismic loads.

The overall NI Common Basemat Structure analysis is performed using the ANSYS finite element computer program. The RCB is modeled in combination with the other structures of the NI Common Basemat Structure and basemat using a mesh of finite elements. The element mesh for the RCB consists of the dome and cylindrical shell wall, which interconnects with the overall NI Common Basemat Structure foundation basemat. No other structures physically connect to the containment structure; therefore, the foundation basemat is the only interfacing structure in the model. Section 3.8.5 describes the modeling of the NI Common Basemat Structure foundation basemat.

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ANSYS SOLID45 solid elements are primarily used to model the RCB concrete dome and cylindrical shell wall. SOLID45 is a three-dimensional, eight-node brick element that is suitable for moderately thick shell structures. It can also provide out-of-plane shear forces and has an elastic-plastic capability. Four and f~~F~~ive layers of SOLID45 elements are used to model through the thickness of the dome and cylindrical shell wall,respectively~~ and dome~~. ANSYS SOLID95, a twenty node brick element, and ANSYS SOLID92, a ten node tetrahedral element, are also used to model the RCB. The buttresses, ring girder, and thickened areas around the base of the containment structure are included in the ANSYS model. Soft elements are used to represent the large openings for the equipment hatch, two airlocks, and construction opening.

effect on the overall design of the RCB and are not included in the overall computer model of containment.

Appendix 3E provides details of the design and reinforcement in the equipment hatch area.

Section 3.8.2 provides design details of the steel portion of containment penetrations.

3.8.1.4.10 Steel Liner Plate and Anchors

The design of the steel liner plate is in accordance with Subarticle CC-3600 of the ASME BPV Code, Section III, Division 2. The steel liner plate is not considered as a structural strength member when performing containment design basis analyses. The steel liner plate is designed to withstand the effects of imposed loads and to accommodate deformation of the concrete containment without jeopardizing leak-tight integrity (GDC 16). The steel liner plate is anchored to the concrete containment in a manner that does not preclude local flexural deformation between anchor points. Calculated strains and stresses for the steel liner plate do not exceed the values given in Table CC-3720-1 of the ASME BPV Code, Section III, Division 2. Strains associated with construction-related liner deformations may be excluded when calculating liner strains for service and factored load combinations as allowed by the code. The liner is anchored to the concrete containment around the outside perimeter of the sides of the embedded portion between elevation -25 feet, 7 inches and elevation -7 feet, 6.5 inches. Anchors are not provided on the inside surface of the liner. Overturning moments and sliding forces of the RB internal structures relative to the liner plate are resisted by the appropriate structural dead weight and lateral bearing.

The steel liner plate anchorage system is designed to accommodate design loads and deformations without loss of structural or leak-tight integrity (GDC 16). The steel liner plate anchorage system is designed so that a progressive failure of the anchorage system is prevented in the event of a defective or missing anchor. The steel liner plate is anchored to the concrete so that the liner strains do not exceed the strain allowable given in Paragraph CC-3720 of the ASME BPV Code, Section III, Division 2. The anchor size and spacing is designed so that the response of the steel liner plate is predictable for applicable loads and load combinations. The anchorage system is designed to accommodate the design in-plane shear loads and deformations exerted by the steel liner plate and normal loads applied to the liner surface. The allowable force and displacement capacity for the steel liner plate anchors does not exceed the values given in Table CC-3730-1 of the ASME BPV Code, Section III, Division 2. The load combinations specified in Section 3.8.1.3.2 are applicable to the steel liner plate anchors. Mechanical and displacement-limited loads are as defined in Subparagraph CC-3730(a) of the ASME BPV Code, Section III, Division 2. Concrete anchors are designed in accordance with ACI 349-06~~,~~ (Appendix ~~B~~D with exceptions stated in Section 3.8.1.2.1, “Codes”), and with the exceptions noted in RG 1.199.

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Steel liner plate penetration assemblies, including nozzles, reinforcing plates, and penetration anchors are designed to accommodate design loads and deformations without loss of structural or leak-tight integrity (GDC 16). Effects such as temperature, concrete creep, and shrinkage are considered. Temporary and permanent brackets and attachments to the steel liner plate are designed to resist the design loads without loss of the liner integrity due to excessive deformation or load from the brackets or attachments.

Design of the steel liner plate and anchorage system is based on minimum strengths for the materials that are specified for fabrication of the steel components and their interface with the concrete containment. Deviations in the geometry of the liner plate due to fabrication and erection tolerances are considered in the design.

The materials of the liner and its stiffening and anchorage components that are exposed to the internal environment of containment are selected, designed, and detailed to withstand the effects of imposed loads and thermal conditions during design basis conditions.

3.8.1.4.11 Containment Ultimate Capacity

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The Ultimate Pressure Capacity Deterministic Analyses for the RCB is performed in accordance with RG 1.136 and guidance provided in SRP 3.8.1.II.4.K (Rev. 2)

Analysis results for the various containment elements are summarized in Table 3.8-6. These results are based on ANSYS non-linear finite element containment model with nominal stress-strain elasto-plastic materials properties under accident temperature and with cracked concrete section behavior.

The Ultimate Nominal Pressure Capacities for the cylinder and dome sections are calculated using the 2 degree-slice finite element model with simulated axisymmetric boundary conditions. The ultimate conditions in these cases are 0.8 percent strain level in tendon areas located away from discontinuities (according to SRP 3.8.1.II.4.K). The simplified cross-checking hand calculation confirms the finite element model results.

The Ultimate Nominal Pressure Capacities for the ring and gusset sections are evaluated using the same finite element model as above with non-linear analysis run until the first 0.8 percent strain level in the rebars in the critical sections.

Non-Linear 3-D Finite Element Model is used for the hatch Ultimate Nominal Pressure Capacities evaluation. The non-linear steel properties for hatch, flanges, and sleeves are based on elastic-perfectly plastic model with bilinear kinematic hardening according to Von Mises yield criteria. Geometric nonlinearity is accounted for in the large displacement (stability) calculation. The results of calculations are summarized in Table 3.8-6.

Since the hatch performs a leak tightness role, the allowable strain criteria in accordance with ASME Code, Section III, Div. 2, Subsection CC, Article CC-3720 is conservatively used for the hatch ultimate pressure capacity evaluation. These allowable strains are: membrane strain of ε_C=0.5%, ε_T=0.3% and combined membrane + bending strain of ε_C=1.4%, ε_T=1%.

The estimated Ultimate Pressure Capacities are determined from the principal strain levels, which approach ultimate in the protruding sleeves while remaining below yield in the hatch and flange areas.

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~~The ultimate capacity of the RCB is determined for use in probabilistic risk assessments (see Section 19.0) and severe accident analyses. The ultimate capacity of the overall structure and primary sub-assemblies of containment is calculated to determine the limiting ultimate pressure. Ultimate capacity modeling of the concrete RCB is performed in accordance with RG 1.136 and guidance from NUREG/CR-6906, Appendix A (Reference 5).~~

~~Table 3.8-6—Containment Ultimate Pressure Capacity (Pu) at Accident Temperature of 309°F provides the results of the containment ultimate pressure capacity analysis. Hand calculations and non-linear finite element analyses were used to support this analysis.~~

~~Pressure capacities for concrete cylinder and dome sections were calculated using material specified minimum strengths as deterministic values, neglecting the liner plate strength contribution. Pressure capacities were also computed at median and 95 percent confidence levels considering variation in material yield strengths (including the liner plate) and variations in geometry. The ultimate pressure capacity reported for the cylinder and dome is taken as median pressure capacity corresponding to a 0.8 percent maximum strain in the tendons away from discontinuities.~~

~~Pressure capacities for the dome ring and gusset were evaluated using a non-linear finite element model. The dome ring section was evaluated at azimuths where there are no dome post-tension cables present in the cross-section. This occurs at corner locations of the dome tendon criss-cross pattern as presented in Figure 3.8-19. The limiting condition in the dome belt is governed primarily by a membrane failure at the transition between the torus and spherical portions of the dome. A second area of high meridional strains from flexure exist on the inside face toward the middle part of the torus. However, membrane failure at the transition region is the limiting condition. The ultimate pressure capacity reported is the median pressure capacity.~~

~~Pressure capacities were evaluated for the reinforced area around the equipment hatch opening. The evaluation considered a horizontal plane and a vertical plane section passing through the centerline of the opening. The vertical plane section, which corresponds to hoop stress direction, was the weaker of the two planes. The ultimate~~

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~~pressure capacity reported is the median pressure capacity for the vertical plane section.~~

~~The equipment hatch cover and cylinder, shown inFigure 3.8-25—Equipment Hatch General Assembly has a cover ultimate pressure capacity based on ASME Section II, Part D material specification minimum required strengths and an elastic, perfectly plastic stress-strain relationship at 400°F. The internal pressure from containment is applied to the convex surface of the cover and non-embedded portion of the cylinder. The ultimate pressure capacity reported corresponds to ASME Service Level C stress limits for the hatch cover and cylinder.~~

3.8.1.4.12 Design Report

Design information and criteria for Seismic Category I structures are provided in Sections 2.0, 2.4, 2.5, 3.3, 3.5, 3.8.1, 3.8.2, 3.8.3, 3.8.4, and 3.8.5. Design results are presented in Appendix 3E for Seismic Category I structure critical sections.

3.8.1.5 Structural Acceptance Criteria

The limits for RCB allowable stresses, strains, deformations and other design criteria are in accordance with the requirements of Subsection CC-3400 of the ASME BPV Code, Section III, Division 2 and RG 1.136 (GDC 1, GDC 2, GDC 4, GDC 16, and GDC 50). This applies to the overall containment vessel and subassemblies and appurtenances that serve a pressure retaining function, except as noted in Section 3.8.2. Specifically, allowable concrete stresses for factored loadings are in accordance with Subsection CC-3420 and those for service loads are in accordance with Subsection CC-3430.

The limits for stresses and strains in the liner plate and its anchorage components are in accordance with ASME BPV Code, Section III, Division 2, Tables CC-3720-1 and CC-3730-1.

Limits for allowable loads on concrete embedments and anchors are in accordance with Appendix B of ACI-349-2006 and guidance given in RG 1.199.

Section 3.8.1.6 describes minimum requirements for concrete, reinforcing, post-tensioning tendons, and the liner plate system for the RCB.

A SIT is performed as described in Section 3.8.1.7.1.

The RCB is stamped to signify compliance with the ASME BPV Code Section III, Division 2.

– Pressurization at years three and seven uses P_a instead of $1.15P_D$:

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- This exception is acceptable because the structural integrity is confirmed at year zero. Additional overpressurization to $1.15P_D$ unduly ~~fatigues~~cycles the structure and interrupts the surveillance tracking of containment response to P_a.

The EPR containment uses fully grouted tendons in each location. This methodology has several advantages:

- Tendons are surrounded with a cementitious grout injected into the tendon duct; the alkaline composition of the grout mixture, in accordance with RG 1.107, Revision 1 (February 1977), inhibits corrosion of the steel strands and prevents the ingress of corrosive fluids (e.g., water).
- In the event of one or more strand failures during the life of the structure, the bond of the strand with grout and the grout to the concrete wall enables the remaining portion of the post-tensioning to be transmitted to the structure.
- Grouted tendons and tendon anchorages are less vulnerable to local damage than ungrouted tendons. Therefore, if the end anchorages are damaged, for instance by fire or missile impact, the post-tensioning force will be maintained along the effective length of the tendon.
- Grouted tendons increase the overall wall tightness by filling any voids from within the structure. This reduces the risk of water or other contaminates from entering through wall cracks or tendon end caps.
- European experience has found that grouted tendons significantly improve concrete crack distribution when the containment is pressurized to a point where the tensile stress of the concrete is exceeded. Less local large tensile strains are likely to occur thus diminishing the risk of having large concrete cracks behind the containment liner. The absence of large cracks improved the safety margin of the liner with regard to air tightness.

The use of grouted tendons precludes the possibility of directly measuring the post-tension force over time by lifting off at the anchorages. The U.S. EPR mitigates this concern by extensively monitoring the movement of the RCB during 10 CFR 50, Appendix J, leak-rate testing at P_a. The pressure test schedule is a part of the inservice inspection program. Movements obtained from the initial test will be used to baseline a structural analysis that will be used to predict the capacity of the RCB over time. Thirty-six RCB locations will be monitored for radial displacement, 6 for vertical displacement and 13 on the dome for tri-directional displacement. Table 3.8-7—ISI Schedule for the U.S. EPR.

The RCB is fully enclosed by the RSB; therefore, the potential for corrosion of the tendon system is significantly reduced.

3.8.3.2 Applicable Codes, Standards, and Specifications

The following codes, standards, specifications, design criteria, regulations, and regulatory guides are used in the design, fabrication, construction, testing, and inservice inspection of concrete and steel RB internal structures (GDC 1, GDC 2, GDC 4 and GDC 5). Section 5.4.14 describes the applicable codes, standards, and specifications for the design of NSSS component supports.

3.8.3.2.1 Codes and Standards

- ACI 301-05, Specifications for Structural Concrete for Buildings.
- ACI 304R-00, Guide for Measuring, Mixing, Transporting, and Placing Concrete.
- ACI 305.1-06, Specification for Hot-Weather Concreting.
- ACI 306R-88 (Re-approved 2002), Cold-Weather Concreting (Reference 49).
- ACI 306.1-90 (Re-approved 2002), Standard Specification for Cold Weather Concreting.
- ACI 308R-01, Guide to Curing Concrete (Reference 50).
- ACI 308.1-98, Standard Specification for Curing Concrete (Reference 39).
- ACI 311.4R-05, Guide for Concrete Inspection (Reference 40).
- ACI 347-04, Guide to Formwork for Concrete.
- ACI 349-01/349-R01, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (exception described in 3.8.4.4 and 3.8.4.5) (GDC 1).

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- ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of D.4.5(c) requires the use of Condition B even when supplemental reinforcement is provided (Reference 63).
- ACI 349.1R-07, Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures (Reference 41).
- AISC 303-05, Code of Standard Practice for Steel Buildings and Bridges (Reference 42).
- ANSI/AISC N690-1994, Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2, 2004 (GDC 1).
- ANSI/AISC 341-05, Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction, including Supplement 1 (Reference 43).

- Miscellaneous cranes and hoists.

3.8.3.2.3 Design Criteria

- ACI 349-01/349-R01, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety-Related Concrete Structures (GDC 1).

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- ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of D.4.5(c) requires the use of Condition B even when supplemental reinforcement is provided (Reference 63).
- ANSI/AISC N690-1994, Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2 (2004) (GDC 1).

3.8.3.2.4 Regulations

- 10 CFR 50, Appendix A, General Design Criteria for Nuclear Power Plants, GDC 1, GDC 2, GDC 4, GDC 5, and GDC 50.
- 10 CFR 50, Appendix B, Quality Assurance Criteria for Nuclear Power Plants and Fuel Processing Plants.
- 10 CFR 50, Appendix S, Earthquake Engineering Criteria for Nuclear Power Plants.

3.8.3.2.5 NRC Regulatory Guides

RGs applicable to the design and construction of RB internal structures:

- RG 1.61, Revision 1, March 2007 (exception described in 3.7.1).
- RG 1.69, December 1973.
- RG 1.136, Revision 3, March 2007 (exception described in 3.8.1.3).
- RG 1.142, Revision 2, November 2001 (exception described in 3.8.3.3).
- RG 1.160, Revision 2, March 1997.
- RG 1.199, November 2003.

3.8.3.3 Loads and Load Combinations

The U.S. EPR standard plant design loads envelope includes the loads over a broad range of site conditions (GDC 1, GDC 2, GDC 4, GDC 5 and GDC 50). The loads on RB internal structures are separated into the following categories:

- Normal loads.
- Severe environmental loads.
- Extreme environmental loads.
- Abnormal loads.

A COL applicant that references the U.S. EPR design certification will confirm that site-specific loads lie within the standard design envelope for RB internal structures, or perform additional analyses to verify structural adequacy.

Section 5.4.14 addresses the loads and loading combinations and design stress limits for the RCS component and pipe supports.

3.8.3.3.1 Design Loads

Loads on RB internal structures are in accordance with ACI 349-2001 and the guidelines of RG 1.142, Revision 2, November 2001 for concrete structures, and in accordance with ANSI/AISC N690-1994 including Supplement 2 (2004) for steel structures. RG 1.142 delineates the acceptability of ACI 349-1997 with exceptions. The U.S. EPR standard plant design is based on the 2001 edition of the code, with the exceptions noted above. Use of the 2001 edition of the code is acceptable as it incorporates needed updates to the 1997 version. This includes anchorage of wall reinforcing without the use of confined cores in certain situations, and is in keeping with RG 1.199, which adopted the 2001 version Appendix B with exceptions in the area of load combinations. In addition, the guide has supplementary recommendations in the areas of materials, installation, and inservice inspection.

Seismic Category I safety-related RB internal structures are designed for the following loads.

Normal Loads

Normal loads are those loads encountered during normal plant operation, startup, shutdown, and construction (GDC 4). This load category includes:

- Dead Loads (D)—Dead loads include the weight of the structure and any permanent equipment or material weights. Dead load effects also refer to internal moments and forces due to dead loads.
- Live Loads (L)—Live loads include any normal loads that vary with intensity or point of application (or both), including moveable equipment. Live load effects also refer to internal moments and forces due to live loads. Live loads are applied, removed, or shifted in location to obtain the worst-case loading conditions. Impact forces due to moving loads are applied according to the loading condition. In general, a live load of ~~400~~500 pounds per square foot is applied to ~~RB internal~~

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~~structures concrete floors, and 100 pounds per square foot live load is applied to~~ RB internal structures concrete floors and a load of 175 pounds per square foot is applied to steel grating floors and platforms.~~steel grating floors and platforms.~~ Live loads are applied to cranes and their supports for the lifting capacity and test load applied for the lifting device. Additional point loads are applied to concrete floors and concrete and steel floor beams in local design.

- Hydrostatic Loads (F)—Hydrostatic loads are due to fluids stored in pools and tanks in the RB internal structures (e.g., the IRWST and refueling canal). Pools and tanks may have either constant or fluctuating liquid levels. Hydrodynamic loads resulting from seismic excitation of fluids are included as a component of the safe shutdown earthquake load.

- Thermal Loads (T_o)—Thermal loads consist of thermally induced forces and moments resulting from normal plant operation and environmental conditions. Thermal loads and their effects are based on the critical transient or steady-state condition. Thermal expansion loads due to axial restraint, as well as loads resulting from thermal gradients, are considered. The following ambient air temperatures are for normal operation.

  RB internal ambient temperatures:

  - During normal operation:
    - Equipment Area: 131°F (maximum), 59°F (minimum).
    - Service Area: 86°F (maximum), 59°F (minimum).
  - During normal shutdown: 86°F (maximum), 59°F (minimum).

- Pipe Reactions (R_o)—Pipe reactions are those loads applied by piping system supports during normal operating or shutdown conditions based on the critical transient or steady-state conditions. The dead weight of the piping and its contents are included. Dynamic load factors are used when applying transient loads, such as water hammer.

- Construction Loads—Construction loads are those loads to which the structure may be subjected during construction of the plant. Construction loads will be applied to evaluate partially completed structures, temporary structures, and their respective individual members. Design load requirements during construction for buildings and other structures will be developed in accordance with Standard SEI/ASCE 37-02. The magnitude and location of construction loads will be applied to generate the maximum load effects of dead, live, construction, and environmental loads. Consideration will be given to the loads and load effects of construction methods, equipment operation, and sequence of construction.

Severe Environmental Loads

Severe environmental loads are those loads that could be encountered infrequently during the life of the plant (GDC 2). The RB internal structures are protected by the

3.8.3.4 Design and Analysis Procedures

Seismic Category I concrete structural elements and members are designed in accordance with ACI 349-2001 and its appendices (GDC 1). Exceptions to the code found in RG 1.142 are incorporated into the design and are accommodated in the loading combinations described in Section 3.8.3.3.2 for concrete structures.

Seismic Category I steel members and assemblies are designed in accordance with the requirements of ANSI/AISC N690-1994 (R2004) (GDC 1).

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Design of concrete embedments and anchors conforms to ~~Appendix B of~~ ACI 349-~~2001~~06 (Appendix D with exceptions stated in Section 3.8.1.2.1, "Codes") and guidelines of RG 1.199. Ductility is provided by designing anchorage systems so that a steel failure mode controls the design.

Section 5.4.14 describes the applicable design and analysis procedures used for the design of steel portions of the NSSS component supports which interface with the RB internal structures concrete and steel embedments.

Computer modeling and classical manual techniques are used to analyze the RB internal structures by applying loads and loading combinations as described in Section 3.8.3.3. An overall computer model of the NI Common Basemat Structure is used which includes the RB internal structures. Local analyses are then performed for specific structural walls, slabs, and members to account for local effects of specific equipment loads, localized pipe break loads, hydrostatic and hydrodynamic loads, and other conditions (e.g., openings and local changes in member cross-sections). The results from the local analyses are combined with overall analysis results to produce the final analysis for the design of Seismic Category I concrete and steel elements and members.

The following sections describe specific techniques and criteria used for analysis and design of the RB internal structures.

Appendix 3E provides a description of specific analysis and design procedures for RB internal structures critical sections.

3.8.3.4.1 Overall Analysis and Design Procedures

The RB internal structures are included in the ANSYS V10.0 SP1 finite element overall computer model of the NI Common Basemat Structure that is described in Section 3.8.1.4. Boundary conditions for the ANSYS computer model and methods used for application of axisymmetric and non-axisymmetric loads, transient and localized loads, and other parameters used in the model are described in Section 3.8.1.4. The RB internal structures are modeled in combination with the overall NI Common Basemat Structure and basemat using a mesh of ANSYS finite

Building Internal Structures ANSYS Model – View of IRWST and Internal Structures Basemat show the finite element model used for analysis of the RB internal structures. Additional descriptions of the RB internal structures computer model are provided in Appendix 3E.

Loads and load combinations defined in Section 3.8.3.3 are used to determine the strength requirements of members and elements. The following criteria apply for load combinations for concrete and steel RB internal structures:

- A one-third increase in allowable stresses for concrete and steel members due to seismic (E') loadings is not permitted.
- Where any load reduces the effects of other loads, the corresponding coefficient for that load is 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with other loads.
- Where the structural effects of differential settlement, creep, or shrinkage may be significant, they are included with the dead load (D) as applicable.
- For load combinations in which a reduction of the maximum design live load (L) has the potential to produce higher member loads or stresses, multiple cases are considered where the live load (L) is varied between its maximum design value and zero.
- ~~For steel members, thermal loads may be neglected when it can be shown that they are secondary and self limiting in nature.~~
- For load combinations including the loads P_a, T_a, R_a, R_{rr}, R_{rj}, or R_{rm}, the maximum values of these loads, including an appropriate dynamic load factor, are used unless a time-history analysis is performed to justify otherwise.
- For load combinations including loads R_{rr}, R_{rj}, and R_{rm}, the load combinations are first satisfied with these loads set to zero. However, when considering these concentrated loads, local section strength capacities may be exceeded under the effect of these concentrated loads, provided there is not a loss of intended function of the structural member or a loss of function of any safety-related SSC.

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Concrete and steel structural elements and members are designed for axial tension and compression forces, bending moments, torsion, and in-plane and out-of-plane shear forces for the controlling loading combinations that are determined from the ANSYS computer analysis and local analyses. Internal structures behave within the elastic range under design basis loads. However, the ability of the structures to perform beyond yield is considered for abnormal loads associated with a pipe break, which results in rupture reactions, jet impingement and pipe whip, and for missile impact loads.

The strength-design methods described in ACI 349-2001 and its appendices, including the exceptions detailed in RG 1.142, are used for the design of concrete walls, floors and other structural elements for RB internal structures (GDC 1). The ductility requirements of this code are satisfied so that a steel reinforcing failure mode controls over concrete failure modes. The recommendations of Appendix C of ACI 349-2001 are met for impulsive and impactive loading conditions (e.g., loading combinations that include pipe break missile impact loads).

Steel member and assembly design utilizes the allowable stress design methods of ANSI/AISC N690-1994 (R2004), including Supplement 2 (GDC 1). Steel items are maintained elastic for normal and extreme loadings in their respective combinations. Local yielding is permitted for abnormal loadings (e.g., pipe break accident loadings).

3.8.3.4.2 Local Analysis and Design

Local analyses are performed for concrete and steel structural elements and members by using sub-models expanded from the overall analysis model and by using manual techniques, in combination with overall model analysis results. Sub-models are performed by refining the element mesh in the overall ANSYS model. Local discontinuities (e.g., openings, thickened areas, local loads, and changes in member cross-section) are included in the sub-models.

Local analysis and design consider the same member and element forces and moments as described for overall design. In addition, local effects (e.g., punching shear and transfer of anchorage loads to the structure) are considered. Local analyses also are used for design of secondary structures (e.g., platforms, equipment supports, crane supports).

The recommendations of ACI 349-2001 and its appendices, including the exceptions in RG 1.142, are followed for concrete element and member local design (GDC 1).

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Design of concrete embedments and anchors conforms to ~~Appendix B of~~ ACI 349-~~2001~~06 (Appendix D with exceptions stated in Section 3.8.1.2.1, "Codes") and guidelines of RG 1.199. Ductility is provided by designing anchorage systems so that a steel failure mode controls the design.

ANSI/AISC N690-1994 (R2004), including Supplement 2, and ANSI/AISC 341-05, are followed for local steel member design (GDC 1).

The design of bolted connections is in accordance with ANSI/AISC N690, Section Q1.16 and AISC 348-04/2004 RCSC. Bolted in connections are fully tensioned, regardless of design methodology, unless justified otherwise.

The design of welded connections is in accordance with ANSI/AWS D1.1/D1.1M 2006 and ANSI/AWS D1.6-99, including January 6, 2005 update.

The design of bolted connections in combination with welded connections is in accordance with Section Q.15.10 of ANSI/AISC N690.

Openings in walls and slabs of RB internal structures are shown on construction drawings. Openings in slabs are acceptable without analysis if they meet the criteria identified in ACI 349, Section 13.4.2. Round pipe sleeves are used in lieu of rectangular penetrations, where possible. Corners of rectangular openings in walls or slabs are provided with diagonal reinforcing to reduce cracking due to stress concentrations at these locations in accordance with ACI 349, Section 14.3.7.

Appendix 3E provides a description of analysis and design results for critical areas of the RB internal structures.

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Section 5.4.14 describes the design of interfacing steel assemblies which support the NSSS components and attach to, or interact with, embedments in the concrete. Steel supports for the RCS components and piping, including the base plates at the face of concrete structures, are designed in accordance with ASME Section III Division 1, Subsection NF. Embedded portions of RCS component and pipe supports, which are beyond the jurisdictional boundary of the ASME Code, are designed in accordance with ~~ACI 349-2001, including Appendix B~~ ACI 349-06 (Appendix D with exceptions stated in Section 3.8.1.2.1, “Codes”), and also in accordance with ANSI/AISC N690-1994 (R2004), including Supplement 2.

3.8.3.4.3 Static Analysis and Design

Dead loads (D), live loads (L), hydrostatic loads (F), pipe reactions (R_o), and normal thermal loads (T_o) are considered in the analysis and design of RB internal structures for the static normal load concrete and service load steel loading combinations. Normal thermal loads are considered as self-relieving for the overall RB internal structures. Concrete and steel members are designed to accommodate these static loads within the elastic range of their section strength.

Static fluid pressure loads are considered for design of the walls and floors of the IRWST and refueling canal. Moving loads are considered for mobile plant equipment (e.g., the polar crane, refueling machine, and other cranes and hoists).

3.8.3.4.4 Seismic and Other Dynamic Analyses and Design

Seismic analyses and designs of the RB internal structures conform to the procedures described in Section 3.7.2. The procedures in ASCE Standard 4-98 are used in the analysis and design of structural elements and members subjected to load combinations that include seismic loadings. Seismic accelerations are determined from the structural stick model described in Section 3.7.2. These accelerations are applied to the ANSYS model of the RB internal structures as static-equivalent loads at the elevations used in the stick model.

Seismic SSE (E') loads are obtained by multiplying the dead load and 25 percent of the design live load by the structural acceleration obtained from the seismic analysis of the structure. To remain consistent with this methodology, the live load in load combinations that include seismic loads is reduced to achieve the same effective live load as in the seismic loads (25 percent). The resulting forces and moments from the remaining 75 percent of the live load are manually determined and added to the ANSYS results. Seismic loads are also considered due to the mass of fluids in tanks and canals as described herein (Section 3.8.3.4.4). The design live load is used for the local analysis of structural elements and members. Consideration is given to the amplification of these accelerations due to local flexibility of structural elements and members. Construction loads are not included when determining seismic loads. Other temporary loads are evaluated for contributing to the seismic loads on a case-by-case basis.

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Seismic loads from the three components of the earthquake are combined using the SRSS method or the 100-40-40 percent rule described in ASCE 4-98. The 100-40-40 combination is expressed mathematically as follows:

~~Where:~~

Let R_1, R_2, R_3 be the maximum responses of an SSC caused by each of the three earthquake components calculated separately. The maximum seismic response attributable to earthquake loading in three orthogonal directions shall be evaluated as:~~R = the reaction force or moment that is applied in the three orthogonal directions x, y, and z.~~

$$R = \cancel{(}\pm 1.0R_{\cancel{X}1} \pm 0.4R_{\cancel{Y}2} \pm 0.4R_{\cancel{Z}3}\cancel{)},$$

$$R = \cancel{(}\pm 0.4R_{\cancel{X}1} \pm 1.0R_{\cancel{Y}2} \pm 0.4R_{\cancel{Z}3}, \text{ or}\cancel{)}$$

$$R = \cancel{(}\pm 0.4R_{\cancel{X}1} \pm 0.4R_{\cancel{Y}2} \pm 1.0R_{\cancel{Z}3}\cancel{)},\cancel{.}$$

Whichever is greatest.

The effects of local flexibilities in floor slabs and wall panels are considered to determine if additional seismic accelerations should be applied to their design beyond those determined from the seismic stick model. Local flexibility evaluations are performed by determining the natural frequency of the floor or wall panel and comparing this to the frequency of the zero period acceleration on the applicable response spectra. Additional acceleration is applied when the natural frequency of the panel results in higher accelerations than the zero period acceleration. In cases where local flexibilities are determined to be a factor, additional out-of-plane accelerations are applied to the inertia loads on these panels for determining out-of-plane bending and shear loads.

Local flood loads (F_a) are applied to walls and floors of the RB internal structures in the overall ANSYS computer model. Concrete and steel members are designed to accommodate these flood loads within the elastic range of their section strength.

3.8.3.4.5 Design Report

Design information and criteria for Seismic Category I structures are provided in Sections 2.0, 2.4, 2.5, 3.3, 3.5, 3.8.1, 3.8.2, 3.8.3, 3.8.4, and 3.8.5. Design results are presented in Appendix 3E for Seismic Category I structure critical sections.

3.8.3.5 Structural Acceptance Criteria

03.08.03-3

Limits for allowable stresses, strains, deformations, and other design criteria for reinforced concrete RB internal structures are in accordance with ACI 349-2001, and its appendices, including the exceptions specified in RG 1.142, with the exception that the shear strength reduction factor of 0.85 is used as allowed in ~~ACI 349-2006~~ACI 349-06 (Appendix D with exceptions stated in Section 3.8.1.2.1, "Codes"). The exceptions specified in RG 1.142 (GDC 1, GDC 2, GDC 4 and GDC 50) are considered.

Limits for allowable loads on concrete embedments and anchors are in accordance with ~~Appendix B of ACI 349-2006~~ACI 349-06 (Appendix D with exceptions stated in Section 3.8.1.2.1, "Codes") and guidance given in RG 1.199.

Limits for the allowable stresses, strains, deformations and other design criteria for structural steel RB internal structures are in accordance with ANSI/AISC N690-1994, including Supplement 2 (GDC 1, GDC 2, GDC 4 and GDC 50).

Limits for allowable stresses, strains, and deformations on steel RCS component and pipe supports, including the base plates for these supports at the face of concrete structures, are in accordance with ASME Section III Division 1, Subsection NF.

The design of RB internal structures is generally controlled by load combinations containing SSE seismic loads. Stresses and strains are within the ACI 349-2001 and ANSI/AISC N690-1994 limits.

Appendix 3E provides design results for critical areas of the RB internal structures.

3.8.3.6 Materials, Quality Control, and Special Construction Techniques

This section contains information relating to the materials, quality control programs, and special construction techniques used in the fabrication and construction of concrete and steel internal structures of the RB internal structures (GDC 1).

- ACI 347-04 - Guide to Formwork for Concrete.
- ACI 349-01/349-R01 - Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (exception described in 3.8.4.4 and 3.8.4.5) (GDC 1).

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- ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of D.4.5(c) requires the use of Condition B even when supplemental reinforcement is provided (Reference 63).
- ACI 349.1R-07 - Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures.
- ACI 350-06 - Code Requirements for Environmental Engineering Concrete Structure (Reference 58).
- ACI 350.3-06 - Seismic Design of Liquid-Containing Concrete Structures (Reference 59).
- AISC 303-05 - Code of Standard Practice for Steel Buildings and Bridges.
- ANSI/AISC N690-1994 - Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2 (2004) (GDC 1).
- ANSI/AISC 341-05 - Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction, including Supplement 1.
- ANSI/ANS-6.4-2006 - Nuclear Analysis and Design of Concrete Radiation Shielding for Nuclear Power Plants (Reference 4).
- AISC 348-04/2004 RCSC - Specification for Structural Joints Using ASTM A325 and A490 Bolts.
- ANSI/AWS D1.1/D1.1M 2006 - Structural Welding Code – Steel.
- ANSI/AWS D1.4-2005 - Structural Welding Code - Reinforcing Steel.
- ANSI/AWS D1.6-99, including January 6, 2005 update - Structural Welding Code – Stainless Steel.
- ANSI/AWS D1.8 2005 - Structural Welding Code – Seismic Supplement.
- ASME BPV Code - 2004 Edition, Section III, Division 2 – Code for Concrete Reactor Vessels and Containments.
- ASME NOG-1-2004 - Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girders).

- ASME B31.3 - 1996 - Process Piping, American Society of Mechanical Engineers (Reference 60).
- ASME B31.4 - 1992 - Liquid Transportation System for Hydrocarbon, Liquid Petroleum Gas, Anhydrous Ammonia, and Alcohols (Reference 61).
- ASME B31.8 - 1995 - Gas Transportation and Distribution Piping Systems.

3.8.4.2.2 Specifications

Industry standards (e.g., those published by the ASTM) are used to specify material properties, testing procedures, fabrication methods, and construction methods.

Structural specifications cover areas related to the design and construction of other Seismic Category I structures. These specifications emphasize important points of the industry standards for these structures and reduce options that would otherwise be permitted by the industry standards. These specifications cover the following areas:

- Concrete material properties.
- Mixing, placing, and curing of concrete.
- Reinforcing steel and splices.
- Structural steel.
- Steel liner plate and embedments.
- Miscellaneous and embedded steel.
- Anchor bolts.
- Expansion anchors.
- Cranes and hoists.

3.8.4.2.3 Design Criteria

- ACI 349-01/349-R01 - Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (GDC 1).

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- ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of D.4.5(c) requires the use of Condition B even when supplemental reinforcement is provided (Reference 63).
- ANSI/AISC N690-1994 - Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2 (2004) (GDC 1).

ANSI/AISC N690-1994 including Supplement 2 (2004) for steel structures (GDC 1, GDC 2, GDC 4, and GDC 5).

Other Seismic Category I structures are designed for the following loads, as described in Section 3.8.4.4:

Normal Loads

Normal loads are those loads encountered during normal plant operation, startup, shutdown, and construction (GDC 4). This load category includes:

- Dead loads (D)—Dead loads include the weight of the structure and any permanent equipment or material weights. Dead load effects also refer to internal moments and forces due to dead loads.

  For buried items, the dead load includes the weight of the soil overburden. The soil overburden load includes the weight of the overlying soil prism.

- Live loads (L)—Live loads include any normal loads that vary with intensity and point of application, including moveable equipment and precipitation loads. Live load effects also refer to internal moments and forces due to live loads. Live loads are applied, removed, or shifted in location to obtain the worst-case loading conditions. Impact forces due to moving loads are applied for the loading condition.

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  ~~In general, a live load of 400 pounds/ft^2 is applied to FB concrete floors, and 100 pounds/ft^2 live load is applied to concrete floors and to steel grating floors and platforms in other Seismic Category I structures.~~In general, a live load of 500 pounds per square foot is applied to FB concrete floors and a load of 175 pounds per square foot is applied to FB and SB steel grating floors and platforms. A live load of 300 pounds per square foot is applied to SB concrete floors. Finally, a live load of 100 pounds per square foot is applied to concrete floors, steel grating floors, and platforms in other Seismic Category I structures. Floor live loads may vary according to the function of individual floors. Truck loads, fuel cask shipment loads, and loads due to replacement of RCS components are considered as live loads in the loading and material handling bays of the FB. Live loads are applied to cranes and their supports for the lifting capacity and test loads applied for lifting devices. Additional point loads are applied to concrete floors and to concrete and steel beams in local design.

  The design live load for rainfall is based on a rate of 19.4 inches per hour, as described in Section 2.4.

  The design live load due to rain, snow, and ice is based on 100 pounds/ft^2 on the ground, as described in Section 2.4. This value is postulated as a meteorological site parameter for the extreme winter precipitation load and includes the weight of the normal winter precipitation event and the weight of the extreme winter precipitation event. Roof snow and ice loads are determined using Chapter 7 of ASCE/SEI 7-05, "Minimum Design Loads for Buildings and Other Structures." The

$$0.9Y = \cancel{1.1}\ (D + \cancel{F +}\ L + H + F + F_b + J + T_a + R_a + F_a + 1.25P_a)$$

$$0.9Y = \cancel{1.1\ (D + F + L + H + T_a + R_a + F_a + P_a + R_{rr} + R_{rj}}\ (D + L + H + F + F_b + J + T_a + R_a + F_a + P_a + R_{rm} + E').$$

3.8.4.4 Design and Analysis Procedures

Analysis and design procedures are similar for the various concrete and steel other Seismic Category I structures but vary somewhat from structure to structure. The general analysis and design procedures applicable to other Seismic Category I structures are explained below. The procedures specific to the following other Seismic Category I structures are also described.

- The RSB and annulus, FB, and SBs.
- The EPGBs.
- The ESWBs.
- Buried conduit and duct banks, and buried pipe and pipe ducts.

Design and analysis procedures described in the following sections also apply to the design of supports for Seismic Category I distribution systems (i.e., pipe supports, equipment supports, cable tray supports, conduit supports, HVAC duct supports, and other component supports) and to Seismic Category I platforms and miscellaneous steel structures located within other Seismic Category I buildings and structures.

3.8.4.4.1 General Procedures Applicable to Other Seismic Category I Structures

Other Seismic Category I concrete structural elements and members are designed in accordance with the requirements of ACI 349-2001 and its appendices (GDC 1). Exceptions to code requirements specified in RG 1.142 are incorporated into the design and are accommodated in the loading combinations described in Section 3.8.4.3.2 for concrete structures.

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The design of concrete walls, floors, and other structural elements for other Seismic Category I structures is performed using the strength-design methods described in ACI 349-2001, with the exceptions that the shear strength reduction factor of 0.85 is used as allowed in ACI 349-~~20~~06 (Appendix D with exceptions stated in Section 3.8.1.2.1, "Codes"). Use of this shear strength reduction factor is acceptable because the loss of strength and stiffness due to cyclic inelastic loading in structural members of nuclear structures is smaller when compared to that of a conventional building structure, where a lower reduction factor is used. The ductility requirements of ACI 349-2001 are satisfied to provide a steel reinforcing failure mode and prevent concrete failure for design basis loadings.

03.08.03-3

The design of anchors and embedments conforms to the requirements of ~~Appendix B of~~ ACI 349-~~2001~~06 (Appendix D with exceptions stated in Section 3.8.1.2.1, "Codes") and RG 1.199. Ductility is provided by designing anchorage systems such that a steel failure mode controls the design. The requirements of Appendix C of ACI 349-2001 are followed for impulsive and impactive loading conditions (e.g., loading combinations that include pipe break missile impact loads or tornado-generated missile impact loads).

Other Seismic Category I steel members and assemblies are designed in accordance with ANSI/AISC N690-1994 (R2004, including Supplement 2) (GDC 1). Steel member design uses the allowable stress design methods of ANSI/AISC N690.

The design of bolted connections is in accordance with ANSI/AISC N690, Section Q1.16 and AISC 348-04/2004 RCSC, "Specification for Structural Joints Using ASTM A325 and A490 Bolts." Bolted connections are designed to be fully tensioned (e.g., slip critical) unless justified otherwise.

The design of welded connections is in accordance with AWS D1.1 or AWS D1.6.

The design of bolted connections in combination with welded connections is in accordance with Section Q.15.10 of ANSI/AISC N690.

Loads and load combinations defined in Section 3.8.4.3 are used to determine strength requirements of members and elements of other Seismic Category I structures. Abnormal pipe break accident loads only apply to limited areas of structures located on the NI Common Basemat Structure. The following criteria apply for load combinations for concrete and steel other Seismic Category I structures:

- The one-third increase in allowable stresses for concrete and steel members due to seismic (E') or wind (W and Wt) loadings is not permitted.
- Where any load reduces the effects of other loads, the corresponding coefficient for that load is 0.9 if it can be demonstrated that the load occurs simultaneously with other loads.
- Where the structural effects of differential settlement, creep, or shrinkage may be significant, they are included with the dead load (D) as applicable.
- For load combinations in which a reduction of the maximum design live load (L) has the potential to produce higher member loads and stresses, multiple cases are considered where the live load (L) is varied between its maximum design value and zero.
- Roofs with a slope of less than 0.25 inches per foot are analyzed for adequate stiffness to preclude progressive deflection as water ponding is created from the snow load or from rainfall on the surface. The analysis considers the potential

blockage of the primary drainage system of the area that is subject to ponding loads. The analysis uses the larger of the snowmelt depth or rain load.

03.08.03-6

- ~~For steel members, thermal loads may be neglected when it can be shown that they are secondary and self limiting in nature.~~

- For load combinations including the loads P_a, T_a, R_a, R_{rr}, R_{rj}, or R_{rm}, the maximum values of these loads, including a dynamic load factor, are used unless a time-history analysis is performed to justify otherwise.

- For load combinations including loads R_{rr}, R_{rj}, R_{rm}, or W_m, these load combinations are first satisfied with these loads set to zero. However, when considering these concentrated loads, local section strength capacities may be exceeded under the effect of these concentrated loads, provided there is not a loss of intended function of the structural member or a loss of function of any safety-related SSC.

- Twenty-five percent of the design live load is considered during static analysis with seismic load combinations. The full potential live load is used for the local analysis of structural members.

- Tornado loads are applied to roofs and exterior walls of other Seismic Category I structures. If tornado pressure boundaries are not established at the exterior walls, interior walls are designed as tornado pressure boundaries.

- For load combinations that include a tornado load (W_t), the tornado load parameter combinations described in Section 3.3 are used.

Concrete and steel structural elements and members are designed for axial tension and compression forces, bending moments, torsion, and in-plane and out-of-plane shear forces for the controlling loading combinations that are determined from analysis. Concrete and steel members and elements remain elastic for loadings other than impact. Local yielding is permitted for localized areas subjected to tornado-generated missile loads, pipe break accident loadings, and beyond design basis loadings. The structural integrity of members and elements is maintained for the loading combinations described in Section 3.8.4.3.

Analysis and design of other Seismic Category I structures are performed using a combination of computer models and local analyses. Computer models are used to perform overall analysis of major structures. The loads and loading combinations described in Section 3.8.4.3 are applied to the overall computer model to design for global effects of the loadings. Local analyses and designs are performed using refined computer submodels and manual calculations. Local analyses and designs are used to account for local discontinuities (e.g., openings, thickened areas, local loads, punching shear checks, and changes in member cross-section). Local analyses are also used to determine designs for items such as component supports, embedments, anchors, platforms, and other miscellaneous structural items. Techniques used for major structures are described in Sections 3.8.4.4.2 through 3.8.4.4.5.

structural mass for seismic analysis of Seismic Category I structures. Seismic loads are also considered due to the mass of fluids in tanks and canals as described below for hydrodynamic loads. The full potential live load, including precipitation, is used for the local analysis of structural elements and members. Consideration is given to the amplification of seismic accelerations obtained from the structural stick model of each structure, due to local flexibility of structural elements and members. Construction loads are not included when determining seismic loads. Other temporary loads are evaluated for contributing to the seismic loads on a case-by-case basis.

03.08.03-10

Seismic loads from the three components of the earthquake motion are combined using the SRSS method or the 100-40-40 percent rule described in ASCE 4-98. The 100-40-40 combination is expressed mathematically as follows:

~~Where:~~

~~R = the reaction force or moment that is applied in the three orthogonal directions x, y, and z:~~

$\cancel{R = (\pm 1.0R_X \pm 0.4R_Y \pm 0.4R_Z)}$

$\cancel{R = (\pm 0.4R_X \pm 1.0R_Y \pm 0.4R_Z)}$

$\cancel{R = (\pm 0.4R_X \pm 0.4R_Y \pm 1.0R_Z)}$.

Let R_1, R_2, R_3 be the maximum responses of an SSC caused by each of the three earthquake components calculated separately. The maximum seismic response attributable to earthquake loading in three orthogonal directions shall be evaluated as:

$R = \pm 1.0R_1 \pm 0.4R_2 \pm 0.4R_3$,

$R = \pm 0.4R_1 \pm 1.0R_2 \pm 0.4R_3$, or

$R = \pm 0.4R_1 \pm 0.4R_2 \pm 1.0R_3$,

Whichever is greatest.

The effects of local flexibilities in floor slabs and wall panels are considered to determine if additional seismic accelerations should be applied to their design beyond those determined from the seismic stick model. Local flexibility evaluations are performed by determining the natural frequency of the floor or wall panel and comparing this to the frequency of the zero period acceleration on the applicable response spectra. Additional acceleration is applied when the natural frequency of the panel results in higher accelerations than the zero period acceleration. In cases where local flexibilities are determined to be a factor, additional out-of-plane accelerations are applied to the inertia loads on these panels for determining out-of-plane bending and shear loads.

impactive effects of the water moving and sloshing in the tanks as a result of seismic excitation. These loads are considered as part of the seismic SSE loads, and components of these loads in the three orthogonal directions are combined in the same manner as other seismic loads. The requirements of ASCE 4-98, "Seismic Analysis of Safety-Related Nuclear Structures," ASCE Manual No. 58, USAEC TID-7024, and other proven methods are used to determine hydrodynamic loadings. The effect of tank structure flexibility on spectral acceleration is included when determining the hydrodynamic pressure on the tank wall for the impulsive mode.

Design for hydrodynamic loads is within the elastic range of concrete and steel members and elements.

Thermal Analysis and Design

Normal thermal loads (T_o) are considered in the analysis and design of other Seismic Category I structures. Abnormal pipe break accident thermal loads (T_a) are considered to have no effect on the overall structure of other Seismic Category I structures and are only considered in local analyses.

03.08.03-6

~~For steel members, thermal loads are neglected when it can be shown that they are secondary and self limiting in nature.~~

For concrete structures, the requirements of ACI 349, Appendix A, ACI 349.1R, or thermal analysis computer programs or similar procedures are used to evaluate thermally induced forces and moments. When considering the combined effects of thermal stress and stress due to other loads, the analysis satisfies the requirements of Appendix A of ACI 349.

Pipe Rupture Loads

Other Seismic Category I structures will be evaluated for pipe rupture loads. Local analyses of other Seismic Category I structures consider the following abnormal loads for areas that house high-energy piping systems:

- Subcompartment pressure loads (P_a).
- Pipe break thermal loads (T_a).
- Accident pipe reactions (R_a).
- Pipe break reaction, jet impingement, and missile loads (R_{rr}, R_{rj}, R_{rm}).
- Local flood loads (F_a).

Subcompartment pressure loads (P_a) resulting from a LOCA event are evaluated as time-dependent loads across concrete walls and floors that enclose high-energy piping as strain rate and magnitude, confining stress, and relative density on pore pressure, damping, and shear modulus will be incorporated into analyses. Response of buried items to burial depth, groundwater, presence of adjacent structures, and soil heterogeneity will be evaluated in seismic analyses.

Buried items will be evaluated for the effects of settlement and ground movement, including potential damage related to compaction of soil during construction, long-term elastic and consolidation settlement (total and differential), freeze-thaw induced settlement, seismic-induced settlement, seismic wave propagation, and seismic-induced permanent ground deformation. The effects of differential settlement between buried pipes and the buildings or structures to which pipes are anchored will be evaluated. At site locations where differential settlement is significant, flexible anchors may be used in lieu of rigid anchors. Support structures will be designed to resist the resulting axial loads, bending stresses, and shear stresses imposed by buried items on the structure.

Refer to the AREVA NP Inc., U.S. Piping Analysis and Pipe Support Design Topical Report (Reference 37) for additional analysis and design procedures applicable to buried piping.

A COL applicant that references the U.S. EPR design certification will describe the design and analysis procedures used for buried conduit and duct banks, and buried pipe and pipe ducts.

A COL applicant that references the U.S. EPR design certification will use results from site-specific investigations to determine the routing of buried pipe and pipe ducts.

A COL applicant that references the U.S. EPR design certification will perform geotechnical engineering analyses to determine if the surface load will cause lateral or vertical displacement of bearing soil for the buried pipe and pipe ducts and consider the effect of wide or extra heavy loads.

3.8.4.4.6 Design Report

Design information and criteria for Seismic Category I structures are provided in Sections 2.0, 2.4, 2.5, 3.3, 3.5, 3.8.1, 3.8.2, 3.8.3, 3.8.4, and 3.8.5. Design results are presented in Appendix 3E for Seismic Category I structure critical sections.

3.8.4.5 Structural Acceptance Criteria

03.08.03-3

Limits for allowable stresses, strains, deformations and other design criteria for other Seismic Category I reinforced concrete structures are in accordance with ACI 349-2001 and its appendices, with the exceptions that the shear strength reduction factor of 0.85 is used as allowed in ACI 349-~~20~~06 (Appendix D with exceptions stated in Section

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3.8.1.2.1, "Codes"). (GDC 1, GDC 2, and GDC 4). Limits for concrete design include the exceptions specified in RG 1.142.

Limits for allowable loads on concrete embedments and anchors are in accordance with the requirements of ~~Appendix B of~~ ACI 349-~~2001~~06 (Appendix D with exceptions stated in Section 3.8.1.2.1, "Codes") and RG 1.199.

Limits for the allowable stresses, strains, deformations, and other design criteria for other structural steel Seismic Category I structures are in accordance with ANSI/AISC N690-1994 (R2004) including Supplement 2 (GDC 1, GDC 2, and GDC 4).

Allowable settlements for other Seismic Category I structures are described in Section 2.5.

The design of other Seismic Category I structures is generally controlled by load combinations containing SSE seismic loads. Stresses and strains are within the ACI 349-2001 limits, with the exceptions previously listed, and ANSI/AISC N690-1994 limits.

Appendix 3E provides design results for critical sections of other Seismic Category I structures.

Structural acceptance criteria for buried Seismic Category I pipe are addressed in the AREVA NP Inc., U.S. Piping Analysis and Pipe Support Design Topical Report.

A COL applicant that references the U.S. EPR design certification will confirm that site-specific Seismic Category I buried conduit, electrical duct banks, pipe, and pipe ducts satisfy the criteria specified in Section 3.8.4.4.5 and those specified in the AREVA NP Inc., U.S. Piping Analysis and Pipe Support Design Topical Report.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

This section contains information relating to the materials, quality control programs, and special construction techniques used in the fabrication and construction of concrete and steel Seismic Category I structures other than the RCB and the RB internal structures.

Construction of concrete radiation shielding structures and certain elements of design that relate to problems unique to this type of structure is in accordance to RG 1.69. The requirements and recommended practices contained in ANSI/ANS-6.4-2006, are generally acceptable for the construction of radiation shielding structures, as amended by the applicable exceptions noted in RG 1.69.

62. ASME B31.8, "Gas Transportation and Distribution Piping Systems," American Society of Mechanical Engineers, 1995.

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63. ACI 349-06/349R-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Appendix D) with the exception of D.4.5(c) requires the use of Condition B even when supplemental reinforcement is provided (Reference 63).

Table 3.8-6—Containment Ultimate Pressure Capacity (P$_u$) at Accident Temperature of 309°F

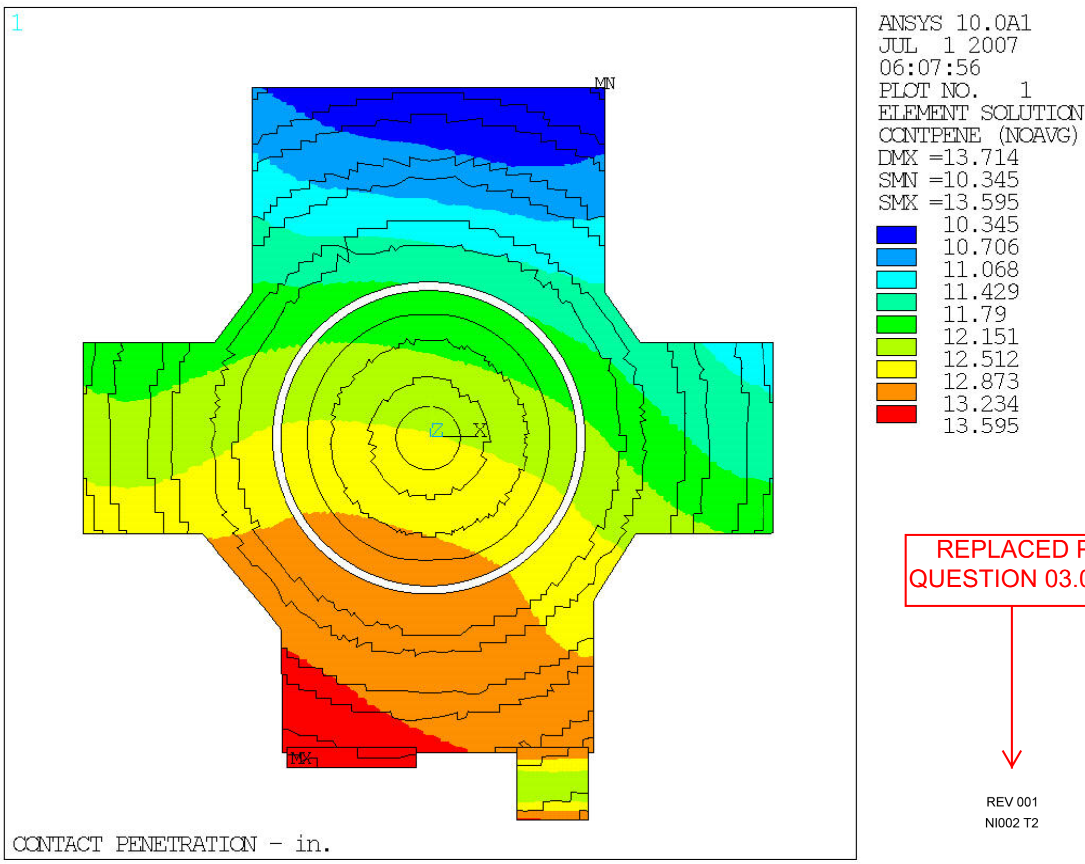
Sections ~~Section Evaluated~~	~~Pressure Capability (psig)~~ Pressure Capacity (P$_u$): Pu (psi) ~~72°F~~ [1]	Ratio P$_u$/P$_d$ ~~395°F~~ [(2)]	Failure Mode/Location ~~/ Limiting Condition~~
Cylinder (Hoop)	267 ~~277~~	4.31 ~~260~~	Failure due to maximum allowable membrane strains away from structural discontinuities. ~~Membrane Failure. 0.8% strain away from discontinuities~~
Dome	249 ~~201~~	4.02 ~~189~~	Failure due to maximum allowable membrane strains away from structural discontinuities. ~~Membrane Failure. 0.8% strain away from discontinuities~~
Dome Belt ~~Ring~~	173 ~~199~~	2.79 ~~187~~	Failure due to maximum allowable flexural strains at structural discontinuities. ~~Membrane and Flexural Failure~~
Gusset Base ~~of Cylinder Wall (Gusset)~~	315 ~~302~~	5.08 ~~284~~	Failure due to maximum allowable flexural strains at structural discontinuities. ~~Flexural Failure (Concrete Compression)~~
~~Reinforcing around~~ Equipment Hatch [(1)] ~~Opening (Vertical Section Critical)~~	156 ~~241~~	2.52 ~~227~~	"Loss" of structural integrity in protruding sleeve area due to principal strain which approaches ultimate. ~~Flexural Failure~~
Equipment Hatch [(1)] ~~Cover~~	125 ~~-~~	2.02 ~~119~~	"Loss" of leak tightness in protruding sleeve due to principal strain which approaches ultimate. ~~ASME Service Level C Limit.~~

Notes:

1. Conservatively calculated under Accident Temperature of 338°F (170°C). ~~Average Normal Operating Temperature.~~
2. P$_d$ – design pressure. ~~Maximum Design Basis Temperature.~~

03.08.01-10

Figure 3.8-106—Elastic Displacement for Soil Case 1u



REPLACED PER QUESTION 03.08.05-6

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