ArevaEPRDCPEm Resource

From: Pederson Ronda M (AREVA NP INC) [Ronda.Pederson@areva.com]
Sent: Friday, July 31, 2009 4:15 PM
To: Tesfaye, Getachew
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, FSAR Ch 3, Supplement 5
Attachments: RAI 155 Supplement 5 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. AREVA NP submitted Supplement 3 to the response on May 29, 2009, to address 20 of the remaining questions. AREVA NP submitted Supplement 4 to the response on June 30, 2009, to address 8 of the remaining questions. The attached file, "RAI 155 Supplement 5 Response US EPR DC.pdf" provides technically correct and complete responses to 11 of the remaining 16 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 5 Questions 03.08.02-2, 03.08.02-7, 03.08.03-4, 03.08.03-17,03.08.05-1, 03.08.05-8, and 03.08.05-12.

The following table indicates the respective pages in the response document, "RAI 155 Supplement 5 Response US EPR DC.pdf" that contain AREVA NP's response to the subject questions.

Question #	Start Page	End Page
RAI 155 — 03.08.02-02	2	2
RAI 155 — 03.08.02-07	3	3
RAI 155 — 03.08.02-08	4	4
RAI 155 — 03.08.03-04	5	5
RAI 155 — 03.08.03-16	6	7
RAI 155 — 03.08.03-17	8	9
RAI 155 — 03.08.05-01	10	10
RAI 155 — 03.08.05-08	11	16
RAI 155 — 03.08.05-10	17	18
RAI 155 — 03.08.05-12	19	19
RAI 155 — 03.08.05-18	20	20

The schedule for technically correct and complete responses to the remaining 5 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 — 03.08.01-20	October 30, 2009
RAI 155 — 03.08.01-24	October 30, 2009
RAI 155 — 03.08.04-06	October 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: WELLS Russell D (AREVA NP INC)
Sent: Tuesday, June 30, 2009 8:34 PM
To: 'Getachew Tesfaye'; Miernicki, Michael
Cc: Pederson Ronda M (AREVA NP INC); BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC)
Subject: Response to U.S. EPR Design Certification Application RAI No. 155, FSAR Ch 3, Supplement 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 73 questions. AREVA NP submitted Supplement 2 to the response on April 30, 2009, to address 9 of the remaining 53 questions. AREVA NP submitted Supplement 3 to the response on May 29, 2009, to address 20 of the remaining 44 questions. The attached file, "RAI 155 Supplement 4 Response US EPR DC.pdf" provides technically correct and complete responses to 8 of the remaining 24 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 03.08.05-14 and 03.08.02-1.

The following table indicates the respective pages in the response document, "RAI 155 Supplement 4 Response US EPR DC.pdf," that contain AREVA NP's response to the subject questions.

Question #	Start Page	End Page
RAI 155 — 03.08.01-11	2	2
RAI 155 — 03.08.02-1	3	3
RAI 155 — 03.08.02-4	4	4
RAI 155 — 03.08.05-7	5	8
RAI 155 — 03.08.05-13	9	10
RAI 155 — 03.08.05-14	11	13
RAI 155 — 03.08.05-15	14	14
RAI 155 — 03.08.05-16	15	15
RAI 155 — 03.08.05-18	16	16

The schedule for technically correct and complete responses to the remaining 16 questions is unchanged, with the exception of question 03.08.05-18, and is provided below. The schedule for the response to question 03.08.05-18 has been changed to July 31, 2009.

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 — 03.08.01-20	October 30, 2009
RAI 155 — 03.08.01-24	October 30, 2009
RAI 155 — 03.08.02-02	July 31, 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 — 03.08.05-08	July 31, 2009
RAI 155 — 03.08.05-10	July 31, 2009
RAI 155 — 03.08.05-12	July 31, 2009
RAI 155 — 03.08.05-18	July 31, 2009

Sincerely,

(Russ Wells on behalf of)

Ronda Pederson
ronda.pederson@areva.com
Licensing Manager, U.S. EPR Design Certification
New Plants Deployment
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: 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

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

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 9:12 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
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 9:11 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-mail the 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,

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: 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
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: 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,

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: 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
(301) 415-3361

Hearing Identifier: AREVA_EPR_DC_RAIs
Email Number: 702

Mail Envelope Properties (5CEC4184E98FFE49A383961FAD402D31011D30AA)

Subject: Response to U.S. EPR Design Certification Application RAI No. 155, FSAR Ch 3, Supplement 5
Sent Date: 7/31/2009 4:15:03 PM
Received Date: 7/31/2009 4:15:09 PM
From: Pederson Ronda M (AREVA NP INC)

Created By: Ronda.Pederson@areva.com

Recipients:
"BENNETT Kathy A (OFR) (AREVA NP INC)" <Kathy.Bennett@areva.com>
Tracking Status: None
"DELANO Karen V (AREVA NP INC)" <Karen.Delano@areva.com>
Tracking Status: None
"VAN NOY Mark (EXT)" <Mark.Vannoy.ext@areva.com>
Tracking Status: None
"Tesfaye, Getachew" <Getachew.Tesfaye@nrc.gov>
Tracking Status: None

Post Office: AUSLYNCMX02.adom.ad.corp

Files	**Size**	**Date & Time**
MESSAGE	28860	7/31/2009 4:15:09 PM
RAI 155 Supplement 5 Response US EPR DC.pdf		401677

Options
Priority: Standard
Return Notification: No
Reply Requested: No
Sensitivity: Normal
Expiration Date:
Recipients Received:

Response to

Request for Additional Information No. 155, Supplement 5

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 03.08.02-2:

SRP 3.8.2 requires that descriptive information be provided for steel containments. FSAR Section 3.8.2.1.1 states that the construction opening closure cap is designated as a class MC component in compliance with Article NE 3000 of the ASME Code, Section III, Division 2. There does not appear to be any information for the construction opening and closure cap. Provide a description and figure(s) showing the details of this large penetration and how it will meet the requirement under GDC 16 to provide a leak tight boundary under design load conditions.

Response to Question 03.08.02-2:

The construction opening diameter is approximately 9'-6". Upon completion of construction, the construction opening is closed by installing a closure cap on the inside of the Reactor Containment Building (RCB). The closure cap is a torispherical dome that is field-welded to the construction opening penetration sleeve. The corresponding opening in the Reactor Shield Building (RSB) is closed with a concrete pour at completion of construction. Steel items not backed by concrete described in U.S. EPR FSAR Tier 2, Section 3.8.2, including the construction opening closure cap and sleeve, are subject to testing and inservice inspection requirements specified in U.S. EPR FSAR Tier 2, Section 3.8.2.7, which satisfies GDC 16. The U.S. EPR FSAR will be revised to clarify closure of the construction opening as indicated above.

FSAR Impact:

U.S. EPR FSAR Tier 2, Sections 3.8.2.1, 3.8.2.1.1, 3.8.2.2.2, and 3.8.2.4.1 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.02-7:

10 CFR 50.55a requires that inservice inspection of steel containments be conducted as outlined in ASME Code Section XI Subsection IWE. In FSAR Section 3.8.2.7, Testing and Inservice Inspection Requirements, no mention is made of the ASME Code, Section XI, Subsection IWE requirements. Provide additional information to identify how each of the Section XI Code requirements and 10 CFR 50.55a supplemental inspection requirements will be met.

Response to Question 03.08.02-7:

ASME Boiler and Pressure Vessel (BPV) Code, Section XI is the U.S. EPR FSAR basis for inservice inspections that are intended to maintain the U.S. EPR in a safe condition once construction requirements have been satisfied. The COL applicant is responsible for developing and implementing a specific inservice inspection program and its procedures in compliance with ASME BPV Code and 10 CFR 50.55a.

U.S. EPR FSAR Tier 2, Section 3.8.2.7 will be revised to specifically state that inservice inspection will comply with ASME BPV Code, Section XI, Subsection IWE requirements, including supplemental inspection requirements of 10 CFR 50.55a.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.8.2.7 will be revised as described in the response and as indicated on the enclosed markup.

Question 03.08.02-8:

GDC 16 requires the containment to act as a leak tight membrane to prevent the uncontrolled release of radioactive effluent to the environment. In FSAR Section 3.8.2.2, Design Load Combinations, describe how differential movement between the RCB and the RSB is considered in the analysis of the equipment hatch, the air locks, and the construction opening, for the design-basis accident pressure and temperature conditions.

Response to Question 03.08.02-8:

Steel pressure boundary components not backed by concrete (i.e., the equipment hatch, airlocks, and construction opening) are procurement items. Complete design, assembly, material specifications, and code boundaries will be part of procurement documentation developed later in the design process. However, the U.S. EPR design is based on the European EPR plant model, which is currently under construction in Europe. Based on European EPR design and procurement experience, the U.S. EPR equipment hatch will be designed to use steel bellows to allow differential movement between the Reactor Containment Building (RCB) and Reactor Shield Building (RSB) structures. Airlock shells are leak-tight welded to sleeves embedded in the RCB. Each airlock then extends through a special embedded sleeve in the RSB that uses bellows to allow differential movement.

Upon completion of construction, the RCB construction opening will be sealed by a welded closure cap on the inside of the RCB. The RSB construction opening will be closed with a concrete pour.

Design, analysis, and assembly of each component will be performed in compliance with U.S. codes and standards will allow the containment to function as a leak-tight barrier throughout the life of the plant, and will accommodate differential movement between the RCB and RSB.

FSAR Impact:

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

Question 03.08.03-4:

FSAR Sections 3.8.3.2.1, 3.8.4.2 and 3.8.5.2 indicate that standards AISC 303-05, Code of Standard Practice for Steel Buildings and Bridges, ANSI/AISC 341-05, Seismic Provisions for Structural Steel Buildings, including Supplement 1, and AISC 348-04/2004 RCSC, Specification for Structural Joints Using ASTM A325 and A490 Bolts are utilized for the design of steel structures. SRP 3.8 references the use of ANSI/AISC N690-1994, including Supplement 2 (2004), Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities. The N690 Standard references other AISC standards in turn, but not these three AISC standards with the noted revisions. Therefore, AREVA is requested to justify the use of these new AISC standards for use of steel structures in FSAR Sections 3.8.3 through 3.8.5. This should include a description and listing of the differences between these new standards and N690 standard (including any of the referenced standards within N690), and justify any differences that are identified as a relaxation of the design provisions.

Response to Question 03.08.03-4:

References to AISC 303, Code of Standard Practice for Steel Buildings and Bridges, and AISC 348, Specification for Structural Joints Using ASTM A325 and A490 Bolts will be revised in U.S. EPR FSAR Tier 2, Sections 3.8.3, 3.8.4, and 3.8.6 to reflect revisions of the respective codes referenced in ANSI/AISC N690-1994, including Supplement 2 (2004), "Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities."

ANSI/AISC 341-05, "Seismic Provisions for Structural Steel Buildings," including Supplement 1, is not referenced in ANSI/AISC N690-1994, including Supplement 2 (2004). Therefore, it will be removed from U.S. EPR FSAR Tier 2, Sections 3.8.3 and 3.8.6.

FSAR Impact:

U.S. EPR FSAR Tier 2, Sections 3.8.3.2.1, 3.8.3.4.2, 3.8.3.6.3, 3.8.4.2.1, 3.8.4.4.1, and 3.8.6 will be revised as described in the response and as indicated on the enclosed markup.

Question 03.08.03-16:

FSAR Sections 3.8.3 through 3.8.5 and Appendix 3E describe the finite element models used for containment internal structures and other seismic Category I structures. To determine the acceptability of these models, provide the additional information requested below for all seismic Category I structures:

1. From the information provided, it is not clear whether the finite element discretization is sufficient. The FSAR does not describe what procedures are used to select the appropriate number of elements for meshing concrete regions such as walls and slabs. The mesh density used for both the global and local finite element models, described in Section 3.8.3 and Appendix 3E, in many cases appear coarse for 4-noded and 3-noded shell elements. Explain how the mesh refinement was determined and validated for each model. Describe any finite element options that were selected to improve the accuracy of the results, and describe why they were appropriate.
2. Since triangular finite elements were used in addition to rectangular elements and it is recognized that generally triangular elements are not as accurate as rectangular elements, what steps were taken in the finite element model development to ensure that sufficient accuracy is achieved. Also, since the angle between some of the finite elements in the model appear to deviate from optimum angles for triangular and rectangular finite elements (e.g., Figure 3.8-34, lower right hand region of elevated slab), explain how it was assured that the results using such finite elements are still accurate.
3. The ANSYS finite element models of the RCB internals are shown in Figure 3.8-32 with the cut models visible in Figures 3.8-33 to 3.8-37 and Appendix 3E. While most of the internal structures use shell elements, clearly define which use solid brick type finite elements. Explain how the shell/solid interfaces are modeled and how does that approach ensure acceptable compatibility at the interface since solid elements do not have rotational degrees of freedom. Explain how solution accuracy is ensured for both linear and nonlinear analyses (presumably used for accident thermal cases).
4. FSAR 3.8.3.4.1 discusses when creep, shrinkage, and differential settlement are considered. Explain the criterion used to distinguish when these effects need to be considered and how they are included in a particular analysis.

Response to Question 03.08.03-16:

1. Mesh density was refined to optimize computing time while providing accurate results. For the static model, a study of two critical sections within the Reactor Building internal structure (RBIS) was performed, which indicates that further mesh refinement produces negligible effect on design parameters (i.e., the required area of rebar); thus, selected mesh density is adequate. For the dynamic model, mesh selection was validated by comparing dynamic model response spectra with response spectra generated using a more refined mesh. Dynamic model response curves match curves generated using a more refined mesh; thus, the selected mesh density is adequate.
2. Triangular elements are used as filler elements during auto meshing. These degenerated linear triangular elements represent an insignificant fraction of the total number of elements in the Nuclear Island (NI) static model. Although triangular elements may not be as accurate as rectangular elements, they adequately transfer forces and moments and provide acceptable accuracy in selected applications. Because triangular elements tend to pick up

local concentrated stresses, results from these elements are carefully considered in section design.

The number of elements that violate shape limits for angles is minimal. During design, results from shape violating elements are considered. For those locations that contain shape violating elements, rather than using local results, an average is used.

3. The base and part of the primary shield wall (up to elevation +3.00 m) of the RBIS are modeled with SOLID45 elements. All other parts of the RBIS are modeled with SHELL43 elements.

   Multi point constraint approach, a well defined approach in ANSYS to confirm compatibility in shell-solid interface, is used for shell solid connection in the NI static model. This 3-D shell-solid assembly provides transition from a shell element region to a solid element region. No alignment is required between the solid element mesh and the shell element mesh. The contact surface or edge is built on the shell element side. The target surface is built on the solid element side. This approach establishes internal constraint equations to confirm compatibility of the rotational degrees of freedom for shell-solid connections.

   The NI static model is allowed to uplift at soil-structure interface. The soil structure interface is modeled with surface-to-surface contact options in ANSYS allowing uplift only. Both linear and nonlinear soil springs are considered for soil stiffness. Soil spring representation and nonlinearity involved in soil structure interface are the only sources of nonlinearity involved in the NI static model. Thus, NI static model analysis nonlinearity is identical to linear analysis for the multipoint constraint approach. Thermal analysis is performed on the RCB part of the NI static model that is modeled with solid elements only and does not include shell-solid assembly.

4. U.S. EPR NI Common Base Mat Structure analysis considers differential settlement for soil sites as addressed in U.S. EPR FSAR Tier 2, Section 3.8.5.5.1. NI Common Basemat Structure foundation design can accommodate tilt settlement up to 0.5 inch in 50 feet for any direction across the basemat, as described in U.S. EPR FSAR Tier 2, Section 2.5.4.10.2. Differential settlement within the NI basemat perimeter will be evaluated for soil sites as part of construction sequence planning.

   Creep, shrinkage, and other time dependent considerations are included in an analysis when they reduce the strength of the structure or cause serviceability concerns. For example, deflections of concrete slabs are handled in accordance with ACI 349-01 Section 9.5 ‘Control of deflections’ during design. Additionally, construction sequences are established to minimize early shrinkage concerns. Currently, no additional considerations are given to creep and shrinkage in the interior structure analysis.

FSAR Impact:

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

Question 03.08.03-17:

FSAR Section 3.8.3.1 “Description of the Internal Structures”, second paragraph, states:

> “The RB internal structures are Seismic Category I, except for miscellaneous structures such as platforms, stairs, guard rails, and other ancillary items. These miscellaneous structures are designed as Seismic Category II to prevent adverse impact on the Seismic Category I structures in the event of a SSE. Seismic classification of structures, systems and components (SSC) is addressed in Section 3.2.”

FSAR Section 3.2.1.2 “Seismic Category II,” states:

> “Per RG 1.29, some U.S. EPR SSCs that perform no safety-related function could, if they failed under seismic loading, prevent or reduce the functional capability of a Seismic Category I SSC, or cause incapacitating injury to main control room occupants during or following an SSE. These non-safety-related SSCs are classified as Seismic Category II.
>
> U.S. EPR SSCs classified as Seismic Category II are designed to withstand SSE seismic loads without incurring a structural failure that permits deleterious interaction with any Seismic Category I SSC or that could result in injury to main control room occupants. The seismic design criteria that apply to Seismic Category II SSCs are addressed in Section 3.7.
>
> Seismic Category II SSCs are subject to the pertinent quality assurance program requirements of 10 CFR Part 50, Appendix B.”

FSAR Section 3.7.2.3.3 “Seismic Category II Structures,” states:

> “The seismic analysis and design of Seismic Category II structures and members meets the requirements for Seismic Category I structures and members.”

FSAR Section 3.7.2.8 “Interaction of Non-Seismic Category I Structures with Seismic Category I Structures,” states:

> “In the case where damage to Category I SSCs cannot be precluded by the criteria above, the structure is classified as Seismic Category II and designed to the same criteria as Seismic Category I structures.”

FSAR Section 3.7.3.8 “Interaction of Other Systems with Seismic Category I Systems”, 1st paragraph (page 3.7-306), states:

> “The U.S. EPR uses state-of-the-art computer modeling tools for design and location of structures, subsystems, equipment, and piping. These same tools are used to minimize interactions of seismic and non-seismic components, making it possible to protect Seismic Category I subsystems from adverse interactions with non-seismic subsystem components. In the design of the U.S. EPR, the primary method of protection for seismic SSCs is isolation from each non-seismically analyzed SSC. In cases where it is not possible, or practical to isolate the seismic SSCs, adjacent non-seismic SSCs are classified as Seismic Category II and analyzed and supported so that an SSE event does not cause an unacceptable interaction with the Seismic Category I items. An

> interaction evaluation may be performed to demonstrate that the interaction does not prevent the Seismic Category I distribution subsystem from performing its safety-related function."

Based on the above, it appears that FSAR does not differentiate between Seismic Category I and Seismic Category II for seismic design/analysis and QA requirements. AREVA is requested to confirm this, and also to specifically describe the analysis methods and acceptance criteria that have been implemented for the seismic design of Seismic Category II miscellaneous structures inside containment, and other seismic Category I structures covered in FSAR Sections 3.8.3 through 3.8.5.

Response to Question 03.08.03-17:

U.S. EPR systems, structures and components (SSC) are classified according to their design basis safety-related function. Structures that support or protect safety-related SSC are designed to Category I standards. When a non-safety-related SSC is capable of interacting with a safety-related SSC, which could impair a design basis safety function, the non-safety-related SSC is classified as Category II. Category II SSC are analyzed and designed to prevent interaction with Category I SSC in a manner that could impair design basis safety functions but are not necessarily designed to Category I standards. In each case, the applied design codes and standards meet or exceed NRC requirements.

U.S. EPR FSAR Tier 2, Section 3.7.3.8 states that isolation is the primary method of protecting Seismic Category I SSC from interaction with non-seismically analyzed or non-safety-related SSC. Isolation methods include structural design, distance, barriers, and intervening structures that prevent impairment of the design basis safety function of protected SSC. When distance, barriers, or intervening structures cannot provide this function, Category II SSC are either completely or partially designed to Category I standards as necessary to prevent Category II SSC from interacting with Category I SSC in a manner that impairs a design basis safety function. 10 CFR 50 Appendix B is applied in accordance with regulatory requirements in all cases.

Category II piping requirements are described by ANP-10264NP-A "U.S. EPR Piping Analysis and Pipe Support Design Topical Report," which has been reviewed and approved by the NRC. ANP-10264NP-A shows acceptance criteria, analysis methods, and modeling techniques for ASME Class 1, 2, and 3 piping and pipe supports. This report covers Seismic Category I and Category II systems and also addresses non-seismic piping interaction with Seismic Category I piping.

The U.S. EPR FSAR will be revised to clarify that Category II SSC are not necessarily designed to Category I standards.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.7.2.3.3, 3.7.2.8, and 3A.2 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.05-1:

FSAR Section 3.8.5.1.1 states that the NI Common Basemat Structure foundation basemat is a cruciform shape that has outline dimensions of approximately 360 feet by 360 feet by 10 feet thick, a foundation basemat of lesser thickness will be considered for rock sites. It is the staff's understanding that the design certification is based on the details for the 10 foot basemat described in FSAR Section 3.8.5 and Appendix 3E. If a foundation basemat of lesser thickness will be used for rock sites, then all the details presented in the FSAR for the design of the 10 foot basemat need to be included in the FSAR for a basemat of lesser thickness. AREVA needs to either delete the statement "a foundation basemat of lesser thickness will be considered for rock sites" or present the complete design details for any other alternate foundation designs that they want the staff to certify. If rock will be considered in the design, then define what is meant by rock and provide the material properties attributed to rock that are applicable to the various analyses and design.

Response to Question 03.08.05-1:

U.S. EPR FSAR Tier 2, Section 3.8.5.1.1 will be revised to delete the statement "a foundation basemat of lesser thickness will be considered for rock sites."

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.8.5.1.1 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.05-8:

FSAR Section 3.8.5.4.2 describes a “second model” that was developed to evaluate the soil bearing pressures, sliding and overturning due to seismic events. AREVA is requested to provide the following information regarding this model:

1. Provide a figure showing the details of this model and explain what computer code is used to perform the analysis.
2. It is indicated that the properties of the model are established in a way that ultimately allows the model to respond in agreement with the SASSI analysis fundamental modes. Reference is made to FSAR Table 3.8.-15 which compares fundamental mode frequencies for three models. Clarify that the third column in this table are the results for the “second model” described above. Explain in detail the models discussed in the first two columns of this table, including how the soil was represented in the model in the first column and how the soil springs were determined in the model in the second column. Explain why the first column of this table refers to an “Equivalent to SASSI Analysis” rather than the SASSI model used for the SSI analysis discussed in FSAR Section 3.7.2. Provide a comparison of results (e.g., bearing pressures, sliding, and uplift) from each analysis corresponding to the three models shown in Table 3.8-15. Also, explain why soil case 1u was not included in the table since it is indicated that this soil case was part of the analytical study.
3. Because of a number of simplifying assumptions made in developing the “second model,” provide a comparison of the maximum soil bearing pressure, displacement, and location from the overall static NI building model and the “second model” used for bearing, sliding, and overturning analysis, for three load cases. The three load cases should correspond to the equivalent static seismic acceleration loads in the vertical, North-South, and East-West directions, applied in the same manner to both models.
4. Provide the basis for using a shear coefficient of 0.7 in the analysis. This should consider the potential for sliding at the various interfaces such as sliding between basemat and upper mud mat, mud mat and waterproofing material, lower mud mat and soil surface, and shear failure within the soil medium beneath the lower mud mat. Describe the extent to which this parameter is applicable to soil conditions found at most potential sites that may use the EPR design. It is also noted that the above reported coefficient of friction appears to be the static coefficient of friction. Since the analysis concludes that the structure slides, the full static coefficient of friction should not be used.
5. It is indicated that full passive pressure is assumed to occur at displacement of 1% of the depth of burial of the foundation depth. This mobilization displacement is clearly a subjective value and on other generic designs numbers of the order of 2% were used. Provide a discussion of the impact of the sensitivity of the computed results to these assumptions.
6. It is indicated that damper elements were obtained from the SASSI results to include with the spring results in the simplified sliding/overturning studies. Clarify what SASSI results are being referred to. Also, provide the following information regarding the damping elements: (a) Were these parameters generated for every soil case?, (b) Were they generated from half-power frequency considerations?, (c) Were separate dampers developed for horizontal and vertical springs?, (d) Were damping parameters selected as functions of location in the basemat as are the spring values?, (e) Were different dampers selected as a function of frequency?, (f) Were the results sensitive to the selection of these parameters?
7. It is stated that the “second model” is excited by simultaneous application of “three EUR seismic transients” that are simultaneously applied to the base of the soil elements, for soil

cases 1u, 2sn4u, 2n3u, 2u, 4u and 5a representing soft, medium and hard sites. Identify the specific three transient motions that were used, location in the soil media where these transients were developed, and where they are described in the FSAR. Explain why the application of these transients at the bottom of the single layer of soil brick elements is appropriate. Provide a description of how they were developed and confirm that these three transients are statistically independent based on the criterion in FSAR 3.7.1.1.2. Furthermore, explain why other soil cases used in the analysis of the EPR were not considered for this analysis.

8. The results of the analyses are summarized in FSAR Table 3.8-16 and discussed in FSAR Section 3.8.5.5.1. It is stated that these results are sufficiently small so that they can be considered inconsequential with respect to sliding and overturning. This conclusion is too qualitative and does not provide sufficient information to demonstrate that the required factors of safety specified in FSAR Table 3.8-11 have been met. Provide a quantitative basis to demonstrate why the sliding and uplift values presented in Table 3.8-16 are acceptable and, as a result meet the required factors of safety in FSAR Table 3.8-11. One approach might be to raise the level of earthquake to the required safety factor of 1.1 and perform a calculation to show that the structure does not overturn and the sliding is sufficiently small such that soil failure does not occur. This evaluation should also demonstrate that the actual soil pressures calculated on the walls and vertical edge of the slab from this seismic analysis provide sufficient margin when compared to the soil passive pressures considered in design. Also, it should be demonstrated that the sliding and uplift that is predicted to occur, using the design earthquake loads (not 1.1 E'), do not have an effect on floor response spectra, building member forces, and other building design parameters, such as the effect of differential displacement on distribution systems exiting the NI common structure.
9. It is stated in FSAR Section 3.8.5.5.1, that because friction will not prevent sliding of the RB internal structures basemat above the containment liner, the surrounding concrete haunch wall is designed with sufficient capacity to resist the total base shear force. Explain how the base shear force is calculated and how the concrete haunch is designed to resist this load. Also, as discussed in item 8 above, provide a quantitative basis for the factor of safety against sliding after taking the effect of the haunch into account.
10. It is stated in FSAR Section 3.8.5.5.1 that the minimum factor of safety against overturning for the RB internal structures basemat above the containment liner is 1.22, occurring for soil case 2sn4u. Explain how this factor of safety is calculated.

Response to Question 03.08.05-8:

1. A new U.S. EPR dynamic analysis was performed using an embedded FEM SSI model to address sliding and overturning issues and the results of these analyses are provided in the Response to Question 03.08.05-8, Part 8. The second model as referenced in this question has been superseded by the new analyses and is no longer applicable.
2. The referenced model used for nonlinear time history analyses was replaced by an embedded FEM SSI analysis to address sliding and overturning issues. The second model referenced in this question was superseded by the new analyses and U.S. EPR FSAR Tier 2, Table 3.8-15 was removed as a result of the Response to Question 03.08.05-8, Part 8.
3. A new U.S. EPR dynamic analysis was performed using an embedded FEM SSI model to address sliding and overturning issues and the results of these analyses are provided in the Response to Question 03.08.05-8, Part 8. The second model and the simplifying

assumptions as referenced in this question have been superseded by the new analyses and are no longer applicable.

4. The static coefficient of friction at the soil-concrete interface is based upon the angle of internal friction and a roughened contact surface. The coefficient of friction of 0.5 and 0.7 used in the design are reflected in the response to Question 03.08.05-8, Part 8.
5. A new U.S. EPR dynamic analysis was performed using an embedded FEM SSI model to address sliding and overturning issues and the results of these analyses are provided in the Response to Question 03.08.05-8, Part 8. The second model as referenced in this question has been superseded by the new analyses and is no longer applicable.
6. A new U.S. EPR dynamic analysis was performed using an embedded FEM SSI model to address sliding and overturning issues and the results of these analyses are provided in the Response to Question 03.08.05-8, Part 8. Damper elements, as referenced in this question, are not used by the new analyses and are thus no longer applicable.
7. A new U.S. EPR dynamic analysis was performed using an embedded FEM SSI model to address sliding and overturning issues and the results of these analyses are provided in response to Question 03.08.05-8 part 8. The new analyses use simultaneous excitation and application of the three EUR seismic transients.
8. New FEM SSI analyses have been performed on the NI structure using fully embedded conditions. The FEM SSI linear analyses consider a coefficient of friction (μ) of 0.5 and 0.7 representing saturated and dry conditions, respectively. The three bounding soil cases for sliding and overturning are used in these analyses. Instantaneous demand/capacity ratios to determine sliding and overturning safety-related factors are generated by time history methods.

   U.S. EPR FEM SSI-SASSI analyses model the tendon gallery as a shear key, sliding friction on the bottom of the basemat, and passive pressure on one side wall to resist seismic induced shear forces. Cohesive or friction forces are omitted from the analyses in any area where soil may de-bond from a wall. The minimum factor of safety against sliding is 1.16. The minimum factor of safety against overturning is 1.78.

   U.S. EPR FSAR Tier 2, Sections 2.5.4.10.1, 3.8.4.4.2, 3.8.5.1, 3.8.5.4.1, 3.8.5.5, 3.8.5.5.1, Tables 3.8-12, 3.8-15, and 3.8-16 are revised to reflect changes in methodology and the resulting conditions.
9. U.S. EPR FSAR Tier 2, Section 3.8.5.5.1 states, "Because friction will not prevent sliding, the surrounding concrete haunch wall is designed with sufficient capacity to resist the total base shear force."

   As addressed in the Response to RAI 155 Supplement 1, Question 03.08.01-2, base shear was obtained from a separate Finite Element Model (FEM) developed specifically to investigate Reactor Building internal structures (RBIS) stability. This model shows that only two load combinations contain loads related to RBIS lateral stability. These load combinations are then reduced by removing independent loads not applicable to RBIS stability analysis. The two controlling load combinations are reduced to:
   - B-05 $D + L + F + E' + R_a$
   - H-05 $D + F + E'$

These two load combinations are then expanded to account for directionality of the various independent loadings (i.e., 100-40-40 combinations of E' and reversible value of R_a). The Response to RAI 155 Supplement 1, Question 03.08.01-2 further addresses permutations of load combination B-05 and H-05. When loads are applied to the model, ANSYS post-processing files are generated to obtain reactions at the RBIS base. These reactions include:

- R_{hx} = horizontal reaction in the x-direction
- R_{hy} = horizontal reaction in the y-direction

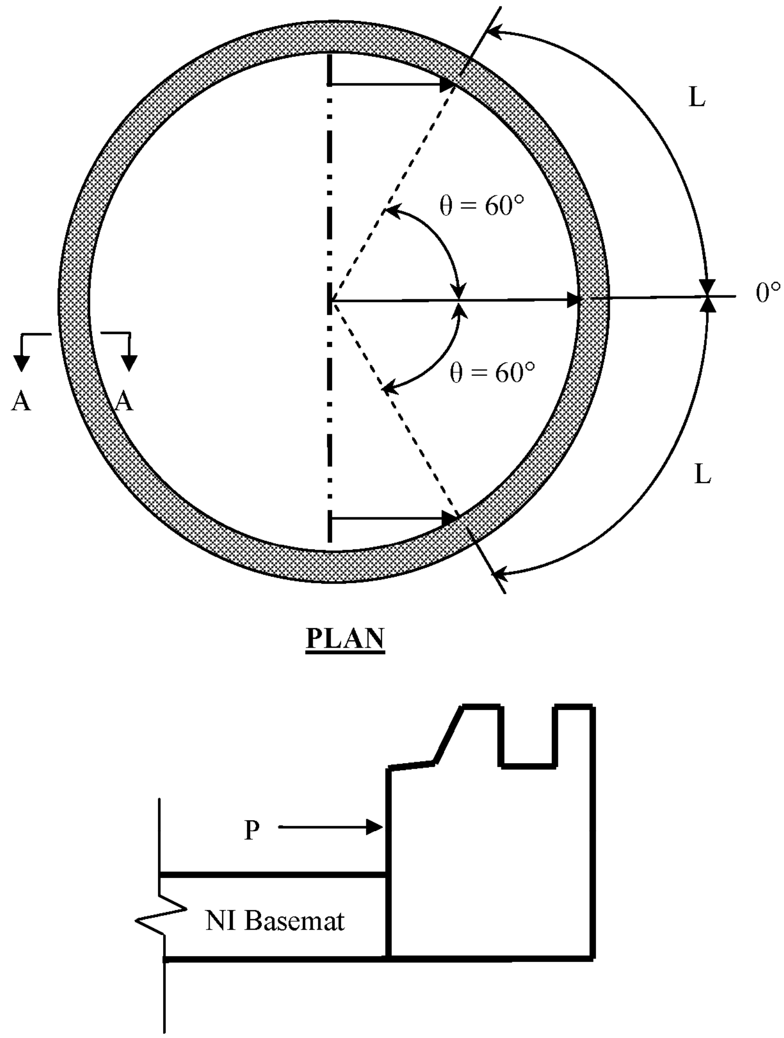
The base shear (i.e., sliding load) is calculated as:

$$F_{slide} = (R_{hx}^2 + R_{hy}^2)^{1/2}$$

F_{slide} is calculated for each applicable permutation of loading combinations B-05 and H-05. The maximum value of F_{slide} is selected and used as the additional lateral load to be resisted by the haunch wall.

The lateral load (F_{slide}) is distributed over 120° arc (i.e., modeled similar to a pipe sitting on a pipe saddle) where the distribution varies with the cosine of theta. Thus, F_{slide} is converted to a distributed load, in terms of kip/ft acting on the gusset. Figure 03.08.05-8(Part 9)-1 shows the plan view.

Figure 03.08.05-8(Part 9)-1—RBIS Lateral Load Distribution

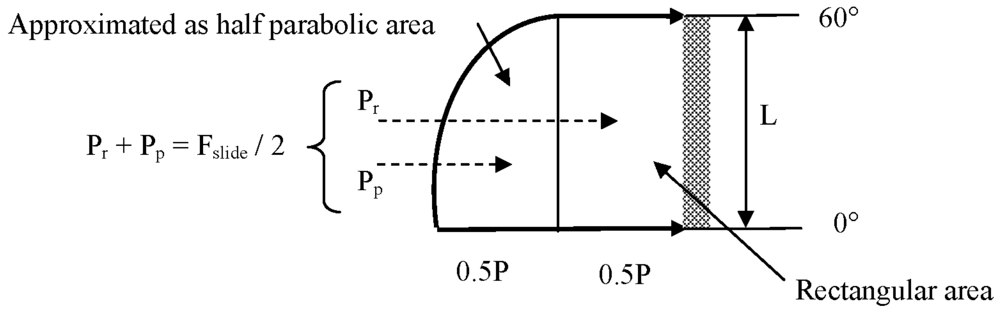


SECTION A-A
Cross-section of Gusset

The maximum value of the distributed load occurs at cos(0°) = 1P and the minimum occurs at cos(60°) = 0.5P, where P is the peak loading value. To determine peak loading the following are performed:

- Half of F_{slide} is applied to each 60° "flattened out" section of the gusset, as shown in Figure 03.08.05-8(Part 9)-2.
- The distribution is approximated as two areas, a rectangle and half a parabola as shown in Figure 03.08.05-8(Part 9)-2.
- The resultant of the two areas is set equal to half of F_{slide} (e.g., $P_r + P_p = F_{slide} / 2$).
- The equation was solved for the peak loading (P).

Figure 03.08.05-8(Part 9)-2—Distributed Load on Half the 120° Gusset Section



Note: P is in terms of kip/ft

Design of the haunch wall is performed in accordance with ASME BPV, Code Section III, Division 2, Article CC-3000. Design loadings include those loads obtained from the global NI FEM, plus peak loading (P) due to the sliding load (F_{slide}).

A quantitative basis for the factor of safety against sliding (FS_{slide}) after taking the effect of the haunch wall into account will be provided in the Response to RAI 155, Supplement 6, Question 03.08.04-6.

10. The Response to RAI 155, Supplement 1, Question 03.08.01-2 described the methodology for determining the RBIS basemat factor of safety against overturning (FS_{ot}).

FSAR Impact:

U.S. EPR FSAR Tier 2, Sections 2.5.4.10.1, 3.8.4.4.2, 3.8.5.1.1, 3.8.5.4.1, 3.8.5.4.2, 3.8.5.5, and 3.8.5.5.1, Tables 3.8-12, 3.8-15, and 3.8-16 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.05-10:

Section 3.8.5.5.1 indicates that bearing pressure demands under the NI Common Basemat structures are 22 ksf for static loads and 25 ksf for dynamic load conditions. For other Category 1 foundations, FSAR Sections 3.8.5.5.2 and 3.8.5.5.3 state that the maximum bearing pressures under static and dynamic loading conditions “will be performed” to confirm that applicable acceptance criteria are met. Provide the maximum bearing pressures under static and dynamic loading conditions for the other Category I foundations, and include them in FSAR Section 3.8.5 and the other applicable sections of the FSAR.

Response to Question 03.08.05-10:

Emergency Power Generating Buildings (EPGB) foundation basemat and the Essential Service Water Building (ESWB) foundation basemat maximum bearing pressures under static and dynamic loading conditions are provided in the Response to Question 03.08.05-12. Wind load combination is not the limiting condition for EPGB or ESWB stability. Factors of safety against overturning, sliding, and flotation are greater than or equal to 1.1.

The following U.S. EPR FSAR revisions implement values provided in the Response to Question 03.08.05-12:

1. The second paragraph of U.S. EPR FSAR Tier 2, Section 3.8.5.5.2 will be revised to state:

   “Maximum soil bearing pressures under the EPGB foundation basemat are 3,800 pounds per square foot for static loading conditions, and 10,800 pounds per square foot for dynamic loading conditions.”

2. U.S. EPR FSAR Tier 2, Section 3.8.5.5.2 will be revised by adding the following to address factors of safety against overturning, sliding, and flotation:

   “The factors of safety against overturning, sliding, and flotation are each greater than or equal to 1.1.”

3. U.S. EPR FSAR Tier 2, Section 3.8.5.5.3 will be revised to state:

   “Maximum soil bearing pressures under the ESWB foundation basemat are 17,800 pounds per square foot for static loading conditions, and 28,200 pounds per square foot for dynamic loading conditions.”

4. U.S. EPR FSAR Tier 2, Section 3.8.5.5.3 will be revised to address factors of safety against overturning, sliding, and flotation:

   “The factors of safety against overturning, sliding, and flotation are each greater than or equal to 1.1.”

Factors of safety against overturning, sliding, and flotation are each greater than or equal to 1.1, where the safety factors of 1.5 for the wind load combination are determined not to govern for stability.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.8.5.5.2 and 3.8.5.5.3 will be revised as described in the response and as indicated on the enclosed markup.

Question 03.08.05-12:

FSAR Section 3.8.5.5.2 for the EPGB and FSAR Section 3.8.5.5.3 for the ESWB state that the evaluation of the foundation basemats for maximum bearing pressures under static and dynamic loading conditions, settlements, flotation, sliding and overturning will be performed to confirm that applicable acceptance criteria are met. For each of these structures provide this information and include it in the FSAR. If it is currently not available, explain when it will be available for review by the staff and included in the FSAR.

Response to Question 03.08.05-12:

The static and dynamic bearing values shown below will be added to U.S. EPR FSAR Tier 1, Table 5.0-1 and U.S. EPR FSAR Tier 2, Table 2.1-1 as requested. U.S. EPR settlement limitations are based on structural requirements and are limited to a differential settlement of 0.5 inch in 50 feet for the design certification. Actual settlement of structures is dependent on site-specific soil characteristics and project sequencing implemented during construction. Additional details regarding settlement are addressed in the Response to RAI 155, Supplement 4, Question 03.08.05-15. U.S. EPR FSAR Tier 2, Table 3.8-11 provides acceptance criteria for floatation and safe shutdown earthquake (SSE) induced sliding and overturning. Corresponding minimum EPGB and ESWB safety-related factors are provided for each structure as follows:

EPG Building

Maximum Static Bearing (Localized)	3.8 ksf at bottom of basemat
Average Static Bearing	2.7 ksf at bottom of basemat, total area
Maximum Static + Dynamic Bearing	10.8 ksf at bottom of basemat, total area
Factor of Safety Against Floatation	9.6
Factor of Safety Against Sliding	1.1
Factor of Safety Against Overturning	61.9

ESW Building

Maximum Static Bearing (Localized)	17.8 ksf at bottom of basemat
Average Static Bearing	5.5 ksf at bottom of basemat, total area
Maximum Static + Dynamic Bearing	28.2 ksf at bottom of basemat
Factor of Safety Against Floatation	3.3
Factor of Safety Against Sliding	1.4
Factor of Safety Against Overturning	1.7

FSAR Impact:

U.S. EPR FSAR Tier 1, Section 2.1.1 Subsection 3.6, Section 2.1.5 Subsection 3.6, Table 2.1.1-4 item 3.6, Table 2.1.5-3 item 3.6, and Table 5.0-1, and U.S. EPR FSAR Tier 2, Table 2.1-1 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.05-18:

A review of EPR FSAR Tier 1, Table 5.0-1, Site Parameters for the U.S. EPR Design and Tier 2, Table 2.1-1, U.S. EPR Site Design Envelope shows that a number of site parameters used in the analysis and design of structures are not included. Some examples include the bearing capacities for all Seismic Category I structures (not just the NI basemat), the dynamic bearing capacities (not just the static bearing capacity), soil parameters such as the soil minimum friction angle of 35 degrees, permissible horizontal and vertical variation in soil properties, and total building settlements beneath each Seismic Category I structure (not just relative displacements). AREVA is requested to review all important analysis and design parameters used in the calculations and ensure that these parameters are included in these two tables, or provide the technical justification for excluding them.

Response to Question 03.08.05-18:

Minimum static and dynamic bearing capacities are addressed in the Response to Question 03.08.05-12. Settlement criteria were addressed in the Response to RAI 155, Supplement 4, Question 03.08.05-15. U.S. EPR FSAR Tier 2, Table 2.1-1 provides the coefficient of friction required for stability.

Local soil conditions, both vertically and horizontally, are rarely homogeneous or isotropic. Soil properties assumed for design certification are based on conservative lower bound soil conditions that are intended to be bounding for design considerations and improvement of these values will increase inherent structural safety-related factors. Site-specific soil conditions are to be evaluated to establish that U.S. EPR design assumptions remain bounding, as required by U.S. EPR FSAR Tier 2, Table 1.8-2, COL item 2.5-10 shown in U.S. EPR FSAR Tier 2, Section 2.5.4.10.3.

FSAR Impact

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

U.S. EPR Final Safety Analysis Report Markups

3.1b Decoupling of SB 2/3 and FB internal structures from their outer external hazards barrier walls, at their exterior walls along the entire wall length and the upper ceiling, and from the RSB above elevation 0 feet, 0 inches.

3.2 The NI site grade level is located between 12 inches and 18 inches below the finish floor elevation at the ground entrances.

3.3 The NI structures include barriers for post-accident radiation shielding as described in Table 2.1.1-3.

3.4 A pipe break hazards analysis summary exists that concludes the plant can be shut down safely and maintained in cold safe shutdown following a pipe break with loss of offsite power.

3.5 Essential SSCs in RB, SBs and FB rooms listed in Table 2.1.1-6 are protected from the dynamic effects of pipe breaks.

03.08.05-12

3.6 ~~Portions of NI Seismic Category I structures located below grade elevation are protected from external flooding by waterstops, water tight seals and waterproofing.~~ NI Seismic Category I structural walls or floors having exterior penetrations located below grade elevation are protected against external flooding by watertight seals.~~Portions of Seismic Category I structures that are located below grade elevation and exposed to aggressive soil or groundwater conditions will use waterstops, water tight seals, and waterproofing materials as required to mitigate deterioration.~~

3.7 The NI structures have key design dimensions that are confirmed after construction.

4.0 Interface Requirements

There are no interface requirements for the NI Structures.

5.0 Inspections, Tests, Analyses, and Acceptance Criteria

Table 2.1.1-4 lists the NI ITAAC.

Table 2.1.1-4—Nuclear Island ITAAC (~~3~~ 4 Sheets)

	Commitment Wording	Inspections, Tests, Analyses	Acceptance Criteria
3.4	A pipe break hazards analyses summary exists that concludes the plant can be shut down safely and maintained in cold safe shutdown following a pipe break with loss of offsite power.	A pipe break hazards analysis will be performed.	A pipe break hazards analyses summary exists that concludes the plant can be shut down safely and maintained in cold safe shutdown following a pipe break with loss of offsite power and confirms whether: • Piping stresses in the RCB penetration area are within allowable stress limits. • Pipe whip restraints and jet shield designs can mitigate pipe break loads. • Loads on safety-related SSCs are within design load limits. • SSCs are protected or qualified to withstand the environmental effects of postulated failures.
3.5	Essential SSCs in RCB, SBs and FB rooms listed in Table 2.1.1-6 are protected from the dynamic effects of pipe breaks.	a. An analysis of essential SSCs in the rooms listed in Table 2.1.1-6 will be performed to determine the protective features required for the dynamic effects of pipe breaks. b. An inspection of as-installed features providing protection for essential systems and components from the effects of piping failures versus construction drawings of protective features identified in the analysis of part (a) will be performed.	a. Essential SSCs in rooms listed in Table 2.1.1-6 are protected from the dynamic effects of pipe breaks. b. Essential SSCs in rooms listed in Table 2.1.1-6 are protected from the dynamic effects of pipe breaks and the features providing protection conform to the construction drawings.
3.6	NI Seismic Category I structural walls or floors having exterior penetrations located below grade elevation are protected against external flooding by	An inspection of NI Seismic Category I exterior structural wall and floor penetrations located below grade elevation will be performed~~An inspection of the NI Seismic Category I~~	Watertight seals exist for exterior penetrations of NI Seismic Category I structural walls and floors located below grade elevation.~~Portions of NI Seismic Category I structures~~

03.08.05-12

Table 2.1.1-4—Nuclear Island ITAAC (~~3~~ 4 Sheets)

03.08.05-12

	Commitment Wording	Inspections, Tests, Analyses	Acceptance Criteria
	watertight seals.~~Portions of NI Seismic Category I structures located below grade elevation are protected from external flooding by waterstops, water tight seals and waterproofing.~~	~~structures will be performed.~~	~~located below grade elevation are protected from external flooding by waterstops, water tight seals and waterproofing.~~
3.7	The NI structures have key design dimensions that are confirmed after construction.	An inspection of key dimensions of the as-installed NI structures will be performed. During construction, deviations from the approved design will be analyzed for design basis loads.	Deviations from the key dimensions and tolerances specified in Table 2.1.1-1 and Table 2.1.1-2 are reconciled and the as-installed NI structures will withstand the design basis loads without loss of structural integrity and safety related functions.

3.5 The ESWB structures are Seismic Category I and are designed and constructed to withstand design basis loads, as specified below, without loss of structural integrity and safety-related functions.

- Normal plant operation (including dead loads, live loads, lateral earth pressure loads, hydrostatic loads, hydrodynamic loads, and temperature loads).
- Internal events (including internal flood loads, accident pressure loads, accident thermal loads, accident pipe reaction, and pipe break loads—including reaction loads, jet impingement loads, and missile impact loads).
- External events (including rain, snow, flood, tornado, tornado-generated missiles, and earthquake).

03.08.05-12

3.6 ~~Portions of the ESWB structures located below grade elevation are protected from external flooding by waterstops, watertight seals and waterproofing.~~ ESWB structural walls or floors having exterior penetrations located below grade elevation are protected against external flooding by watertight seals.

3.7 The ESWB structures have key design dimensions that are confirmed after construction.

4.0 Interface Requirements

There are no interface requirements for the ESWB structures.

5.0 Inspections, Tests, Analyses, and Acceptance Criteria

Table 2.1.5-3 lists ESWB ITAAC.

Table 2.1.5-3—Essential Service Water Building ITAAC (3 Sheets)

	Commitment Wording	Inspections, Tests, Analyses	Acceptance Criteria
3.6	ESWB structural walls or floors having exterior penetrations located below grade elevation are protected against external flooding by watertight seals.~~Portions of ESWB structures located below grade elevation are protected from external flooding by waterstops, watertight seals, and waterproofing.~~	An inspection of ESWB exterior structural walls and floors located below grade will be performed.~~An inspection of the as-installed ESWB structures will be performed.~~	Watertight seals exist for exterior penetrations of ESWB structural walls and floors located below grade elevation.~~Portions of ESWB structures located below grade elevation are protected from external flooding by waterstops, watertight seals, and waterproofing.~~
3.7	The ESWB structures have key design dimensions that are confirmed after construction. 03.08.05-12	An inspection of key dimensions of the as-installed ESWB structures will be performed. During construction, deviations from the approved design will be analyzed for design basis loads.	Deviations from the key dimensions and tolerances specified in Tables 2.1.5-1 and 2.1.5-2 are reconciled and the as-installed ESWB structures will withstand the design basis loads without loss of structural integrity and safety related functions.

Next File

Table 5.0-1—Site Parameters for the U.S. EPR Design (4 Sheets)

Tornado	
Parameter	**Value(s)**
Tornado (maximum speed, pressure drop, radius of maximum rotational speed, rate of pressure drop, missile spectra)	Maximum tornado wind speed of 230 mph. Maximum rotational speed of 184 mph. Maximum tornado pressure drop of 1.2 pounds per square inch at 0.5 psi per second. Radius of maximum rotational speed is 150 ft
Soil	
Parameter	**Value(s)**
Soil properties: Minimum shear wave velocity	Minimum shear wave velocity (low strain best estimate average value at bottom of basemat) of 1000 feet per second.
Minimum static bearing capacity	Minimum static bearing capacity of 22,000 lb/ft^2 in localized areas at the bottom of the Nuclear Island basemat and 15,000 lb/ft^2 on average across the total area of the bottom of the Nuclear Island basemat.
Minimum dynamic bearing capacity 03.08.05-12	Minimum static bearing capacity of 3,800 lbs/ft^2 in localized areas at the bottom of the EPGB basemat and 2,700 lbs/ft^2 on average across total area at the bottom of the EPGB basemat. Minimum static bearing capacity of 17,800 lbs/ft^2 in localized areas at the bottom of the ESWB basemat and 5,500 lbs/ft^2 on average across total area at the bottom of the ESWB basemat. Minimum dynamic bearing capacity of ~~34,560~~26,000 lb/ft^2 at the bottom of the Nuclear Island basemat. Minimum dynamic bearing capacity of 10,800 lbs/ft^2 at the bottom of the EPGB basemat. Minimum dynamic bearing capacity of 28,200 lbs/ft^2 at the bottom of the ESWB basemat.
Liquefaction potential	No potential for liquefaction.
Maximum ground water level	Maximum ground water level is 3.3 ft below grade.
Maximum Differential Settlement (across the basemat)	1/2 inch in 50 ft in any direction
Slope Failure Potential	No slope failure potential is considered in the design of safety-related SSC for U.S. EPR design certification.

Table 2.1-1—U.S. EPR Site Design Envelope
Sheet 2 of 7

U.S. EPR Site Design Envelope	
Soil (Refer to Section 2.5)	
Minimum Static Bearing Capacity	22 ksf in localized areas at the bottom of the Nuclear Island basemat and 15 ksf on average across the total area of the bottom of the Nuclear Island basemat. 03.08.05-12 • 3,800 lbs/ft^2 in localized areas at the bottom of the EPGB basemat and 2,700 lbs/ft^2 on average across total area at the bottom of the EPGB basemat. • 17,800 lbs/ft^2 in localized areas at the bottom of the ESWB basemat and 5,500 lbs/ft^2 on average across total area at the bottom of the ESWB basemat.
Minimum Dynamic Bearing Capacity	• ~~34.56 ksf~~26,000 psf at the bottom of the Nuclear Island basemat. • 10,800 lbs/ft^2 at the bottom of the EPGB basemat. • 28,200 lbs/ft^2 at the bottom of the ESWB basemat.
Minimum Shear Wave Velocity (Low strain best estimate average value at bottom of basemat)	1000 fps
Liquefaction	None
Maximum Differential Settlement (across the basemat)	1/2 inch in 50 feet in any direction
Slope Failure Potential	No slope failure potential is considered in the design of safety-related SSC for U.S. EPR design certification.
Maximum Ground Water	3.3 ft below grade

hysteretic damping properties are not explicitly considered. The COL applicant will address site-specific response of soil and rock to dynamic loading, including the determination of strain-dependent modulus-reduction and hysteretic damping properties.

2.5.4.8 Liquefaction Potential

The design of the U.S. EPR assumes that the plant is not founded on liquefiable materials (GDC 2).

The COL applicant will address site-specific liquefaction potential. As stated in Section 3.7.1, the evaluation of liquefaction is performed for the seismic level of the site-specific SSE.

2.5.4.9 Earthquake Site Characteristics

Section 3.7.1 describes the seismic design basis for the U.S. EPR. Section 2.5.2 presents a brief summary of the seismic design basis.

Site-specific earthquake site characteristics will be described by the COL applicant.

2.5.4.10 Static Stability

Static stability pertaining to bearing capacity and settlement for the U.S. EPR is described in the following section. Additional information is provided in Section 3.8.5 for the foundations of Seismic Category I structures.

2.5.4.10.1 Bearing Capacity

03.08.05-8 (Part 8)

The maximum bearing pressure under static loading conditions for the foundation basemat beneath the NI Common Basemat Structures is 22,000 lb/ft^2, which includes the dead weight of the structure and components and 25 percent of the live load. The maximum bearing pressure under safe shutdown earthquake loads combined with other loads, as described in Section 3.8.5, is ~~34,560~~26,000 lb/ft2. Refer to Appendix 3E for details of these bearing pressures under the basemat (GDC 2).

A COL applicant that references the U.S. EPR design certification will verify that site-specific foundation soils beneath the foundation basemats of Seismic Category I structures have the capacity to support the bearing pressure with a factor of safety of 3.0 under static conditions.

2.5.4.10.2 Settlement

Safety-related structures, systems and components are housed primarily in structures supported by the foundation basemat for the NI Common Basemat Structures and independent foundation basemats for the EPGBs and the ESWBs. The design of the

the EPGB, modifications are made to the slab stiffness at elevation +51 ft, 6 inches to accurately represent the stiffness of composite beams. For the ESWB, two additional modeling features are used:

- Space frame elements are used to simulate the fill support beams and the distribution header supports.
- Rigid water mass, calculated in accordance with the procedure in ASCE 4-98, Reference 1 and ACI 350.3 (Reference 3), is lumped on the appropriate basin walls. Both low water and high water level are separately considered.

Figure 3.7.2-57—Isometric View of GTSTRUDL FEM for Emergency Power Generating Building and Figure 3.7.2-58—Section View of GTSTRUDL FEM for Emergency Power Generating Building illustrate an isometric view and a section view of the 3D FEM of the EPGB. Figure 3.7.2-59—Isometric View of GTSTRUDL FEM for Essential Service Water Building and Figure 3.7.2-60—Section View of GTSTRUDL FEM for Essential Service Water Building, depict the 3D FEM of the ESWB.

For walls and slabs, adjustment is made to account for cracked section properties. Specifically, a value of $0.5E_c$ is typically used to determine out-of-plane stiffness of these concrete walls and floors. There remains the possibility that the wall stiffness may be between the fully cracked and uncracked conditions. To bound the dynamic response in the SSI analysis, SDOF out-of-plane oscillators based on uncracked section properties are included in the SASSI model at the center of selected slabs and walls.

3.7.2.3.3 Seismic Category II Structures

03.08.03-17

~~The seismic analysis and design of Seismic Category II structures and members meets the requirements for Seismic Category I structures and members~~Conventional seismic structures with the potential of impairing the safety function of a Seismic Category I SSC will be classified as Seismic Category II in accordance with the criteria identified in Section 3.2.1.2. Seismic Category II SSC will be designed and analyzed to the codes and standards associated with conventional seismic SSC but the analyses will also consider the SSE event. Category II structures are to be seismically analyzed and supported to prevent unanalyzed loads from being transferred to a protected Category I SSC. The procurement, quality control, and QA requirements for Category II SSC will be performed to the conventional siesmic codes and standards unless more stringent requirements are identified.

3.7.2.3.4 Conventional Seismic (CS) Structures

The analysis and design of Conventional Seismic building structures shall be in accordance with the applicable requirements of the International Building Code (IBC) (Reference 4). Structural interaction between Conventional Seismic building structures that have the potential to interact with Seismic Category I structures is

SSC to perform their safety functions. The basis for the seismic interaction assessment guidelines given below is the prevention of structure-to-structure impact.

- The collapse of the non-Category I structure does not cause the non-Category I structure to strike a Category I SSC.
- The collapse of the non-Category I structure does not impair the integrity of seismic Category I SSC, nor result in incapacitating injury to control room occupants.

03.08.03-17

- ~~Conventional Seismic structures that have the potential to interact with Seismic Category I structures are assessed for collapse potential under SSE and tornado loading (acting independently). Seismic demand for the SSE is computed in accordance with ASCE 4-98, Reference 1 and the methodologies in Section 3.7.2 Seismic load combinations are developed in accordance with ASCE 43-05 (Reference 5), using a limiting acceptable condition for the structure characterized as short of collapse, but structurally stable (i.e., Seismic Design Category 5 - Limit State A) as specified in the Standard.~~The non-Category I structure will be analyzed and designed to prevent its failure under SSE conditions such that the margin to safety is equivalent to that of a Category I structure.
- For Conventional Seismic structures that have the potential to interact with Seismic Category I structures, the combined seismic deflection is less than the separation distance (i.e., gap) between the structures.
- In the case where damage to Category I SSC cannot be precluded by the criteria above, the structure is classified as Seismic Category II and designed to the same criteria as Seismic Category I structures.

The seismic interaction criteria and assessment guidelines are summarized in Table 3.7.2-29—Seismic Structural Interaction Criteria for Building Structures. The Vent Stack, NAB, Access Building (AB), and the Turbine Building (TB) are Conventional Seismic structures that have potential to interact with the NI Common Basemat Structures. Results of the seismic interaction assessment for those structures are presented below, with associated discussions of the Radioactive Waste Processing Building (RWPB) and Fire Protection Storage Tanks and Building.

Vent Stack

The vent stack is described in Section 3.7.2.4.2 as a steel structure approximately 100 ft high located on top of the stair towercase structure between the FB and SB 4 (see Figure 3B-1). The vent stack is classified as Seismic Category II and designed to the same requirements as Seismic Category I structures. The stack is also designed for design basis tornado loading. Therefore, the vent stack has no potential for adverse interaction with the NI Common Basemat Structures.

Section 6.2.6 contains a description of the associated leak-rate test procedure, Containment Integrated Leakage Rate Test (CILRT). Containment pressure testing will occur in conjunction with the CILRT.

Sufficient physical access is provided in the annulus between the RCB and the RSB to perform inservice inspections on the outside of the containment. Space is available inside of the RCB to perform inservice inspections of the liner plate. Gaps are provided between the liner and RB internal structures concrete structural elements, which provide space necessary to inspect the liner at wall and floor locations inside containment. Inservice inspection of the embedded portion of the containment liner and the surface of the concrete containment structure covered by the liner are exempted in accordance with Section III of the ASME BPV Code for Class CC components.

3.8.2 Steel Containment

The steel containment section describes major RCB penetrations and portions of penetrations not backed by structural concrete that are intended to resist pressure. Section 3.8.1 describes the concrete RCB.

3.8.2.1 Description of the Containment

03.08.02-2

Steel items that are part of the RCB pressure boundary and are not backed by concrete include the equipment hatch, airlocks, construction opening, piping penetration sleeves, electrical penetration sleeves, and fuel transfer tube penetration sleeve. Section 3.8.1.1 describes RCB steel items that are backed by concrete, such as the liner plate.

3.8.2.1.1 Equipment Hatch, Airlocks, and Construction Opening

The equipment hatch, illustrated in Figure 3.8-25 is a welded steel assembly with a double-gasketed, flanged, and bolted cover. Provision is made for leak testing of the flange gaskets by pressurizing the annular space between the gaskets. The cover for the equipment hatch attaches to the hatch barrel from inside of the RCB. The cover seats against the sealing surface of the barrel when subjected to internal pressure inside the RCB. The equipment hatch opens into the Seismic Category I FB, which provides protection of the hatch from external environmental hazards (e.g., high wind, tornado wind and missiles, and other site proximity hazards, including aircraft hazards and blasts). The equipment hatch barrel has an inside diameter of approximately 27 feet, 3 inches.

One personnel airlock and one emergency airlock are provided for personnel to access the RCB. Figure 3.8-26—Personnel Airlock, Emergency Airlock General Overview illustrates a typical arrangement for the airlocks. Each airlock is a welded steel assembly that has two doors, each with double gaskets. The airlocks open into containment so that internal pressure inside the RCB seats the doors against their sealing surfaces. Provision is made to pressurize the annular space between the gaskets during leak testing.

The doors mechanically interlock so that one door can not be opened unless the second door is sealed during plant operation. Provisions are made for deliberately overriding the interlocks by the use of special tools and procedures for ease of access during plant maintenance. Each door is equipped with valves for equalizing the pressure across the doors. The doors are not operable unless the pressure is equalized. Pressure equalization is possible from the locations at which the associated door can be operated. The valves for the two doors interlock so that only one valve can open at a time and only when the opposite door is closed and sealed. Each door is designed to withstand and seal against design and testing pressures of the containment vessel when the other door is open. A visual indication outside each door shows whether the opposite door is open or closed. In the event that one door is accidentally left open, provisions outside each door allow remote closing and latching of the opposite door.

The personnel airlock at [ ] opens into a [ ] which is a Seismic Category I structure. The emergency airlock opens into the [ ], which is a Seismic Category I structure. Therefore, both airlocks are protected from external environmental hazards (e.g., high wind, tornado wind and missiles, and other site proximity hazards, including aircraft hazards and blasts). The personnel airlock and the emergency airlock have inside diameters of approximately 10 feet, 2 inches.

03.08.02-2

The construction opening is located at [ ] and opens to the heavy load operating floor level from [ ] This passage serves as personnel and material access into the RB during construction. The construction opening has an outside diameter of approximately 9 feet, 6 inches. Upon completion of construction work, the cavity in the RCB is permanently sealed with a metal closure cap welded ~~in place metal closure cap~~to an embedded sleeve.

The equipment hatch, two airlocks, and construction opening closure cap and sleeve are designated as Class MC components in compliance with Article NE-3000 of the ASME BPV Code, Section III, Division I, and are stamped pressure vessels designed and tested in accordance with this code (GDC 1 and GDC 16).

3.8.2.1.2 Piping Penetration Sleeves

Piping penetrations through the RCB pressure boundary are divided into the following three general groups:

Security Sensitive Information redacted from this page for transmittal of an unrelated FSAR change ONLY.

03.08.02-2

- Equipment hatch, ~~and~~ airlocks, and construction opening closure cap and sleeve.
- Piping penetration sleeves.
- Electrical penetration sleeves.
- Fuel transfer tube penetration sleeve.

3.8.2.2.3 Design Criteria

The design of steel pressure retaining components of the RCB that are not backed by concrete complies with the following:

- Article NE-3000 of the ASME BPV Code 2004 Edition, Section III, Division 1 (GDC 1 and GDC 16).

3.8.2.2.4 Regulations

- 10 CFR 50, Licensing of Production and Utilization Facilities.
- 10 CFR 50, Appendix A – General Design Criteria for Nuclear Power Plants GDC 1, 2, 4, 16, and 50.
- 10 CFR 50, Appendix J – Primary Reactor Containment Leakage Testing for Water Cooled Power Reactors.

3.8.2.2.5 NRC Regulatory Guides

RGs applicable to the design and construction of steel portions of the RCB that resist pressure, but are not backed by structural concrete:

- RG 1.7, Revision 3.
- RG 1.57, Revision 1.
- RG 1.84, Revision 33.
- RG 1.136, Revision 3 (exception described in 3.8.1.3).
- RG 1.193, Revision 1.

3.8.2.3 Loads and Load Combinations

The U.S. EPR standard plant design loads envelope includes the expected loads over a broad range of site conditions. Design loads and loading combinations for steel portions of the RCB that are not backed by concrete are described in the following sections (GDC 1, GDC 2, GDC 4, GDC 16, and GDC 50). Section 3.8.1.3 addresses loads and loading combinations for design of the steel liner plate.

shells is to perform a non-linear analysis. Code class MC components are screened for cyclic service analysis according to the criteria given in Article NE-3221.5 of the ASME BPV Code.

Refer to Section 3.5.3 for a description of requirements for missile barrier design and ductility requirements applicable to the design of steel portions of the RCB.

The following sections provide individual descriptions of the design and analysis procedures performed to verify the structural integrity of the steel items. Section 3.8.1 addresses the design and analysis procedures used to qualify the RCB concrete structure for openings provided through the containment pressure boundary for these items. Containment ultimate capacity analysis results are described in Section 3.8.1.4.11, which includes evaluation of major containment steel penetrations.

3.8.2.4.1 Equipment Hatch, Airlocks, and Construction Opening

The equipment hatch described in Section 3.8.2.1.1 is supported entirely by the concrete shell of the RCB. The barrel of the equipment hatch is embedded in the concrete containment shell and welded at the periphery to the liner plate. The liner plate is thickened in the vicinity of the equipment hatch penetration. The equipment hatch cover is dished and stiffened by a reinforcing ring where it interfaces with the barrel of the equipment hatch.

The two airlocks described in Section 3.8.2.1.1 are supported by attachment to sleeves embedded in the concrete shell of the RCB and by supports attached to the RSB wall. These supports provide for differential movements of the containment and shield walls. The doors for both ends of the airlocks are flat, and the bulkhead ends of the components are dished.

The construction opening closure cap described in Section 3.8.2.1.1 is attached to and supported by a sleeve embedded in the concrete shell of the RCB. The closure cap is a dish shaped metal structure welded to the embedded sleeve flange.

03.08.05-2

The equipment hatch, airlocks, and construction opening closure cap and sleeve will be evaluated for the combinations of loads described in Section 3.8.2.3.2. Analyses and limits for the resulting stress intensities in the equipment hatch, airlocks, and the construction opening closure cap and sleeve will be designed in accordance with Articles NE-3130, NE-3200, NE-3325, and NE-3326 of Section III, Division 1 of the ASME BPV Code.

3.8.2.4.2 Piping, Electrical, and Fuel Transfer Tube Penetration Sleeves

The penetration sleeves are welded to the containment liner plate and are anchored to the RCB concrete shell. Penetration sleeves are subjected to various combinations of

Quality control for containment steel items conforms to Articles NE-2000, NE-4000, and NE-5000 of Section III, Division 1 of the ASME BPV Code (GDC 1).

Section 3.8.1.6 provides a description of welding requirements for steel items for the RCB, quality control for steel items for the RCB, and materials used for penetration sleeves, steel embedments, and corrosion retarding compounds.

Use of neoprene-based gaskets and seals are kept to a minimum because of the presence of fluoride or chloride ions and the increased potential for stress corrosion cracking.

Steel items such as the equipment hatch, airlocks, fuel transfer tube, and penetrations are prefabricated and installed as subassemblies during construction. No special techniques are used for construction of containment steel items not backed by concrete. Section 3.8.1.6 provides additional information of modular construction techniques used for the RCB.

3.8.2.7 Testing and Inservice Inspection Requirements

A SIT is performed for steel containment components not backed by concrete in accordance with Article NE-6000 of Subsection NE of the ASME BPV Code, Section III, Division 1 (GDC 1).

03.08.02-7

~~Testing and i~~Inservice ~~surveillance~~inspections for the steel ~~items consists of leakage testing of the containment and~~ pressure retaining subassemblies follow the requirements of the ASME BPV Code, Section XI, Subsection IWE with the additional requirements of 10 CFR 50.55a (GDC 1 and GDC 16). Section 6.2.6 describes the leakage tests and associated acceptance criteria.

3.8.3 Concrete and Steel Internal Structures of Concrete Containment

3.8.3.1 Description of the Internal Structures

RB internal structures consist of reinforced concrete walls and floors, steel framing members, and other concrete and steel structural elements that are located inside of the RCB. The RB internal structures provide support for components and radiation shielding for the RCS and refueling operations. The foundation basemat inside of the RCB supports the RB internal structures at the bottom interface. To prevent an interaction between the structures for design basis loading conditions, clearance is maintained between the containment wall and internal structures. RB internal structures important to safety are not shared with another unit (GDC 5).

03.08.03-17

The RB internal structures are Seismic Category I, except for miscellaneous structures such as platforms, stairs, guard rails, and other ancillary items. These miscellaneous structures are designed as Seismic Category II to prevent ~~adverse impact~~impairment of

03.08.03-17

the design basis safety function ~~on~~of the Seismic Category I ~~structures~~safety-related SSC in the event of a SSE. Seismic classification of structures, systems and components (SSC) is addressed in Section 3.2.

The following figures show the main levels of the RB internal structures and sectional views of the building:

- Figure 3.8-2—Reactor Building Plan at Elevation -20 Feet (top of the foundation basemat inside containment).
- Figure 3.8-3—Reactor Building Plan at Elevation -8 Feet (top of concrete at start of containment wall).
- Figure 3.8-4—Reactor Building Plan at Elevation +5 Feet (top of heavy floor for nuclear steam supply system (NSSS) component support).
- Figure 3.8-5—Reactor Building Plan at Elevation +17 feet (plan at centerline of reactor vessel piping nozzles).
- Figure 3.8-6—Reactor Building Plan at Elevation +29 feet (top of grating floor for component access).
- Figure 3.8-7—Reactor Building Plan at Elevation +45 feet (top of grating floor for component access).
- Figure 3.8-8—Reactor Building Plan at Elevation +64 feet (top of concrete operating floor).
- Figure 3.8-9—Reactor Building Plan at Elevation +79 feet (top of partial concrete floor).
- Figure 3.8-10—Reactor Building Plan at Elevation +94 feet (top of pressurizer cubicle).
- Figure 3.8-11—Reactor Building Section A-A.
- Figure 3.8-12—Reactor Building Section B-B.
- Figure 3.8-13—Reactor Building Section C-C.

The RB internal structures consist of the following major structures that support nuclear steam supply system (NSSS) components, provide access for plant operation and maintenance, and support safety-related functions of the plant:

- Reactor vessel (RV) support structure and reactor cavity.
- Steam generator (SG) support structures.
- Reactor coolant pump (RCP) support structures.

- 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).

- 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 303-0~~5~~0, Code of Standard Practice for Steel Buildings and Bridges (Reference 42).

03.08.03-4

- 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).~~

- AISC 348-~~04~~00/~~2004~~2000 RCSC, Specification for Structural Joints Using ASTM A325 and A490 Bolts (Reference 44).

- ANSI/AWS D1.1/D1.1M 2006, Structural Welding Code - Steel.

- ANSI/AWS D1.4-2005, Structural Welding Code - Reinforcing Steel.

- ANSI/AWS D1.6-1999, including January 6, 2005 update, Structural Welding Code – Stainless Steel.

- ANSI/AWS D1.8-2005, Structural Welding Code – Seismic Supplement (Reference 45).

- ASME Boiler and Pressure Vessel Code - 2004 Edition, Section III, Division 2 - Code for Concrete Reactor Vessels and Containments (GDC 1).

- ASME Boiler and Pressure Vessel Code - 2004 Edition, Section III, Division 1 – Nuclear Power Plant Components (GDC 1).

- ASME NOG-1-04, Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder).

3.8.3.2.2 Specifications

Industry standards (e.g., those published by the ASTM) are used to specify material properties, testing procedures, fabrication methods, and construction methods. Section 3.8.3.6 addresses the applicable standards used.

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).

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.

03.08.03-4

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~~00/~~2004~~2000 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.

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.

Fabrication and Placement

Fabrication and placement of reinforcing bars for RB internal structures is in accordance with ACI 349-2001, Chapter 7.

Welding conforms to the ASME BPV Code, Section III, Division 2, Subsection CC, as supplemented by RG 1.136 and AWS D1.4-2005 (GDC 1).

Mechanical splices are subject to the testing and acceptance criteria of ACI 349-2001, Section 12.14.3.

3.8.3.6.3 Structural Steel

03.08.03-4

Structural steel materials for the RB internal structures conform to ANSI/AISC N690-1994 including Supplement 2 (2004) and AISC 303-~~05~~00 (GDC 1).

Materials

Seismic Category I structural steel conforms to ASTM material specifications identified in ANSI/AISC N690, Section Q1.4.1. Materials for structural steel members include those listed in Table 3.8-8.

High strength bolting materials conform to ASTM A325 (Reference 54), or ASTM A490 (Reference 55). Other bolting materials conform to ASTM A307 (Reference 56).

Structural bolts conform to the ASTM material specifications identified in ANSI/AISC N690, Section Q1.4.3, or other materials identified in the AISC/RCSC. Bolting materials for structural steel include those listed in Table 3.8-9. Anchor rods conform to the material specifications in ASTM F1554 (Reference 46).

Structural bolts utilize nuts and washers as recommended by ASTM for the particular bolting material and as identified in AISC/RCSC. Structural bolting nut and washer materials for structural steel include those listed in Table 3.8-10—Structural Bolting Nut and Washer Materials.

Structural steel, steel pipe, or tubing used in composite compression members in Seismic Category I concrete structures conforms to the specifications in Section 3.5.6 of ACI 349-2001.

Welding materials conform to ANSI/AWS D1.1/D1.1M 2006, or ANSI/AWS D1.6-99, including the January 6, 2005 update, except as modified by ANSI/AISC N690, Sections Q1.17.1 and Q1.17.2.1. The compatibility of filler metal with base metal is specified in Table 3.1 of AWS D1.1.

Fabrication and Erection

Fabrication and erection of structural steel, welding, and bolting conforms to the following codes:

- ANSI/AISC N690-1994, Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2 (2004).

03.08.03-4

- AISC 348-~~04~~00/~~2004~~2000 RCSC, Specification for Structural Joints Using ASTM A325 and A490 Bolts.
- ANSI/AWS D1.1/D1.1M 2006, Structural Welding Code – Steel.
- ANSI/AWS D1.6-1999, including January 6, 2005 update, Structural Welding Code – Stainless Steel.
- ANSI/AWS D1.8 2005, Structural Welding Code – Seismic Supplement.

3.8.3.6.4 Quality Control

In addition to the quality control procedures addressed in Section 3.8.3.6.1, Section 3.8.3.6.2, and Section 3.8.3.6.3, refer to Chapter 17 for a description of the quality assurance program for the U.S. EPR (GDC 1).

3.8.3.6.5 Special Construction Techniques

The RB internal structures are constructed using proven methods common to heavy industrial construction. Special, new, or unique construction techniques are not used.

Modular construction methods are used to the extent practical for prefabricating portions of the IRWST liner, refueling canal liner, reinforcing, concrete formwork, and other portions of the RB internal structures. Such methods have been used extensively in the construction industry. Rigging is pre-engineered for heavy lifts of modular sections. Permanent and temporary stiffeners are used on liner plate sections and other modularized items to satisfy code requirements for structural integrity of the modular sections during rigging operations.

Steel decking and plates and supporting steel beams may be used to form concrete floors. In these instances, the decking thickness is in addition to the floor thickness shown on the dimensional arrangement drawings, provided in Appendix 3B. The decking, plates, and beams may be left in place, in which case they are designed for applicable seismic loads and other loading conditions. Other types of formwork that may also be used is left in place and become a permanent part of the structure. Such items conform to code requirements and are designed to prevent their failure from affecting Seismic Category I SSC.

3.8.4.2 Applicable Codes, Standards, and Specifications

The following codes, standards, specifications, design criteria, regulations, and regulatory guides are used for the design, fabrication, construction, testing, and inservice inspection of Seismic Category I structures other than the RCB and RB internal structures (GDC 1, GDC 2, GDC 4, and GDC 5).

3.8.4.2.1 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 - Hot-Weather Concreting.
- ACI 306R-88 (Re-approved 2002) - Cold-Weather Concreting.
- ACI 306.1-90 (Re-approved 2002) - Standard Specification for Cold Weather Concreting.
- ACI 308R-01 - Guide to Curing Concrete.
- ACI 308.1-98 - Standard Specification for Curing Concrete.
- 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).
- 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).

03.08.03-4

- AISC 303-~~05~~00 - 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.~~

03.08.03-4

- ANSI/ANS-6.4-2006 - Nuclear Analysis and Design of Concrete Radiation Shielding for Nuclear Power Plants (Reference 4).

- AISC 348-~~04~~00/~~2004~~2000 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.

- 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.

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.

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.

03.08.03-4

The design of bolted connections is in accordance with ANSI/AISC N690, Section Q1.16 and AISC 348-~~04~~00/~~2004~~2000 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.

Flood Design

In addition to designing for the external flood loads described in Section 3.8.4.3.1, Seismic Category I structures are protected against external flooding by the following methods:

- Exterior wall penetrations below plant flood level are sealed to prevent flood waters from entering Seismic Category I buildings.
- Finished yard grade around Seismic Category I structures is sloped to direct flood water and runoff away from the structures.
- Finished floor elevations are at one foot above plant finished grade where openings are provided for personnel and maintenance access.
- Water stops are provided in below grade exterior construction joints.
- Floor drainage is provided for building interior floors to collect water that could potentially enter the buildings.

See Section 3.4 for additional information on flood protection.

3.8.4.4.2 Reactor Shield Building and Annulus, Fuel Building, and Safeguard Buildings – NI Common Basemat Structure

Loads from the loading combinations described in Section 3.8.4.3 are applied to the NI Common Basemat Structure, which includes the RSB, the FB, and SBs. Vertical loads transfer to the NI Common Basemat Structure foundation basemat through concrete exterior walls, concrete interior walls, and concrete and steel columns. Lateral loads transfer to the NI Common Basemat Structure foundation basemat by diaphragm action of the concrete roof slabs and intermediate concrete floor slabs, which transfer loads to the interior and exterior concrete shear walls. Lateral loads transfer to the soil subgrade by friction and passive earth pressure.

The reinforced concrete roof slabs and intermediate floor slabs are analyzed and designed as two-way slabs. Reinforced concrete walls are analyzed and designed as shear walls and compression members, which are also subjected to out-of-plane bending moments, torsion, and out-of-plane shear. Analysis and design of the NI Common Basemat Structure foundation basemat is addressed in Section 3.8.5.

03.08.05-8 (Part 8)

The dynamic increment for the seismic soil surcharge loads on the exterior below-grade walls of NI Common Basemat Structure is determined by multiplying the surcharge static load by the maximum vertical zero period acceleration (ZPA) at the ground surface determined in the seismic analyses described in Section 3.7.2. Lateral pressure due to seismic loads for the below grade NI perimeter walls are obtained from a FEM SSI analysis. Seismic-induced lateral soil pressures on below-grade walls are evaluated for the following cases:

03.08.05-8 (Part 8)

- The seismic soil pressure is equal to the sum of the static earth pressure plus the dynamic earth pressure calculated in accordance with ASCE 4-98, Section 3.5.3.2.
- The seismic soil pressure is equal to the passive earth pressure for structures other than NI common basemat structures.

The NI Common Basemat Structure is included in the ANSYS V10.0 SP1 finite element overall computer model of the NI Common Basemat Structure that is described in Section 3.8.1.4.1. The NI Common Basemat Structure model includes the RSB, FB, and SBs as well as the RCB, RB internal structures, and the NI Common Basemat Structure foundation basemat that are described in other sections. 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 NI Common Basemat Structure is modeled using a mesh of ANSYS finite elements representing primary load-carrying walls, floors, columns, and beams. 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. 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.

ANSYS SHELL43 solid shell elements are used to model walls and floors and other concrete elements in the NI Common Basemat Structure. SHELL43 is a three-dimensional, four-node shell element that is suitable for moderately thick shell structures. SHELL43 can also provide out-of-plane shear forces and has an elastic-plastic capability. BEAM44 members are used to model beams and columns. The ANSYS finite element computer program is used to analyze the NI Common Basemat Structure for the loads and load combinations described in Section 3.8.4.3.

The finite element model used for the analysis of the NI Common Basemat Structure is shown in Figure 3.8-86—ANSYS Finite Element Model of Nuclear Island Common Basemat Structure - Outside View, Figure 3.8-87—ANSYS Finite Element Model of Nuclear Island Common Basemat Structure - Section Through Fuel Building and Safeguard Building 2/3 Island, and Figure 3.8-88—ANSYS Finite Element Model of Nuclear Island Common Basemat Structure - Section Through Safeguard Buildings 4 and 1.

Local analyses are used to analyze other Seismic Category I structures for locally applied loadings that have no significant effect on the overall behavior of the structures. Local analyses are performed for the pipe rupture loads described in Section 3.8.4.4.1 and for the missile impact loads also described in Section 3.8.4.4.1, as well as for other loadings and local structural areas.

3.8.5 Foundations

3.8.5.1 Description of the Foundations

Foundations for Seismic Category I structures are provided for the following buildings and structures:

- NI Common Basemat Structure foundation basemat.
- EPGB foundation basemats.
- ESWB foundation basemats. The ESWBs house the ESWCTs and the ESWPBs.

Foundations for buried items are included in Section 3.8.4. Section 3.7.2 addresses design requirements for Non-Seismic Category I structures to preclude adverse interaction effects on Seismic Category I structures.

Figure 3B-1 provides a site plan of the U.S. EPR standard plant showing the outline of the foundation basemats for the NI Common Basemat Structure, EPGBs, and ESWBs, along with the location of each foundation basemat.

Structures described within this section are not shared with any other power plant units (GDC 5).

A COL applicant that references the U.S. EPR design certification will describe site-specific foundations for Seismic Category I structures that are not described in this section.

3.8.5.1.1 Nuclear Island Common Basemat Structure Foundation Basemat

03.08.05-1

The NI Common Basemat Structure foundation basemat is a heavily reinforced concrete slab that supports the NI Common Basemat Structure Seismic Category I structures. The RCB and the RSB are located near the center of the NI Common Basemat Structure foundation basemat, and they are surrounded by the FB and the four SBs. The NI Common Basemat Structure foundation basemat is a cruciform shape that has outline dimensions of approximately 360 feet by 360 feet by 10 feet thick~~, a foundation basemat of lesser thickness will be considered for rock sites~~. The bottom of the NI Common Basemat Structure foundation basemat is founded approximately at elevation -41 feet and is embedded into the supporting soil approximately 40 feet~~below plant grade~~. The NI Common Basemat Structure foundation basemat outline and section views are presented in Figures 3B-1, 3.8-11, 3.8-12, 3.8-13, 3.8-50, 3.8-51, 3.8-52, 3.8-63, 3.8-74, and 3.8-85.

The NI Common Basemat Structure foundation basemat provides anchorage of the vertical post-tensioning tendons in the RCB, which is described in Section 3.8.1. The portion of the NI Common Basemat Structure foundation basemat that is considered to provide support and anchorage for the RCB is the area under the circumference of the outer face of the RSB wall, as shown on Figure 3.8-11, Figure 3.8-12 and Figure 3.8-13. This portion of the NI Common Basemat Structure foundation basemat is designed in accordance with the ASME BPV Code 2004 Edition, Section III, Division 2. A circular gallery is provided beneath the NI Common Basemat Structure foundation basemat for maintenance access to the bottom of the vertical post-tensioning tendons provided in the RCB shell wall. The tendon access gallery is approximately 11 feet wide by 14 feet high, including an approximately 36 inch thick foundation slab under the gallery structure. No credit is taken in the design for the tendon gallery transmitting vertical loads into the soil ~~in vertical or horizontal bearing. C~~,and the connection of the tendon gallery to the NI Common Basemat Structure foundation basement allows for differential movement between the concrete structures. However, the tendon gallery acts as a shear key and transfers lateral loads into the basemat.

03.08.05-8 (part 8)

Sections 3.8.1 and 3.8.3 describe the interface of the RCB containment liner plate and upper internal basemat above the liner for supporting the RB internal structures. Sections 3.8.4 describes the interface of the RSB, FB, and SBs with the NI Common Basemat Structure foundation basemat. Concrete walls and columns of these NI Common Basemat Structure Seismic Category I structures are anchored into the NI Common Basemat Structure foundation basemat with reinforcing bars to transmit vertical, horizontal, and bending moment loads into the basemat and to enhance the rigidity of the basemat.

Horizontal shear loads are transferred from the NI Common Basemat Structure foundation basemat to the underlying soil by friction between the bottom of the basemat, mud mat (or both), and the soil, and by passive earth pressure on the below-grade walls of the NI Common Basemat Structure Seismic Category I structures and tendon gallery are analyzed to act as shear keys~~; shear keys are not used~~. Section 2.5.4.2 describes the friction coefficient properties of soil addressed for the U.S. EPR.

Buildings adjacent to the NI Common Basemat Structure are separated from the NI Common Basemat Structure foundation basemat to allow for differential seismic movements between buildings. Refer to Figure 3B-1, which illustrates the gaps between buildings.

Waterproofing membranes used under or within the NI Common Basemat Structure foundation basemat will be evaluated on a site-specific basis, as described in Section 3.8.5.6.

3.8.5.1.2 Emergency Power Generating Buildings Foundation Basemats

Each EPGB foundation basemat supports a building superstructure and associated equipment. At the super-structure and foundation basemat interface, heavily where:

F_b = the buoyant force of the design basis flood at maximum site water level. Refer to Section 3.8.4.3.1 for definitions of the other load parameters.

The U.S. EPR Seismic Category I foundations are also designed for the effects of short term and long term settlements. Section 2.5 provides the settlement limits considered for the U.S. EPR.

There are no OBE loads applicable to the design of Seismic Category I foundations, since an OBE level of one-third the SSE has been selected. See Section 3.7 for a description of the OBE.

3.8.5.4 Design and Analysis Procedures

Design and analysis procedures are similar for the various Seismic Category I foundations but vary somewhat from structure to structure. The general analysis and design procedures applicable to Seismic Category I foundations are provided in the following sections. Procedures specific to the following Seismic Category I foundations also are described.

- NI Common Basemat Structure foundation basemat.
- EPGBs foundation basemats.
- ESWBs foundation basemats.

3.8.5.4.1 General Procedures Applicable to Seismic Category I Foundations

Concrete foundation basemats for Seismic Category I structures are analyzed as flat slabs on elastic supports to represent the underlying soil. The underlying soil medium is represented by FEM for SSI analyses for the NI and by soil springs for other Category I structures as described in subsequent sections. Loads are applied to the foundation basemats by the interfacing reinforced concrete walls and structural steel columns that comprise the building structures being supported, as well as by equipment supported directly on the foundations. Intersecting concrete walls also serve to stiffen the foundation basemat slabs to increase resistance to bending moments resulting from soil pressures under the slabs. Foundations are analyzed for the various factored loads and load combinations identified in Section 3.8.5.3.

03.08.05.05-8 (part 8)

Seismic Category I foundation basemat structures transfer vertical loads from the buildings to the subgrade by direct bearing of the basemats on the subgrade. Horizontal shears, such as those produced by wind, tornados, and earthquakes are transferred to the subgrade by friction along the bottom of the foundation basemat, shear key, or by passive earth pressure ~~(or both)~~. Waterproofing membranes used under or within the Seismic Category I foundations will be evaluated on a site-specific basis, as described in Section 3.8.5.6.

Design and analysis procedures for Seismic Category I foundations are the same as those described in Sections 3.8.1.4 and 3.8.4.4 for the respective structures that apply loads on the foundations.

Seismic Category I concrete foundations are designed in accordance with ACI 349-01 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 3.8.5.3. In addition, the portion of the NI Common Basemat Structure foundation basemat that supports the RCB is designed in accordance with the ASME BPV Code–2004 Edition, Section III, Division 2 for support and anchorage of the concrete RCB as described in Section 3.8.1.

The design of concrete foundations for Seismic Category I structures is performed using the strength-design methods described in ACI 349-01, with the exception that a shear reduction factor of 0.85 is used as allowed in ACI 349-06 (Reference 39). The ductility provisions of ACI 349-01 are satisfied to provide a steel reinforcing failure mode and to prevent concrete failure for design basis loadings.

Foundation design is performed for the spectrum of soil cases described in Section 3.7.1. Section 2.5 and Section 3.7 describe seismic parameters and design methods used for analyzing and designing Seismic Category I structures.

Soil-structure interaction and structure-soil-structure interaction effects are considered in the seismic analyses of Seismic Category I structures as described in Section 3.7.2. Figure 3B-1 illustrates separation distances between Seismic Category I structures upon which these interaction evaluations are based.

03.08.05-8 (Part 8)

The NI Common Basemat Structure is designed for an average static soil bearing pressure of 14,500 pounds per square foot and a maximum static bearing pressure of ~~22,000~~34,560 pounds per square foot. Accordingly, Seismic Category I foundations are sized and reinforced to accommodate these bearing pressure values.

The following criteria apply for load combinations for concrete and steel Seismic Category I foundations:

- The one-third increase in allowable stresses for concrete and steel members due to seismic (E') or wind (W and W_t) 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.

- 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.

- 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.

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

Loads and load combinations defined in Section 3.8.5.3 are used to determine strength requirements of members and elements of Seismic Category I foundations. 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 3.8.5.3.

For the loading combinations identified in Section 3.8.5.3, the minimum factors of safety required to prevent sliding and overturning are specified in Table 3.8-11—Minimum Required Factors of Safety Against Overturning, Sliding, and Flotation for Foundations.

03.08.05-8 (Part 8)

Normal lateral earth pressure loads consider saturated soil up to a groundwater elevation of -3.3 feet relative to site finished grade. Lateral soil loads due to external floods consider saturated soil up to elevation -1.0 feet relative to site finished grade. Seismic loads from all 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 same as described in Section 3.8.4.4. The SSE components of soil loads are determined using densities for saturated soil to account for the weight of the soil plus the weight of either normal or flood water levels. Earthquake-induced lateral soil pressures are obtained from SSI analyses for NI common basemat structures and are developed in accordance with Section 3.5.3 of ASCE 4-98 for the other Category I structures. The design of embedded elements, such as embedded walls on basemats, assumes that the lateral pressure due to the SSE is in phase with the inertial loads. In cases where passive pressure is assumed to act on embedded structures in the stability check against sliding, the walls of the structure are evaluated to withstand such earth pressure. Section 3.8.4.4.2 provides further information on how seismic-induced lateral earth pressures are determined for the NI Common Basemat Structure. These lateral load effects are considered in structure sliding and overturning analyses. Refer to

Section 2.5.4.2 for the soil parameters used to determine soil loads and lateral earth pressure.

When the effects of vertical seismic acceleration are included in the stability check against sliding, the unfactored dead weight of the structure is used to calculate the resistance to sliding due to friction.

Buoyancy effects of saturated soil due to a groundwater level of elevation -3.3 feet below finished grade or to a flood water level of elevation -1.0 feet below finished grade are considered when performing sliding and overturning analyses. For uplift evaluations (i.e., flotation and seismic overturning), dead load includes the weight of water permanently stored in pools and tanks. ~~Justification is provided for live loads that are included in loading combinations when evaluating structures for the effects of sliding and overturning.~~

03.08.05-8 (Part 8)

The effects of differential foundation settlements are applied concurrently with the dead load using the same load factors. Also, the effects of varying settlements between adjacent foundations are considered for the design of mechanical and electrical systems (e.g., piping, cables) that are routed between structures founded on separate basemats. See Section 3.8.4.4.5 for analysis and design procedures for Seismic Category I buried items that interface with structures on separate foundations.

3.8.5.4.2 Nuclear Island Common Basemat Structure Foundation Basemat

The NI Common Basemat Structure foundation basemat is analyzed and designed using the ANSYS V10.0 SP1 finite element overall computer model (a static model) for NI Common Basemat Structure Seismic Category I structures, which is described in Section 3.8.1.4.1. The NI Common Basemat Structure model includes the RCB, RB internal structures, RSB, FB, and SBs, as well as the NI Common Basemat Structure foundation basemat. This model is also used to determine the static bearing pressure on the supporting soils. ~~A second model (a dynamic model)~~The dynamic model is used to determine dynamic soil bearing pressures as well as sliding and overturning factors of safety.

ANSYS SOLID45 solid elements are used to model the concrete basemat foundation in the NI Common Basemat Structure static analysis. SOLID45 is a three-dimensional, eight-node element that is suitable for moderately thick structures. Depending on the thickness of the basemat, between three to five layers of SOLID45 elements are used in the model, with an average of four elements in the typical 10 feet thick basemat areas. Figure 3.8-103—Nuclear Island Common Basemat Structure Foundation Basemat ANSYS Model illustrates the model used for design of the basemat.

Springs are used to represent soil that provides support for the concrete foundation basemat in the ANSYS model. These springs represent the compressibility of the soil and were developed to reflect the pressure distribution under the NI Common

Basemat Structure. Springs values vary for each soil case based on the soil properties and the spring location under the modeled foundation. The distribution used is elliptical in nature and takes the form of:

$$K(x,y) = K_o[A - B^*(1 - x^2/2l^2 - y^2/2b^2)^{1/2}]$$

where:

K(x, y) is the subgrade modulus at x, y corrected for mat stiffness (pounds/ft^2 per foot)

K_o is the weighted average subgrade modulus (pounds/ft^2 per foot)

A & B are constants for a soil type based on its properties, bearing pressure distribution and shape of the foundation.

x = is the coordinate in the length direction of the Foundation Mat (feet)

y = is the coordinate in the width direction of the Foundation Mat (feet)

b = half width of foundation

l = half length of foundation.

03.08.05-8 (Part 8)

The Gazetas equation (Reference 57) was used to evaluate the total soil spring (K_o) for the design of the foundation basemat of the NI Common Basemat Structure. 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. Table 3.8-13—Static Spring Distribution provides the distribution equations and K_o values for each soil case.

Soil stiffness springs are modeled through the use of contact elements applied to the base of the NI Common Basemat Structure. These elements do not allow tension force transfer between the soil and the foundation. Sliding is not modeled in the static analysis. Figure 3.8-106—Elastic Displacement for Soil Case 1u, Figure 3.8-107—Elastic Displacement for Soil Case 2u, Figure 3.8-108—Elastic Displacement for Soil Case 1n2u, Figure 3.8-109—Elastic Displacement for Soil Case 3u, Figure 3.8-110—Elastic Displacement for Soil Case 4u, Figure 3.8-111—Elastic Displacement for Soil Case 5a, Figure 3.8-112—Elastic Displacement for Soil Case 5u, Figure 3.8-113—Elastic Displacement for Soil Case 2sn4u, Figure 3.8-114—Elastic Displacement for Soil Case 2n3u, and Figure 3.8-115—Elastic Displacement for Soil Case 3r3u illustrate elastic displacements, from loading, and dead load + 0.25* live load + equipment load using the springs listed in Table 3.8-13.

Tri-linear soil springs are developed for design of the foundation basemat for soil cases 4u and 2sn4u, as defined in Section 3.7.1, in order to mitigate unrealistic analysis results generated by the NI Common Basemat Structure static model. Seismic forces were conservatively applied using maximum ZPA accelerations from the soil structural interaction (SSI) analysis for points throughout the structure. These accelerations are applied to the building masses simultaneously, without consideration of timing. This methodology results in conservative sets of seismic forces, in some cases base shears are 20 percent to 55 percent larger than those calculated by the SSI analysis, applied to the structure. When these conservatively high forces are applied to soils represented by stiff springs the resulting overturning moment is exaggerated and skews the analysis results. The introduction of tri-linear springs to the model mitigates the exaggerated response.

Tri-linear springs development uses the linear development as the starting point. The subsurface soil is assumed to be relatively high plasticity clay. Based on the modulus degradation for clays with plasticity index in the range 50 to 70, a relationship is developed between displacement of the foundation basemat and the corresponding average reaction imposed by the underlying soil medium on the foundation basemat. Using an incremental approach, the methodology calculates the reaction at the base of the foundation basemat for a small increment of basemat displacement, using the appropriate soil spring associated with the shear modulus at this step. In the next incremental step, the solution is advanced using a reduced shear modulus consistent with the shear strain at a representative depth associated with the soil reaction from the previous step. For the two aforementioned soil cases (4u and 2sn4u) the resultant bearing pressure versus subgrade modulus values are provided in Table 3.8-14—Tri-Linear Subgrade Modulus vs. Bearing Pressures.

The results of the soil spring analyses are used in determining forces and moments in the basemat for concrete design and for determining the acceptability of the supporting soil media under static loading conditions.

03.08.05-8 (Part 8)

A ~~second model was developed~~FEM model for SSI analysis of the embedded portions of the NI common basemat was used to evaluate the soil bearing pressures, sliding and overturning due to seismic events. This model explicitly represents ~~the nonlinearities of sliding and uplift,~~ the transient nature of the seismic loadings, the properties of the soils, and the dynamic characteristics of the structure. This approach produces a more realistic picture of the NI Common Basemat Structure response to seismic loadings than is possible using the static model alone.

The NI Common Basemat Structure superstructure is modeled using lumped parameter systems identical to those used for the soil-structure interaction analysis. The masses, stiffnesses, and eccentricities of the buildings are mathematically computed, and spatially arranged ~~and tuned (to the fundamental frequencies of the SSI model)~~ to ~~correctly~~ represent the dynamic characteristics of the NI Common Basemat structures. ~~The basemat is modeled using shell elements and properties are provided to make this surface as rigid as practical for analytical purposes. The NI Common~~

~~Basemat Structure basemat is very rigid by virtue of its thickness and many stiffened walls.~~

~~Soil is modeled with one layer of solid elements beneath the shells, representing the basemat. Properties of these solids are established in a way that ultimately allows the model to respond in agreement with the SASSI analysis fundamental modes (see Table 3.8-15—DeletedFundamental Mode Frequencies for Dynamic Model Tuning).~~

~~Interaction of the basemat shells and soil elements is modeled with contact elements; tension in the soil is not allowed. Shear resistance at the interface is modeled by setting the shear coefficient to a constant value of μ = 0.7, μ = tan φ where φ=35° for soil. Passive lateral soil pressure is also modeled and the full passive pressure is assumed to mobilize at a horizontal displacement of one percent of the embedded depth (see Figure 3.8-116—Passive Soil Pressure). Horizontal springs with constants representing this curve are connected to the basemat for conservatism, as opposed to some distance up the embedded walls.~~

~~Damper elements (dashpots) are connected to the element upper and lower nodes to account for soil damping in the three principal directions. These values are derived from the SASSI analysis transfer functions. A constant 5 percent critical damping is used for the superstructure.~~

The model is excited by simultaneous application of three EUR seismic transients (CSDRS) to the base of the ~~soil elements~~foundation basemat~~,~~ for soil cases ~~1u,~~ 2sn4u, ~~2n3u, 2u,~~ 4u, and 5a representing soft, medium and hard soils. Transients are applied, one each, in the three principal building directions. The weight of the building,~~;~~ including the water in the in-containment refueling water storage tank (IRWST), fuel pool, and the four emergency feedwater storage tanks (because this water is always present within the NI Common Basemat Structure), and full buoyancy are the other loadings included in this analysis. ~~Active dynamic earth pressure is small when compared to the building seismic shear and is not considered in this model.~~

~~Figure 3.8-16—DeletedDynamic Analysis Results summarizes the results of the non-linear dynamic analysis.~~

03.08.05-8 (Part 8)

Section 3.8.1, Section 3.8.3, and Section 3.8.4 provide descriptions of interfacing structures that induce loads on the NI Common Basemat Structure foundation basemat. The figures in those sections illustrate the concrete shear walls and columns that transfer loads to the NI Common Basemat Structure foundation basemat. The tendon gallery beneath the NI Common Basemat Structure foundation basemat is ~~not~~ relied upon as a shear key to aid in resisting lateral forces on the basemat. ~~The area under the tendon gallery is not considered in transferring vertical forces into the underlying soil media.~~

foundation basemat that support the RCB are within the limits in accordance with ASME BPV Code, Section III, Division 2.

Seismic Category I foundations are required to satisfy the factors of safety against overturning, sliding, and flotation defined in Table 3.8-11. The calculated minimum factors of safety for the NI Common Basemat Structure are provided in Table 3.8-12—Minimum Factors of Safety Against Overturning, Sliding, and Flotation for Foundations – NI Common Basemat Structure. ~~For the load combination containing seismic loads, the calculated minimum factors of safety are less than the values provided in NUREG-0800, for overturning and sliding of the NI Common Basemat Structure. The acceptability of these calculated values is further addressed in the following section for the NI Common Basemat Structure foundation basemat.~~

03.08.05-8 (Part 8)

Acceptance criteria for soil conditions for the media supporting Seismic Category I foundations are addressed in Section 2.5.

Acceptance criteria for settlement for Seismic Category I foundations are addressed in Section 2.5.

Additional acceptance criteria for critical areas of these structures are described in Appendix 3E.

A COL applicant that references the U.S. EPR design certification will evaluate site-specific methods for shear transfer between the foundation basemats and underlying soil for soil parameters that are not within the envelope specified in Section 2.5.4.2.

3.8.5.5.1 Nuclear Island Common Basemat Structure Foundation Basemat

Appendix 3E provides details of the design of the NI Common Basemat Structure foundation basemat critical areas.

Maximum soil bearing pressures under the NI Common Basemat Structure foundation basemat are 22,000 pounds per square foot for static loading conditions, and ~~34,560~~26,000 pounds per square foot for dynamic loading conditions.

The NI Common Basemat Structure foundation basemat for the U.S. EPR plant design can accommodate tilt settlements up to 0.5 inches in 50 feet in any direction across the basemat, as described in Section 2.5.4.10.2. Differential settlements and local settlements within the perimeter of the foundation, are not likely to affect the structure, systems, or components due to the extremely thick foundation stiffened by numerous shear walls. The combined stiffness allows the NI Common Basemat Structure foundation basemat to bridge potential foundation irregularities.

03.08.05-8 (Part 8)

For worst-case loading combinations on the NI Common Basemat Structure foundation basemat, the ~~conservative~~time history methodology used to calculate sliding and uplift safety factors due to seismic loadings is described in Section 3.8.5.4.2. The calculated values meet the requirements of Table 3.8-11.~~, as provided in Table 3.8-16, are sufficiently small that they can be considered inconsequential with respect to sliding and overturning.~~

03.08.05-8 (Part 8)

For worst-case loading combinations on the RB internal structures basemat above the containment liner, the minimum safety factor against sliding is 0.16 occurring for soil case 2sn4u, based solely on friction between the liner and the supporting concrete. Because friction will not prevent sliding, the surrounding concrete haunch wall is designed with sufficient capacity to resist the total base shear force. The minimum safety factor against overturning is 1.22 occurring for soil case 2sn4u.

3.8.5.5.2 Emergency Power Generating Buildings Foundation Basemats

Appendix 3E provides details of the design of the EPGB foundation basemats critical sections.

~~Evaluation of the EPGB foundation basemat for maximum bearing pressures under static and dynamic loading conditions, settlements, flotation, sliding, and overturning will be performed to confirm that applicable acceptance criteria are met.~~Maximum soil bearing pressures under the EPGB foundation basemat are 3,800 pounds per square foot for static loading conditions, and 10,800 pounds per square foot for dynamic loading conditions. The factors of safety against overturning, sliding, and flotation are each greater than or equal to 1.1.

03.08.05-10

3.8.5.5.3 Essential Service Water Building Foundation Basemats

Appendix 3E provides details of the design of the ESWB foundation basemats critical sections.

~~Evaluation of the ESWB foundation basemat for maximum bearing pressures under static and dynamic loading conditions, settlements, flotation, sliding, and overturning will be performed to confirm that applicable acceptance criteria are met.~~Maximum soil bearing pressures under the ESWB foundation basemat are 17,800 pounds per square foot for static loading conditions, and 28,200 pounds per square foot for dynamic loading conditions. The factors of safety against overturning, sliding, and flotation are each greater than or equal to 1.1.

3.8.5.6 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 Seismic Category I foundations.

40. ACI 311.4R-05, “Guide for Concrete Inspection,” American Concrete Institute, Inc., 2005.

41. ACI 349.1R-07, “Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures,” American Concrete Institute, Inc., 2007.

03.08.03-4

42. AISC 303-~~05~~00, “Code of Standard Practice for Steel Buildings and Bridges,” American Institute of Steel Construction, Inc.

43. ~~ANSI/AISC 341-05, “Seismic Provisions for Structural Steel Buildings,” American National Standards Institute, 2005.~~

44. AISC 348-~~04~~00, “Specification for Structural Joints Using ASTM A325 or A490 Bolts,” American Institute of Steel Construction, ~~2004~~2000.

45. ANSI/AWS D1.8-2005, “Structural Welding Code – Seismic Supplement.” American National Standards Institute, 2005.

46. ASTM F1554-07, “Standard Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength,” American Society for Testing and Materials, 2007.

47. ASTM C150-07, “Standard Specification for Portland Cement,” American Society for Testing and Materials, 2007.

48. ASTM C595-07, “Standard Specification for Blended Hydraulic Cements,” American Society for Testing and Materials, 2007.

49. ACI 306R-88, “Cold-Weather Conditioning,” American Concrete Institute, 1988.

50. ACI 308R-01, “Guide to Curing Concrete,” American Concrete Institute, 2001.

51. ASTM A82-07, “Standard Specification for Steel Wire, Plain, for concrete Reinforcement,” American Society for Testing and Materials, 2007.

52. ASTM A185-07, “Standard Specification for Steel Welded Wire Reinforcement Plain for Concrete,” American Society for Testing and Materials, 2007.

53. ASTM A497-07, “Standard Specification for Steel Welded Wire Reinforcement Deformed for Concrete,” American Society for Testing and Materials, 2007.

54. ASTM 325-07, “Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength,” American Society for Testing and Materials, 2007.

55. ASTM A490-06, “Standard Specification for Structural bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength,” American Society for Testing and Materials, 2006.

56. ASTM A307-07, “Standard specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength,” American Society for Testing and Materials, 2007.

Table 3.8-10—Structural Bolting Nut and Washer Materials

Item	ASTM Designation
Nuts	A194 (all grades) A563 (all grades) F1852
Washers	F436

Note:

1. Use of A563 nuts conforms to Appendix X1 of ASTM A563, which provides guidance on the suitability of A563 nuts for specific bolting materials.

Table 3.8-11—Minimum Required Factors of Safety Against Overturning, Sliding, and Flotation for Foundations

Load Combination	Minimum Factors of Safety		
	Overturning	Sliding	Flotation
D + H + W	1.5	1.5	-
D + H + Wt	1.1	1.1	-
D + H + E’	1.1	1.1	-
D + Fb	-	-	1.1

Table 3.8-12—Minimum Factors of Safety Against Overturning, Sliding, and Flotation for Foundations – NI Common Basemat Structure

Load Combination	Minimum Factors of Safety		
	Overturning	Sliding	Flotation
D + H + W	261 (E-W) 492 (N-S)	25.0 (E-W) 72.5 (N-S)	-
D + H + Wt	216 (E-W) 274 (N-S)	24.1 (E-W) 66.5 (N-S)	-
D + H + E’	~~See 3.8.5.4.2~~2.00 (E-W) 1.78 (N-S)	~~See 3.8.5.4.2~~1.16 (E-W) 1.58 (N-S)	-
D + Fb	-	-	4.9

03.08.05-8 (Part 8)

Table 3.8-15—Deleted~~Fundamental Mode Frequencies for Dynamic Model Tuning~~

~~Soil Case~~	~~Concentrated Springs & Stick Model (Equivalent to SASSI Analysis)~~			~~Non-Linear Dynamic Model (Distributed Springs under Shell)~~			~~Non-Linear Dynamic Model (Solid elements under Shell)~~		
	~~Fx (Hz)~~	~~Fy (Hz)~~	~~Fz (Hz)~~	~~Fx (Hz)~~	~~Fy (Hz)~~	~~Fz (Hz)~~	~~Fx (Hz)~~	~~Fy (Hz)~~	~~Fz (Hz)~~
~~2n3u~~	~~2.18~~	~~2.09~~	~~3.84~~	~~2.24~~	~~2.28~~	~~3.82~~	~~2.13~~	~~2.18~~	~~3.81~~
~~2u~~	~~1.38~~	~~1.32~~	~~2.41~~	~~1.42~~	~~1.45~~	~~2.40~~	~~1.38~~	~~1.41~~	~~2.41~~
~~2sn4u~~	~~2.75~~	~~2.66~~	~~5.15~~	~~2.80~~	~~2.83~~	~~5.17~~	~~2.70~~	~~2.73~~	~~5.17~~
~~4u~~	~~3.12~~	~~3.02~~	~~5.84~~	~~3.16~~	~~3.17~~	~~5.91~~	~~3.18~~	~~3.17~~	~~6.15~~
~~5a~~	~~4.14~~	~~3.98~~	~~8.91~~	~~4.15~~	~~3.99~~	~~9.12~~	~~4.04~~	~~3.90~~	~~8.99~~

03.08.05-8 (Part 8)

Table 3.8-16—Deleted~~Dynamic Analysis Results~~

~~Soil Case Name~~	~~Maximum Bearing Pressure~~		~~Maximum Sliding X~~		~~Maximum Sliding Y~~		~~Maximum Uplift Z~~	
	~~Bearing Pressure~~	~~Time~~	~~Sliding -X East-West~~	~~Time~~	~~Sliding -Y North-South~~	~~Time~~	~~Uplift -Z~~	~~Time~~
	~~(ksf)~~	~~(s)~~	~~(inch)~~	~~(s)~~	~~(inch)~~	~~(s)~~	~~(inch)~~	~~(s)~~
~~5a~~	~~27.42~~	~~7.5150~~	~~0.0020~~	~~6.9075~~	~~0.0027~~	~~9.9125~~	~~0.0133~~	~~5.1125~~
~~4u~~	~~33.84~~	~~4.8950~~	~~0.3165~~	~~6.7850~~	~~0.2298~~	~~9.2325~~	~~0.2371~~	~~1.8050~~
~~2u~~	~~20.22~~	~~6.2250~~	~~0.0175~~	~~6.9750~~	~~0.0124~~	~~4.4175~~	~~0.1056~~	~~4.3425~~
~~2n3u~~	~~28.57~~	~~9.5575~~	~~0.0292~~	~~12.9350~~	~~0.0294~~	~~9.5200~~	~~0.4180~~	~~7.7525~~
~~2sn4u~~	~~34.56~~	~~9.8225~~	~~0.2381~~	~~7.1875~~	~~0.1600~~	~~7.6825~~	~~0.6604~~	~~5.2850~~
~~1u~~	~~17.25~~	~~10.4500~~	~~0.0071~~	~~9.7575~~	~~0.0056~~	~~12.8100~~	~~-0.0030~~	~~2.4800~~

3A Criteria for Distribution System Analysis and Support

This appendix provides the design criteria for the U.S. EPR distribution system analysis and supports. As noted in Section 3.7.3, this appendix describes criteria for design of supports for:

- Piping.
- Heating, ventilation, and air conditioning (HVAC) ducts.
- Cable trays.

3A.1 Piping and Supports

Information on piping, instrumentation, and supports is provided in AREVA NP Topical Report ANP-10264NP-A, "U.S. EPR Piping Analysis and Pipe Support Design" (Reference 1).

3A.2 Heating, Ventilation, and Air Conditioning Ducts and Supports

HVAC ductwork and its associated support structures are designed to withstand the loadings and load combinations presented in Section 3A.2.2 and Section 3A.2.3, based on the Codes and Standards provided in Section 3A.2.1. A typical HVAC duct system includes structural components (e.g., sheet metal ducts, duct stiffeners, duct supports) and inline components (e.g., heaters and dampers).

03.08.03-17

Safety-related, Seismic Category I HVAC ductwork, supports, and restraints meet the stress allowables provided in paragraph SA-4220 of ASME AG-1 (Reference 2). Seismic Category II HVAC ductwork, supports, and restraints are analyzed to make sure that a failure would not ~~adversely impact~~impair the safety function of safety-related equipment or components. ~~Seismic Category II requirements are satisfied by conservatively analyzing the Seismic Category II HVAC ductwork, supports, and restraints to the same criteria as Seismic Category I~~Seismic Category II requirements are satisfied by meeting the criteria as established in Section 3.7.2.3.3..

Non-Seismic HVAC ductwork meets Sheet Metal and Air Conditioning Contractors National Association (SMACNA) standards (Reference 5). Non-Seismic HVAC ductwork support and restraint systems meet the analysis requirements of the American Institute of Steel Construction (AISC) Manual (Reference 3).

3A.2.1 Codes and Standards

HVAC ductwork, ductwork supports, and ductwork restraints conform to the following codes and standards:

- ASME AG-1-2003, Code on Nuclear Air and Gas Treatment, with 2004 Addenda (Reference 2).